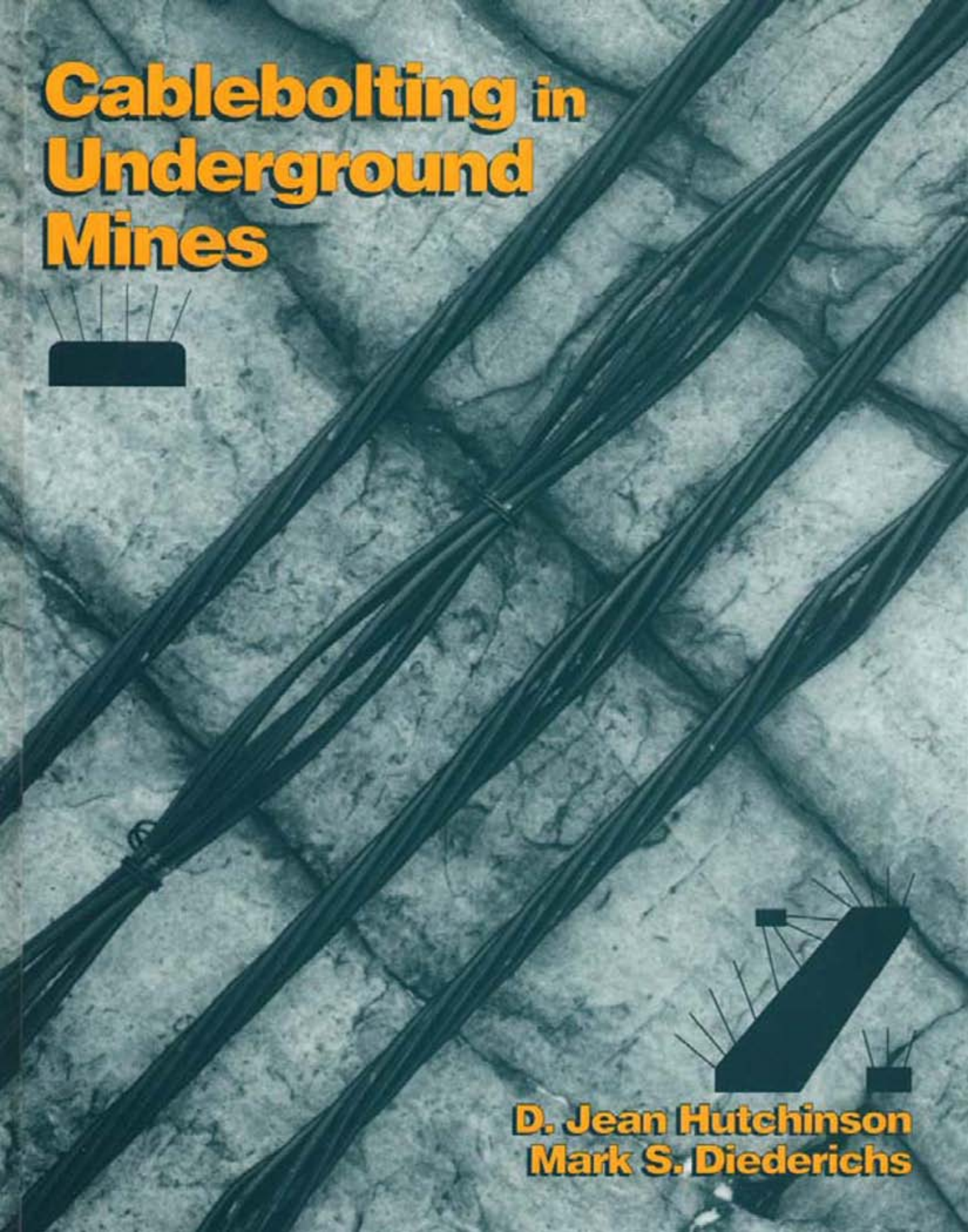

Cablebolting in Underground Mines

D. Jean Hutchinson
Mark S. Diederichs

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Modern cablebolt configurations (top to bottom):
plain strand, birdcaged strand, nutcaged strand, bulbed strand

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Foreword

The need for this book arose during an industry sponsored research project on rock support in underground hard rock mines carried out jointly at the Canadian Universities of Toronto, Queen's and Laurentian between 1989 and 1993. The topic of cablebolting could not be covered adequately within this broad research field and so a follow-up research project was initiated by the Geomechanics Research Centre, at Laurentian. This book is the end product of this research project which involved visits to mines in Canada, Australia and Papua New Guinea and a six month visit by the authors to the Rock Reinforcement Group of the C.S.I.R.O. in Perth, Australia.

Jean Hutchinson and Mark Diederichs, who were graduate students and research engineers in the Department of Civil Engineering at the University of Toronto at the start of the first project, carried out this follow-up work as a wife and husband team with the Geomechanics Research Centre in Sudbury. They brought many new insights and their own brand of enthusiasm to the project. The many hours of hard work which went into preparing this book are reflected in the clarity of the text and tables and the excellent quality of the numerous illustrations.

Cablebolting in Underground Mines contains a wealth of information which will be useful, not only to underground miners, but to anyone concerned with the design of support for underground excavations for any purpose in any kind of rock. This is the first book to bring together the practical details on grout mixes, grout pumps, cable characteristics and the theoretical background required for the rational design of cablebolt support systems. There are numerous mining and civil engineering projects around the world in which cables are being used for support and where the information contained in this book will be very valuable.

I commend the authors for their efforts in producing this fine volume and look forward to using it in my own consulting and educational activities in rock engineering.

Evert Hoek

Vancouver
October 1995

Preface

Cablebolts are high capacity flexible tendons composed of multi-wire strand which are normally installed and grouted in drilled holes at regular spacings to provide reinforcement and support of excavations in rock. They can be cut to any length and installed in single or multi-strand configurations and can be installed from small adits and tunnels (drifts) where limited clearance would preclude the use of rigid tendons.

The last decade has seen a dramatic increase in cablebolt usage in underground mining. There has been a corresponding increase in associated research around the world.

Researchers and industry experts came together for a day in June of 1992 during a cablebolting workshop held as part of the International Symposium on Rock Support in Sudbury, Canada. Interest in the topic was impressive as demonstrated by the standing-room only crowd at the workshop. In the aftermath of this gathering, authors Mark Diederichs and Jean Hutchinson of the Geomechanics Research Centre, together with Peter Kaiser and Dougal McCreath of Laurentian University and with Chris Windsor and Alan Thompson of the Rock Reinforcement Group (Commonwealth Scientific and Industrial Research Organization, C S.I.R.O., Australia), decided that there was a need to bring together, in an easy-to-use and comprehensive handbook, the current state-of-the-art in cablebolting.

Funding for the project, obtained from Canadian and Australian mining companies, was coordinated by the Mining Research Directorate (M.R.D.) in Canada and by the Australian Mineral Industries Research Association (A.M.I.R.A.). The authors visited 50 mines in Canada, Australia and Indonesia, observing mining techniques, ground conditions and cablebolting practice, consulting with mine staff to determine the current state-of-the-art in cablebolting and to tapping local expertise. The authors also communicated with numerous international researchers in the field.

The result is this comprehensive handbook, covering virtually all aspects of cablebolting, for support of underground excavations, from theory to practice with an emphasis on (but not restricted to) applications in the mining industry. It is an essential guide for the rock mechanics or ground control engineer in mining and in civil construction and is an excellent reference for researchers and developers in the field.

Recent innovations in cablebolting are presented and the current body of international research is summarised in the context of cablebolt type selection and support system design (Chapter 2). Installation and quality control procedures are outlined along with suggestions for crew training and management (Chapter 3). Practical techniques for support performance assessment and design verification (Chapter 4) round out the "Cablebolting Cycle" as introduced in Chapter 1.

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The contributions of the sponsoring companies and groups have made the whole project possible, for which we are extremely grateful. Thanks to Charlie Graham of the M.R.D. and to Jim May and Roger Wischusen of A.M.I.R.A. for coordinating the necessary funding for this work.

We are deeply indebted to all of the very busy ground control and mining engineers in Canada, Australia and Papua New Guinea, who guided us around their mines, and who discussed their cablebolting design, installation procedures, problems and solutions with us. This book could not have been written at all without their input. The mine site visits conducted during this project provided us with the necessary practical grounding for this handbook.

We are also pleased to have been able to work with a number of fine researchers prior to and during the course of this project, all of whom provided excellent opportunities for discussion and evolution of thought on cablebolts and rock mechanics. During the course of the project we gained a great deal from discussions with Will Bawden and Andy Hyett at Queens' University, Kingston, Canada; John Goris of the U.S. Bureau of Mines, Spokane, USA; Phil Oliver, formerly of INCO Mines Research, Sudbury, Canada; and Ernesto Villaescusa of Mount Isa Mines, Queensland, Australia.

In Australia, the entire Rock Reinforcement Group provided truly five-star hospitality and excellent technical assistance. Chris Windsor, Alan Thompson, Glynn Cadby, Wayne Robertson, Pat Carden, Bob Middleton, Rosalie Thompson, and Michele Currey made our stay in Perth a memorable personal and professional venture. Thanks in particular to Chris and Alan for making it all possible.

At home at the Geomechanics Research Centre and at Laurentian University, thanks are due to Peter Kaiser, Dougal McCreath, Dwayne Tannant and Véronique Falmagne. Thanks also to all the staff at the G.R.C. for their support.

A number of people have assisted by reading and providing feedback on the chapters of the book as they have evolved. At the risk of leaving someone out, we would like to thank Jane Alcott, Gary Auld, Trevor Carter, Chris Langille, Peter Mikula, Doug Milne, Marnie Pascoe and Norikazu Shimizu for their comments.

A very special acknowledgement is due to Evert Hoek who introduced us so elegantly and brilliantly to the subject of rock mechanics and whose leadership ultimately made much of this work possible.

Finally, we thank our families. The authors were married to each other just before the start of this project and remain happily married at the end. While we thank each other for the understanding and mutual support which made this possible, our families were invaluable in keeping real life and work in perspective.

1 INTRODUCTION: Cablebolting in Underground Mines

1.1 What is a Cablebolt?

A conventional cablebolt is a flexible tendon consisting of a number of steel wires, wound into strand, which is grouted into a borehole. Cablebolts are normally installed in regularly spaced boreholes to provide reinforcement and support for the walls, roof and floor of underground or surface openings.

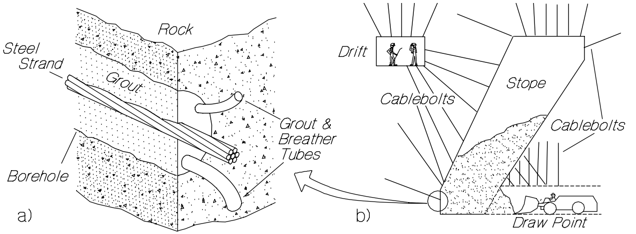


Figure 1.1.1: a) Cablebolt element b) Typical cablebolt array

Cablebolting is a very versatile form of support, since the cable strands can bend around fairly tight radii, making installation of long bolts from confined working places possible, and because they can be fabricated using a number of different configurations of the steel wires providing a variety of performance characteristics. It is not difficult to place more than one cablebolt strand in a single borehole, to increase tensile capacity, if the borehole diameter is large enough. In addition, face restraint can be attached in the form of plates, straps and mesh. Cablebolts can be used in combination with other support systems such as shotcrete, mechanical bolts or grouted rebar.

The capacity of the steel cablebolt element is transferred to the rockmass through grout. Grout used in cablebolting applications is usually composed of Portland cement and water. At some mine sites additives are added to the mix to improve the pumping characteristics of the grout. Other grouts, including resin and shotcrete have been investigated for certain cablebolting applications. As well, alternative materials such as fibreglass have been developed to replace the steel itself. This handbook will focus primarily on cablebolts developed from the conventional seven-wire steel strand and on cement grouts.

1.2 Why Cablebolt?

Cablebolts are used in underground hard rock mines to:

- provide a safe working environment,
- increase rockmass stability, and
- control dilution of waste rock from the stope boundaries.

In any mining or construction project, safety is of paramount importance. Different support methods such as mechanical rockbolts and screen, shotcrete or grouted rebar are normally employed in smaller span mining tunnels or drifts to protect workers from smaller blocks and loose rock which may fall from the roof or sidewall. For larger spans in major intersections, large underground chambers or in active mining stopes, cablebolts become an attractive support system due to the increased load capacity and the potential for increased bolt length. Larger spans in general mean greater potential for large free blocks or broken rock falls. This increased demand requires an increase in support system capacity which can be effectively provided by cablebolts to ensure adequate safety.

It should be noted that in areas where areal restraint systems such as screen are used to protect miners from smaller pieces of loose rock and to prevent surface unravelling of poor rockmasses, cables can be used to supplement but not to replace this form of restraint. This form of demand cannot be accommodated by cablebolts alone. Chapter 2 discusses demand-capacity relationships.

Cablebolts can reach far into the rockmass and reinforce large volumes of rock to prevent separation along planes of weakness such as joints. By maintaining a continuum nature within the rockmass, the cablebolts help to mobilize the inherent strength of the rockmass, thereby improving overall stability. In addition, by supporting blocks of rock at the excavation surface, the remaining rockmass is prevented from loosening and weakening. Cablebolts thus restrict the dangerous and costly effects of progressive instability and failure.

Cablebolts can be installed remotely in long boreholes to reach the planned stope boundary and provide pre-reinforcement to the otherwise inaccessible walls and backs created by today's bulk mining methods. Cables are one of the only options for support of inaccessible rock faces for stability and dilution control.

Dilution control can have a very direct and large influence on the cost of a stope. The cost of dilution is many-fold: waste rock with little or no economic value is mucked, trammed, crushed, skipped, milled and impounded in a tailings disposal area, all at great cost. In addition, the mill works at effectively only partial capacity, despite producing at the maximum possible milling rate. The unscheduled delays required to deal with oversize muck, mucking waste rock and with consequent changes to the mining schedule are also costly.

Anderson and Grebenc (1995) provide an excellent discussion of the several components of dilution as shown in Figure 1.2.1 and defined here. The required information is collected from a laser survey of each stope after mining is complete (see Section 4.2 for further discussion of laser stope surveying). Factors which must be considered in assessing the performance of a stope include:

$$\%\ Dilution = \frac{Waste\ dilution\ (t) + Backfill\ dilution\ (t)}{Planned\ tonnes\ (t)} \ x\ 100$$

$$\%\ Recovery = \frac{Planned\ tonnes\ (t) - Ore\ lost\ in\ stope\ (t)}{Planned\ tonnes\ (t)} \ x\ 100$$

$$\%\ Overbreak = \frac{Ore\ sloughing\ (t)}{Planned\ tonnes\ (t)} \ x\ 100$$

Dilution control has a high priority at Hemlo Gold mines. Every stope is surveyed, so that the factors listed above can be calculated. Anderson and Grebenc provide a very illustrative case history of dilution control through the understanding of the cause of failure in one stope and the effective design of support (cablebolt and drift backfill) for the adjacent stope. The difference in the cablebolting pattern and dilution of the stope walls is shown in Figure 1.2.2.

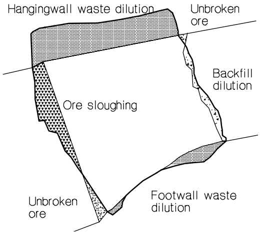


Figure 1.2.1: Definition of terms (after Anderson and Grebenc, 1995)

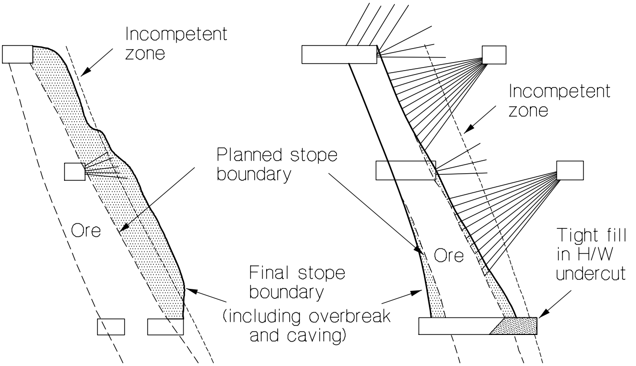


Figure 1.2.2: Cablebolting pattern and dilution surveyed in adjacent stopes at Hemlo Gold Mine (after Anderson and Grebenc, 1995)

Other examples illustrating the economic importance of rock dilution are given throughout the mining literature including Bawden (1993), Elbrond (1994), Pakalnis et al. (1995), Planeta et al. (1990), Planeta and Szymanski (1995), Stillborg (1986), and others. Many mining handbooks include calculations for tracking the progress of waste rock dilution through the mining and milling processes to determine its overall economic impact. A detailed treatment of mine economics, however, is beyond the scope of this handbook.

All modern mining will have some minimal dilution limit resulting from the smoothing of stope outlines to facilitate blasting or due to other sources of planned dilution. In many situations, particularly where there is a distinct ore/waste contact, unplanned dilution due to sloughing waste rock can quickly render the stope uneconomic. It is this unplanned dilution component which can be tackled through improved stope design and through the use of cablebolt support.

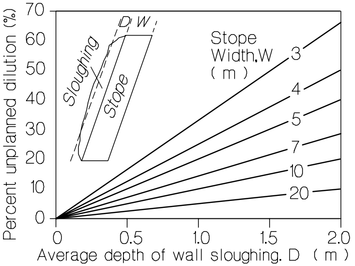


Figure 1.2.3: Dilution vs sloughing & span (after Pakalnis et al., 1995)

Many mines have found cablebolts to be effective in reducing or eliminating this sloughing and thereby reducing dilution. Alternatively, the use of cablebolts can facilitate the safe extraction of larger stopes normally resulting in increased productivity. Figure 1.2.3 shows theoretical dilution values as a function of span and sloughing depth for an unsupported stope of simplified geometry.

As this dilution moves through the system incurring additional mucking, haulage and hoisting costs as well as (and most importantly) displacing profitable ore (grade reduction) in the mill (Bawden et al., 1989), it becomes apparent that the economic consequences can be extreme (Figure 1.2.4). Additional costs and losses are incurred due to the effect of unplanned downtime required to handle oversized waste rock. In areas where cablebolting is effective in reducing dilution, the cost of cablebolting (Section 1.9) is often minuscule by comparison.

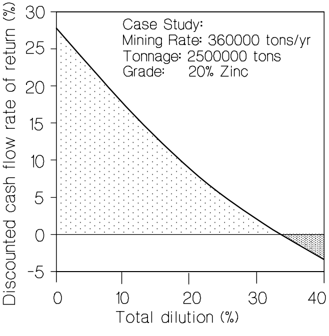


Figure 1.2.4: Economic impact of dilution (after Bawden, 1993)

1.3 Cablebolt Applications

Cablebolts can be used to support, reinforce or retain the rockmass around most excavations found at an underground mine, including:

- Drifts and intersections.
- Open stope backs.
- Open stope walls.
- Cut and fill stopes.
- Drawpoints.
- Permanent openings.

The particular cablebolt pattern selected will depend upon the intended function of the cablebolts and the access for installation. Access for cablebolting is usually provided by production drifts. While the cost of driving drifts solely for installing support is expensive, a number of mine sites have reduced the extent of rockmass failure around open stopes (and thereby dilution) by installing more effective cablebolt patterns from "cabling drifts" (see Figure 1.2.2). Some sketches showing examples of cablebolt layouts for different stope and access configurations are shown below. These patterns can be used individually or in combination to provide the most effective cablebolt pattern.

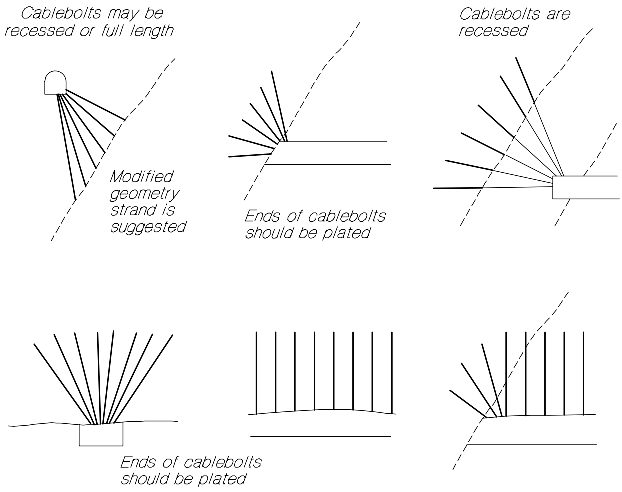
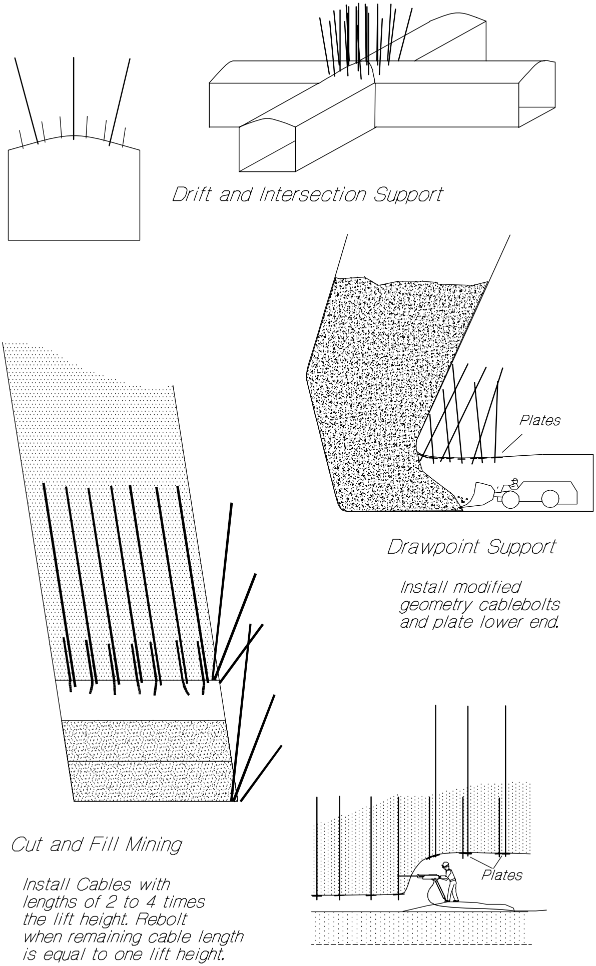


Figure 1.3.1: Cablebolt layout examples for mining excavations

Cablebolt Applications



Drift and Intersection Support

Drawpoint Support

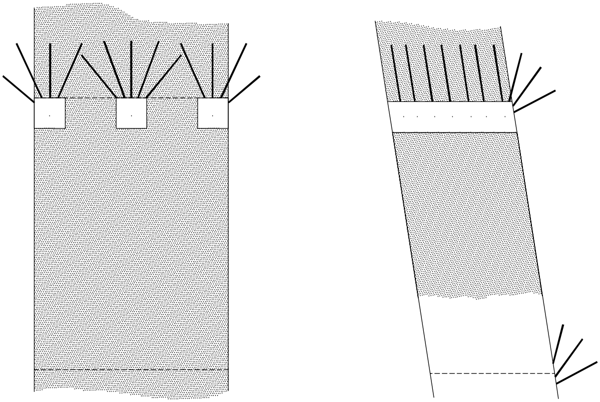
Install modified geometry cablebolts and plate lower end.

Cut and Fill Mining

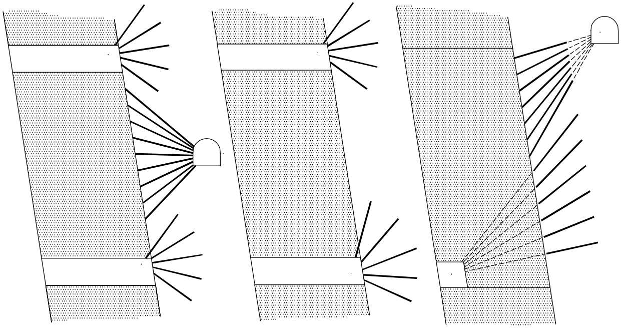
Install Cables with lengths of 2 to 4 times the lift height. Rebolt when remaining cable length is equal to one lift height.

Figure 1.3.2: Example cablebolt applications and layouts

Cablebolt Applications



Open Stope Back Support



Open Stope Hangingwall Support

Figure 1.3.3: Example cablebolt applications and layouts

1.4 The Cablebolting Cycle

The Cablebolting Cycle shown in Figure 1.4.1 provides a comprehensive overview of the steps involved in the cablebolting operation. It also forms the basis of the structure of this handbook: Chapter 2 discusses Design, Chapter 3 outlines Implementation and Chapter 4 covers Verification.

The cycle displayed on this diagram represents a cyclical, iterative process which should be worked through a number of times as mining progresses to ensure that the cablebolting process is well-tuned.

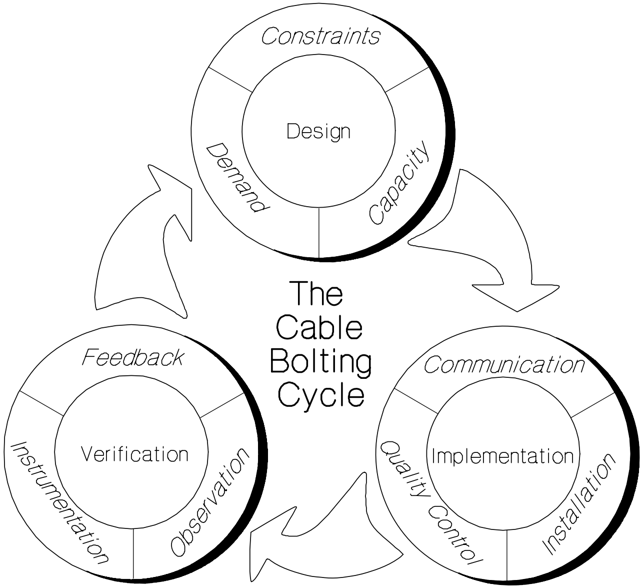


Figure 1.4.1: The cablebolting cycle

1.4.1 Design

The design of any system is based on efficiently matching the available capacity with the required demand while remaining bounded by certain specified constraints. In the case of excavation support design the demand arises from the disturbed rock mass, after the equilibrium of stress and of gravity is disturbed by the creation of an opening. The rock must move to adjust to a new equilibrium. If this is not possible, support must aid in achieving this equilibrium. This requires support properties such as stiffness, load capacity and load displacement capacity.

Cablebolt *capacity* is discussed at length in Chapter 2. Key aspects to consider:

- Loading configuration and testing (Sections 2.2 and 2 8)
- Cable array (Section 2.3)
- Strand capacity (Section 2.4)
- Grout strength / stiffness (Section 2.5)
- Cable/grout and grout/rock bond capacity (Section 2.6)
- Plating and surface fixtures (Section 2.7)

Demand assessment is covered in Chapter 2 using

- Empirical approaches based on experience (Sections 2.14 to 2.17)
- Mechanistic approaches based on behavioral analysis (Section 2.18)

Constraints are placed on the support system design by the economics of mining including cablebolting costs and potential losses due to instability. Other constraints include regulated safety standards, mining and development sequences and access for equipment. The determination of a cablebolting configuration including equipment selection is covered in Sections 2.9 to 2.12.

1.4.2 Implementation

Implementation of cablebolts, as discussed in Chapter 3, involves effective *communication* and *quality control* in addition to *installation*. Communication is critical to the successful implementation of a support system. Recommendations for crew instruction, training and communication are discussed in Sections 3.2 to 3.5. Installation procedures are detailed in Sections 3.6 to 3.10. Quality control guidelines and monitoring procedures are presented in Section 3.11 and 3.12.

1.4.3 Verification

Chapter 4 briefly discusses verification and performance assessment. In order to justify the expense of a support system and to optimize the efficiency of a particular design, a verification program must be implemented, consisting of visual *observation* (Section 4.2) where possible, *instrumentation* (Section 4.3) and measurement of rockmass and support performance and must include *feedback* (Section 4.4 to 4.6) into the design process.

1.5 The Cablebolt Toolbox

The cablebolt toolbox includes a wide variety of items that allow the user to design a truly effective cablebolt element for most potential rockmass failure conditions. The toolbox includes a number of different cablebolt strand configurations and different grouting materials. Additional items in the cablebolt toolbox include surface restraint elements such as plates and straps.

The basic cablebolt that has been used around the world for a number of years is the plain strand cablebolt. In the last 20 years, a number of different types of modified cablebolt strand have been developed in response to problems encountered with poor performance of plain strand cablebolts at mine sites. Some of these modified geometry cablebolt strands are shown in Table 1.5.1. Further discussion of the characteristics of the modified strands is made in Section 2.9.

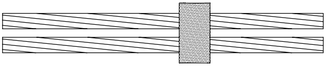
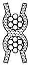
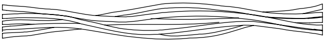
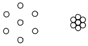
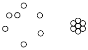
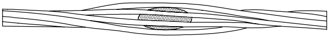
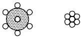
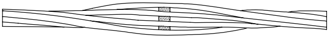
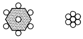
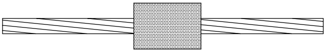
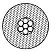
Cablebolts have traditionally been grouted with a grout composed of cement and water. The grout water:cement ratio observed at mine sites by the authors ranges from 0.3 to 0 6 or 0 7. Laboratory cablebolt pull tests on plain strand conducted at the US Bureau of Mines by Goris (1990) and at Queen's University by Reichert, Bawden and Hyett (1992) have shown that the water:cement ratio (*W:C*) of the grout should be kept in the range of 0.3 to 0.4 for optimum cablebolt capacity and performance. The lower *W:C* grout will give the best capacity in pull out tests. However increasing scatter of laboratory *UCS* test results at *W:C* = 0.30, problems with mixing, pumping and grouting very thick grouts with some equipment, and concerns about the ability of very thick grouts to flow into the cages of modified strands, create conditions in which a thinner but adequately strong grout (*W:C* = 0.35 - 0.4) is specified in design.

A detailed discussion of the influence of grout strength on cablebolt capacity is provided in Sections 2.5 and 2.6. A schematic summary of grout mix design is given in Figure 2.5.9.

Other materials and concrete admixtures have been used to grout cablebolts at some mines. Epoxy based grouts have been used in highly corrosive environments to protect the steel strand. Sanded grouts and shotcrete grouts have also been investigated in laboratory tests.

Plates and straps are used to "tie" the cablebolt element to the exposed rockmass surface. This is important in areas where the surface rockmass is not sufficiently retained by the linear cablebolt element, and where there is access to the "working end" of the cablebolts. In this case, the cablebolts must be correctly tensioned during installation of the surface retaining elements which must be securely attached to the end of the cablebolt with a wedge and barrel device (Section 2.7).

Table 1.5.1: The cablebolt toolbox (after Windsor, 1992). Detail in Section 2.9.

	Longitudinal Section	Cross Section
Single plain strand		
Double plain strand with spacers		
Birdcaged strand		
Bulbed strand		
Ferruled strand		
Nutcaged strand		
Epoxy-coated or encapsulated strand		
Buttoned or swaged strand		

1.6 Cablebolt Function

Cablebolt support performs a combination of *reinforcement* and *holding* functions. As reinforcement, the cables prevent separation and slip along planes of weakness in the rockmass. If moderately rough joint and fracture surfaces can be kept from separating, the influence of these discontinuities can be minimized. An effectively continuous rockmass is almost always stronger than a discontinuous one and therefore cablebolts help to mobilize the inherent strength of the jointed or fractured rockmass. Cablebolts cannot, however, increase the overall rupture strength of a continuous rockmass and are unlikely to prevent hard rock from fracturing under high stress. If the inherent strength of the rockmass is not enough to resist the effect of induced stresses or if discontinuities are unfavourably oriented resulting in free and removable blocks, cablebolts can be effective *holding* elements, keeping the failed rock or free rock blocks in place.

Cablebolts are inefficient *retention* (maintenance of small loose surface particles) elements in poor quality rockmasses unless used in combination with screen, shotcrete, straps or other surface coverage. The same is true for fractured rock at depth. If the fracturing is intense, the cablebolts may not be able to retain the fractured rock in place. If the rockmass at the excavation surface is held together by other surface retention systems, cables attached to the retainer elements can provide effective holding capacity.

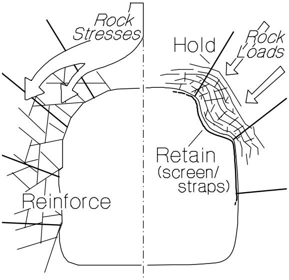


Figure 1.6.1: Primary support functions (after Kaiser et al., 1995)

Hard Rock vs Soft Rock

This handbook was developed to suit the needs of the hard rock mining industry. According to Carter (1995), hard rock includes most igneous rocks, non-schistose metamorphic rocks and well cemented sedimentary rocks. Soft rock includes highly weathered or altered igneous rocks, weakly cemented rocks such as schists, phyllites, shales, silts and fault zones. Although coal is often considered to be soft rock, most of the guidelines and recommendations in this book apply to many coal mining environments. Some so-called hard rock mining environments can contain zones of very poor quality (talc schists, altered dykes). In general, the guidelines and recommendations contained in this handbook can be applied, with suitable caution, to most rock types encountered in underground mining. An exception may be viscous materials such as salt and potash. Support logic described in this book can only be applied to the short term behaviour of these materials. Excavations with excessive water inflow in poor quality rock and with extremely poor, squeezing soil-rock are also not covered here.

Cablebolt Function

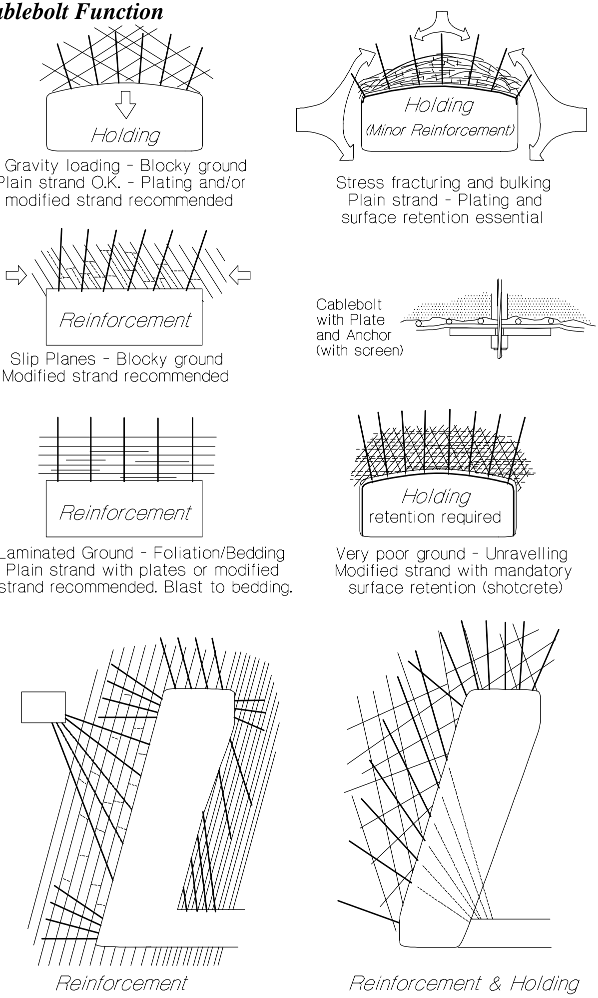


Figure 1.6.2: Typical cablebolt functions (italics indicates cablebolt function only)

1.7 Equipment

There are a number of different cablebolt installation operations that require specialized materials and equipment. This equipment might have been purchased for other applications at the mine site, but generally is uniquely useful for cablebolting applications. Wherever possible, it is advisable to dedicate specific equipment units for cablebolting and to make the maintenance and care of these units the responsibility of the cablebolt crew. Regular maintenance and post-shift clean-up can be made an integral part of the cablebolting payment incentive system to ensure minimum unscheduled downtime.

The equipment is discussed in a number of places in this handbook, so will be summarized in a list here for reference. Some of the items in the list will be required at all sites, and some are designed for a specialized task that might be carried out at all sites. This is not an exhaustive list, and there are likely to be alternative types of equipment that are better for certain operations.

In addition to the cablebolt materials including strand (Sections 2.4 and 2.9), grout (Section 2.5), plates and anchors (Section 2.7), tubing (Section 2.10 and 2.11.3) and attachments (Section 2.11.4), the cablebolting equipment list could include:

- Stationary cablebolt reel or revolving dispenser for dispensing the cablebolt (Section 2.11.4).

- Hydraulic cutter, air powered grinder, oxy-acetylene torch, or explosives for cutting the cablebolt (Section 2.11.4).

- Custom built cablebolt pushers (Section 2.11.4).

- Paddle, drum or colloidal grout mixers (Section 2.11.2).

- Piston or progressing cavity grout pumps (Section 2.11.2).

- Tension jack for tensioning the cablebolt during surface element (plate, strap) installation (Section 2 7).

- Cablebolting truck equipped with all of the items listed above, and any tools required by the crew (Section 2.11.4).

In the experience of the authors, quality control and productivity are both greatly enhanced by a well equipped, self-contained, mobile and organized cablebolting crew. In general, where cablebolting is to be a major ground control priority, time and money spent creating an efficient, well trained cablebolting unit will pay dividends.

1.8 Cablebolt Installation Options

There are a number of different methods in use for grouting cablebolts. The selection of the best method depends upon the orientation of the borehole, the type of cablebolt, the grout flow characteristics and the grouting equipment available.

A brief description of the different installation methods is given in the following pages. Installation methods are discussed in more detail in Section 2.10 and in Chapter 3. The cablebolt installation methods most commonly used are:

- Breather Tube Method. This method is used for upholes only and with grout of 0.375 - 0.45 water:cement ratio. The optimum grout for this installation method is 0.4 *W:C*. (The consideration of other important items such as cablebolt types, breather tube diameter etc. may alter the range of optimum grout water:cement ratio specified in design). This method should be used with caution in areas with open fractures in the back which may cause grout loss with thinner grouts and may prevent complete filling of the hole. In loose, thinly laminated ground, caution is required to avoid over pressurizing fractures causing the laminations to separate and rupture.
- Grout Tube Method. This method can be used for any hole orientation and with grout of 0.30 to 0.375 water:cement ratio. The optimum grout for this installation method is 0 35. These thicker grouts may cause pumping difficulties with less powerful pumps and long holes. The use of modified geometry cablebolt elements may require slightly wetter grout at *W:C* of 0.37.

In both of these methods, the tube(s) are attached to the cablebolt strand prior to the placement of the cablebolt in the borehole. The grout front flows along the entire length of the borehole in these methods.

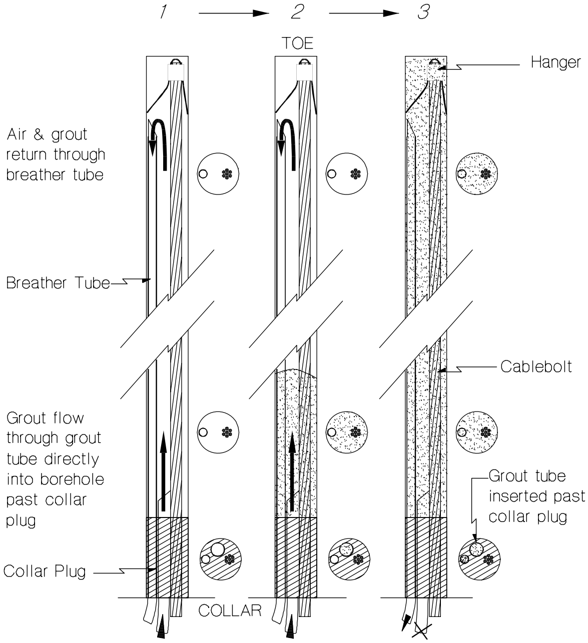
Variations on the grout tube method in use at some mines are:

- Retracted Grout Tube Method. The grout tube and cablebolt are placed in the borehole and then the reusable grout tube is withdrawn from the borehole as the grout is being pumped. The grout is placed in position in the borehole and does not flow over appreciable distances. Care and experience is required to prevent void formation in the grout column. A highly skilled operator is required.
- Grout and Insert Method. In this method the borehole is grouted using the Retracted grout tube method and then the cablebolt is pushed into the grout filled borehole. The diameter of the borehole can be reduced in this method, and the grout tube is reusable. This method is generally used with automated cablebolting equipment. Overly rapid tube withdrawal or cable insertion will result in a poorly coupled system.

Each of these installation methods is described in the following pages. The method most likely to completely grout the hole should be selected.

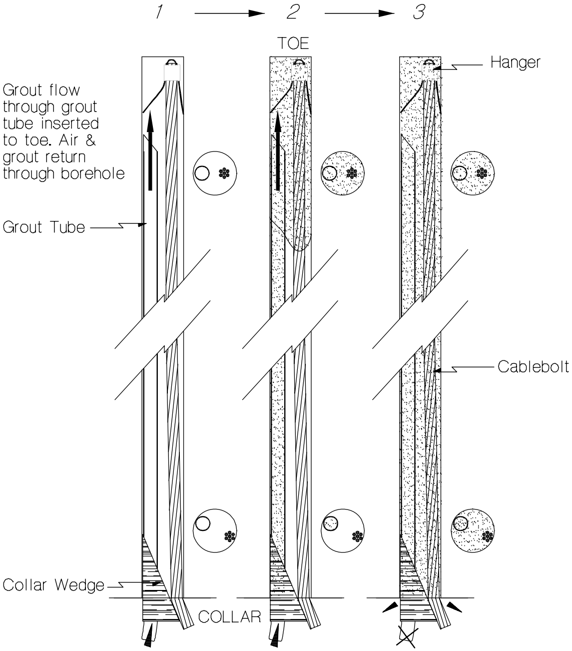
1.8.1 The Breather Tube Installation Method

- In this method, the breather tube extends to the toe of the hole, while only a short length of grout tube is used at the collar of the hole. A cablebolt hanger and borehole collar plug are required.
- Grout of 0.4 water:cement ratio is optimum for this method.
- The grout is pumped through the short grout tube into the borehole. The grout flows upward against gravity in the hole. Air and then grout are expelled from the hole through the breather tube. Return of good quality grout through the breather tube is essential to indicate that the borehole is full of grout.
- A piston pump or progressing cavity pump can be used.
- Problems encountered with this method include: leaking or blown out collar plugs, caused by poorly plugged collars or undersized breather tubes, grout much wetter than design consistency; and no grout flow from the breather tube due to loss of grout into a badly fractured rockmass, an undersized breather tube for the design grout consistency, or inadequate pumping time.



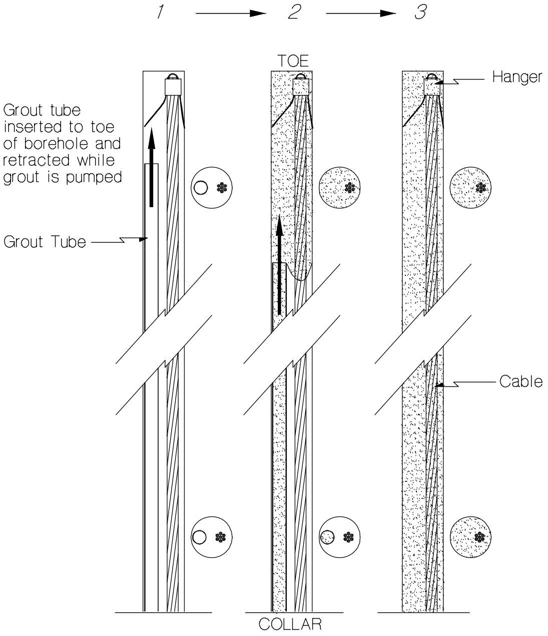
1.8.2 The Grout Tube Installation Method

- The grout tube extends to the toe of the hole. A cablebolt hanger at the toe and/or a wooden wedge inserted at the collar secure the bolt in upholes.
- Grout of ≤ 0.37 water:cement ratio should be used for upholes.
- In upholes, the grout is pumped to the toe of the hole through the grout tube. The grout then flows downward with gravity inside the borehole. The grout must be thick enough so that at the instant the pump is stopped, the position of the grout flow front will freeze in the hole. A thick consistency "donut" of grout appearing at the collar indicates complete grouting of the hole. Obstructions, such as the wires of a modified cablebolt strand or spacers, may divide the grout front, leaving voids in the grout column.
- A continuous stream of grout is required, so a progressing cavity pump is usually used.
- Voids can easily be created in upholes: too thin grout will slump or spiral down the hole, and thick grout may hang up in the hole preventing complete grouting.



1.8.3 The Retracted Grout Tube Installation Method

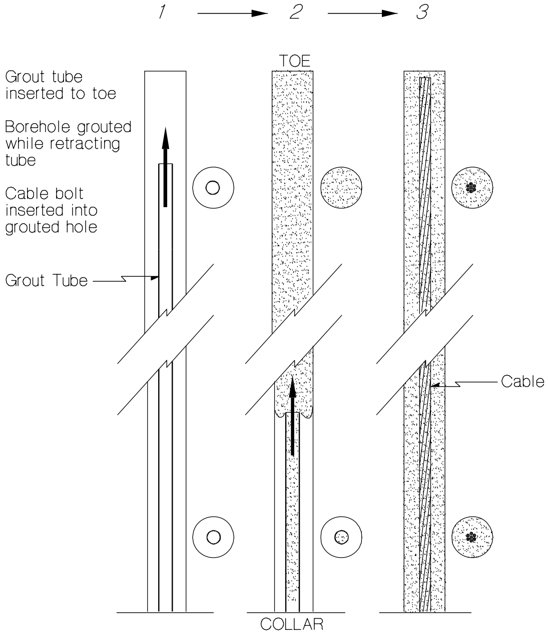
- The grout tube extends to the toe of the hole, but is retracted and can be reused. A cablebolt hanger is required to secure the cablebolt in upholes.
- Grout of ≤ 0.37 water:cement ratio should be used for upholes.
- The grout is pumped to the end of the grout tube, which is withdrawn slowly from the borehole. In this method, the grout is placed at the required position and flows only a short distance within the borehole. If the grout tube is withdrawn too quickly, voids will be created in the grout column. The grout must be thick enough so that it will hang up in an uphole. This method is the most reliant of the four on good crew skills and training.
- The pump must have enough power to pump thick grout into the longest hole.
- Voids are easily created: too thin grout will slump down upholes, and too thick grout may freeze in the grout tube. If the grout tube is withdrawn from the hole too quickly, voids will also be left in the grout column.



1.8.4 The Grout and Insert Installation Method

This method is generally used for cablebolting machines only, since a lot of force is required to push a cablebolt through the column of grout.

- In this method the reusable grout tube is pushed to the end of the hole, then is retracted during grouting. The cablebolt is inserted into the grout filled hole.
- Grout of 0.37 to 0.35 water:cement ratio should be used for upholes.
- The grout is pumped to the end of the grout tube, which is withdrawn slowly from the borehole. In this method, the grout is tremmied into place so that it flows only a short distance within the borehole. The grout must be thick enough so that it will not slump down in upholes (*W:C* ≤ 0.37), but not so thick that it will not fully encapsulate the cablebolt strand.
- The pump must have enough power to pump thick grout into the longest hole. Either a piston or progressing cavity pump can be used.



1.9 The Cost of Cablebolting

The cost of cablebolting varies and can often seem high, relative to other types of support. However, if the cablebolts have been well designed and installed, they should reduce mining costs appreciably by reducing expensive dilution costs, more than paying for themselves. In addition, cablebolts that are performing well will increase the safety of the people working in the mine, and will increase the stability of the immediate and of the surrounding mining excavations. Some costs for cablebolting are included here for general information.

Table 1.9.1: Unit cost for cablebolting including drilling (after Goris et al., 1994)

Mine	Cablebolt applications at different mine sites	Cost (1992 $ Canadian) $ / m of cablebolt
A	Single cablebolts with 0.3 by 0.3 m plates	$29.46
B	Double cablebolts	28.84
C	Double cablebolts	28.54
D	Single cablebolts	31.83
E	Single cablebolts	19.69

Table 1.9.2: Typical cost for a 12.2 m long twin strand cablebolt (after Goris et al., 1994)

Item	Cost (1992 $ Canadian) $ / cablebolt
Hole drilling, including labour	145 67
Twin cablebolt strand	44.04
Cablebolt hanger	4.25
0.3 by 0 3 m steel plate	2.45
Wedge and barrel	3.30
Grout tube to toe of hole	12.46
Cement	10.00
Labour	52.09
Total cost	274 26
Average cost per metre	22.48

Table 1.9.3: Production rates for cable insertion and for grouting (after Goris et al., 1994)

Mine	Crew Size	Cable Length m	Grout Water:Cement *W:C*	Productivity rate meter of cablebolt / shift	
				Cable Insertion	Grouting
F[1]	2	9 - 15	0 32:1	90	180
G[1]	2	15	0 45:1	69	166
H[2]	3	15 - 20	0 32:1	166	230

NOTE: *1 - Mines F & G; 8 hour shift. 2 - Mine H; 12 hour shift* (Drilling not included)

1.10 A Note About Units

This handbook uses S.I. units exclusively. This is to avoid confusion and to simplify computation. The following conversions are provided for convenience:

Distance (Length, Width, etc.)

1 foot (ft) = 0.3048 metres (m)
1 inch (in) = 0.0254 metres (m) = 25.4 millimetres (mm)
1 metre (m) = 1000 mm = 3.2808 feet (ft) = 39.3701 inches (in)

Area

1 square foot (ft^2 or sq. ft.) = 0.0929 square metres (m^2 or sq. m.)
1 square inch (in^2 or sq. in.) = 0.000645 (m^2) = 645.16 (mm^2)
1 square metre (m^2 or sq. m.) = 10.7639 (ft^2)

Volume

1 cubic foot (ft^3 or cu. ft.) = 0.0283 cubic metres (m^3 or cu. m.) = 28.316 litres (l)
1 cubic inch (in^3 or cu. in.) = 1.6387x10^{-5} m^3 = 0.0164 l
1 litre (l) = 0.001 m^3 = 0.0353 ft^3 = 0.02642 U.S. Gallons
= 0.21998 U.K. Gallons

Mass

1 kilogram (kg) = 2 2046 pounds (lbs) (mass)
1 tonne (t) = (1000 kg) = 2204.622 pounds (lbs) (mass) = 1.1023 tons (short)
1 short ton = 2000 lbs (mass) = 0 9072 tonnes (t)

Force

1 Newton (N) = 0.2248 pounds (force)
1 kN = 1000 kg.m.s^{-2} = 224 91 lbs (mass) x 1 g (gravitational acceleration)

Pressure

1 MegaPascal (MPa) = 145.05 pounds (force) per square inch (PSI)
1 PSI = 6.895 kN/m^2 = 6.895 kPa

Note: Force - Mass equivalency:

Under gravitational acceleration (1g = 9.81 m/s^2), 1000 kg of mass (1 tonne) produces 9.81 kN of force. In this manual, the relationship, 10 kN = 1 tonne, is used for simplicity and familiarity. The 2% error is not significant, but it is important to remember that tonnes represent mass, while kiloNewtons represent force (mass *x* acceleration), and that 10 kiloNewtons is the approximate force generated by 1 tonne of mass at 1 g of acceleration (due to gravity). Also remember that a metric tonne is approximately 6% larger than an Imperial ton.

1.11 Useful Definitions

Density, Unit Weight and Specific Gravity

In engineering applications, these terms are often incorrectly used interchangeably to describe the relative heaviness of materials. It is important to understand the differences in the meaning of these terms in order to avoid critical errors in design calculations.

Density, ρ

Density describes the amount of material or mass that is contained within a specific volume. One litre of water, for example, contains one kilogram of the liquid. One cubic metre contains one thousand kilograms or one metric tonne of liquid water. Therefore we say that the density of water is one tonne/m^3 or one kilogram/litre. A felsic granite, for example has a density of 2.7 tonnes/m^3, while a high grade sulphide can have a density greater than 3.3 tonnes/m^3.

Unit weight, γ

Unit weight describes the weight or force exerted by gravity on a unit volume of material. It is obtained by multiplying the density by gravitational acceleration or 9.81 m/s^2. (A more convenient conversion factor of 10m/s^2 can be used for most practical applications). The resultant value is expressed most conveniently in units of kiloNewtons/m^3 (kN/m^3) or MegaNewtons/m^3 (MN/m^3). The unit weight of water, therefore, is given by:

$$1000\ kg/m^3 \ x\ 9.81\ m/s^2 = 9810\ N/m^3 = 0.0098\ MN/m^3$$

Granite has an approximate unit weight of 0.027 MN/m^3, and a high grade sulphide an approximate unit weight of 0.033 MN/m^3.

Specific Gravity, S.G.

The specific gravity of a material is simply the dimensionless ratio of either the unit weight or the density of a material to the respective unit weight or density of water. The specific gravity of water is, of course, unity or 1. The specific gravity of granite becomes approximately 2.7, while the sulphide has a specific gravity of approximately 3.3. Note the absence of units. This is therefore a convenient term to state the relative heaviness of materials, since it is independent of the system of measurement and the units used.

When performing calculations in this book for hard rock underground applications, it is convenient to use a specific gravity of 3.0 or a unit weight of 0.03 MN/m^3, if the true value is not known. This value is an average between barren waste rock and higher grade metallic ore.

2 DESIGN
Application of Engineering Principles

2.1 Introduction

This chapter summarizes most of the key considerations involved in the design of cablebolt systems in underground mining environments.

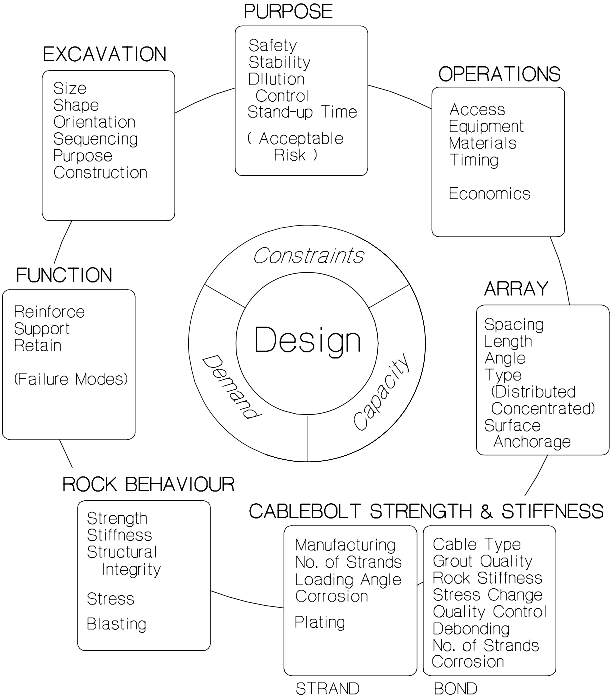


Figure 2.1.1: Key considerations for cablebolt design in underground mines

2.1.1 Design Acceptability Criteria

Safety

In areas with high occupational or flow-through traffic such as refuge and shaft stations, garages, crusher and equipment rooms and haulageways, safety is likely to be of paramount importance. While small pieces of loose will go unnoticed in an open stope, they pose a serious hazard where personnel are present. **In using this handbook, supplement designs as needed to ensure adequate safety.**

Stability

Under the influence of stress, gravity and vibration, rock can strain, yield, deteriorate and ultimately disintegrate. Instability and failure can be defined as any limiting point in this progression. Permanent critical openings such as crusher stations and shafts may require a no-damage (yield) criterion while in temporary drifts, time dependent collapse may be acceptable. Stability concerns can be local (serviceability, access, rehabilitation costs) or can be global (destabilization of mining block, pillar collapse, shaft pillar integrity, subsidence, etc.). The consequences of instability should be evaluated as a part of mining engineering.

Dilution

In open stopes, it is not economically practical to attempt to prevent all forms of instability. Limited dilution (waste rock overbreak or minor sloughing) is often accepted within economic limits. The costs and revenue loss due to such ***predictable*** dilution must be weighed against the costs of support (materials, delays, labour) in order to determine the course of action - *support or no support*. In non-entry open stope design, the decision whether (or not) to support at all can have the most financial impact in the support design process.

Stand-up Time

Rockmasses are subject to time dependent deterioration in the vicinity of excavations. An opening may be initially stable but may degrade over time, eventually becoming unserviceable. The required stand-up times (supported and unsupported) of an excavation should be established and considered in design.

Factor of Safety

Design calculations (bolt spacing, length, critical span, stand-up time) are often based on numerous assumptions. In addition, the uncertainty inherent in the measurement or collection of engineering data as well as the variability of the underground environment mandate the use of a safety factor (multiplier, additive, percentage,etc.) as appropriate to ensure that safe margins are built into the design.

2.2 Capacity and Demand

2.2.1 Introduction

In this handbook, the term *capacity* is used in a very general fashion. It is used to encompass all aspects of cablebolt performance. In this chapter, the performance specifications and expectations for many of the more common and available cablebolt configurations are presented. Particular attention will be paid to the plain strand (seven-wire) cable, since all other modified geometry strands were developed to overcome deficiencies in the plain strand cablebolt.

Demand considerations address the necessary enhancements required to stabilize an excavated rockmass. These arise as a result of excavation size and geometry, the strength and structural integrity of the rockmass, the induced stresses around the excavation and the aggressive and/or changing nature of the excavation environment. The issues of demand are addressed through rockmass classification, empirical and mechanistic design, local experience and through rockmass monitoring. The demand requirements must then be matched to the capacity of the selected support system.

Support systems based on the seven-wire plain or modified strand cablebolt are primarily frictional, fully coupled devices. That is, load is transferred to/from the rockmass along their entire length unless debonded sections occur as a result of design or installation error. This transfer occurs as a direct result of friction between the cable strand and the encapsulating grout. Load must also be transferred between the grout and the surrounding rock. Modified strand has been developed to increase the degree of cable-grout interlock, thereby increasing the efficiency of the load transfer. If this so-called bond is optimized, then the performance of the cablebolt system will be controlled by the quantity of steel strand (cablebolt distribution), the geometry of the cablebolt array and environmental changes after installation. The overall performance of the cablebolt system can be subdivided into five capacity categories which can be directly related to the categories of demand which they address. These are listed on the following pages.

In most cases the performance data presented is the result of a synthesis of available testing results from the literature and from unpublished contributions. Every attempt has been made to simplify data into a practically useable form for design and system selection. While the information contained here should be sufficiently self-contained for preliminary design, references are cited for those who wish to delve deeper into the body of literature regarding component properties, behaviour, performance and testing. Cablebolt research continues to be a healthy international industry and it is profitable to keep up to date with new developments through industry and research publications and through suppliers.

Capacity Considerations

The capacity of a cablebolt element is based on the properties of the strands, on the bond and frictional resistance of the interface between the cable and the grout, on the quality of the grout and on the load transfer between the cable and the surrounding rock. The capacity of the cablebolt support system is also determined by the cablebolt pattern density, orientation and length. The system capacity can be expressed as the sum of the following considerations:

- ***Immediate stiffness*** describes the relationship between initial loading increments and their associated displacements (cable stretch + interface slip) within the cablebolt system. Of significance is the stiffness over the first 1 - 10 mm of displacement.
- ***Ultimate ductility*** describes the maximum displacement that can be accommodated by the cablebolt system before total bond failure or cable strand rupture. High ductility or displacement capacity is desirable in highly stressed or dynamically active ground.
- ***Ultimate load capacity*** describes the maximum static load which can be sustained by the cablebolt before strand rupture or total bond failure (free slip).
- ***Surface retention*** is required to ensure the local integrity of a rock face and/or to guarantee personal safety. Cablebolts spaced too far apart may permit face disintegration between the cables, requiring additional surface fixtures.
- ***Longevity and sensitivity*** are important considerations when cablebolts are exposed to corrosive environments, blasting and changes in local stress and confinement. In addition, creep or relaxation within the cablebolt system can impair the support effectiveness.

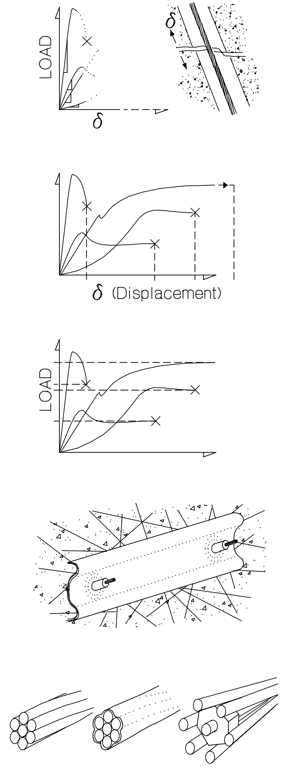


Figure 2.2.1: Capacity considerations

Demand Considerations

Demand assessment is based on a systematic consideration of the key rockmass properties (initial stress, strength, stiffness, structural integrity, etc.) and of the expected disturbing influences (stress change and gravity loading). Potential failure modes can then be identified and where necessary, support can be designed to maintain stability. Demand can be separated into five main components, each corresponding to the respective capacity component on the previous page:

- ***Dilation control***; rockmasses normally contain joints and fractures. These surfaces are primarily frictional (shear strength dependent on normal pressure) and dilational (open during shear due to roughness). If these surfaces can be held together by stiff reinforcement, the interlocking roughness and frictional strength of the rockmass are maintained.

- ***Displacement***; where high stresses exist or where smooth and continuous discontinuities allow for large amounts of slip under stress, displacements in the rockmass may be severe. These stresses are normally shortlived, however, as stresses redistribute, leaving free deadload blocks or broken rock zones.

- ***Gravity loading*** of the support system will dominate design if the rockmass is allowed to disintegrate or if structural features form free deadload blocks.

- ***Surface ravelling*** can occur when cablebolts are spaced too far apart with respect to the block size in the rockmass or if the near-face cable segments possess inadequate bond strength.

- ***Service life and robustness***; corrosive mine water can impair long-term capacity of cablebolts. Stress and confinement reductions in the rockmass can impair rockmass stability and also can reduce cablebolt capacity.

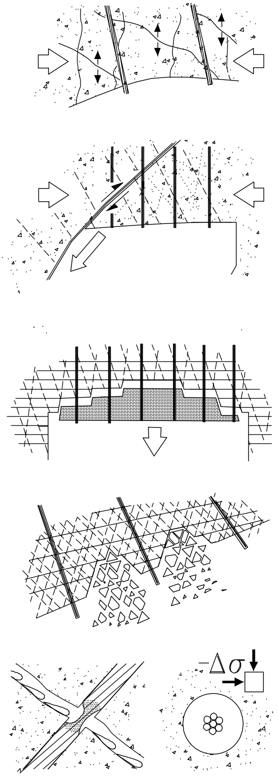


Figure 2.2.2: Demand considerations

2.2.2 Loading Configurations for Cablebolts

The first three categories of capacity and demand can also be subdivided into subcategories of loading type:

- Axial or tensile
- Shear
- Combination axial/shear

These modes of cablebolt loading occur individually or in combination within an array of cablebolts as illustrated below. It is important to estimate the most likely direction of motion (not always down or along a joint) in order to identify the operative loading modes.

In Situ Loading of Cablebolts

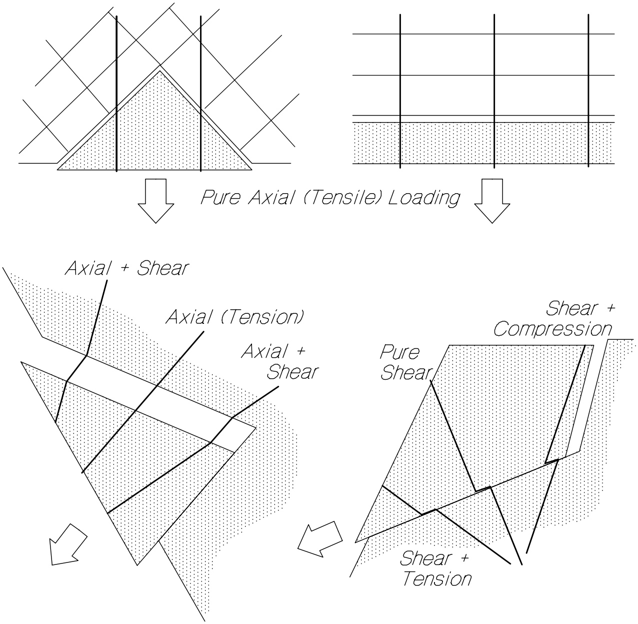


Figure 2.2.3: In situ loading of cablebolts (after Windsor & Thompson, 1993)

Testing Configurations for Cablebolts

Axial loading tests are relatively simple to design and perform in the laboratory and are also possible in the field. For this reason, the vast majority of available data on cablebolt performance deals with axial tension testing.

Unconstrained tests (Maloney et al.,1992) are easily carried out but because they allow the cable to rotate, they tend to give a *lower bound* strength. In these tests a length of cable is grouted into a rigid pipe (see Hyett et al., 1992 for equivalent stiffness relationships for pipe sections) with one long free length, for gripping and pulling, at one end and a short free length at the other. To ensure a constant embedment length, this shorter length should be equal to or greater than the desired pull-out displacement. The long free end is used for gripping in the test machine. Use grips which will not cause premature rupturing of the cable. Displacements should be measured between a marked point on the cable and the exit end of the grouted cylinder as shown in Figure 2.2.4.

Non-rotating (Goris, 1990; Hyett et al., 1992) and double pipe (Windsor and Thompson, 1993; Villaescusa et al., 1992) tests are slightly more complex and tend to give an *upper bound* on pull-out strength. The procedures for these tests are detailed in these references. Note that the cable does not rotate during pullout as it does in the unconstrained test. This forces the cable to shear through the grout flutes and increases pull-out resistance. Displacement should be measured between the opposing exit points in the two pipe sections as shown in Figure 2.2.4.

The difference between the constrained non-rotating single pipe test (Hyett et al , 1992) and the double pipe tests is that in the former, the fixed section of pipe is considerably longer than the test section and/or a swaged or welded anchor is placed on the cable within the grouted test section to prevent slip. In the double pipe test, both sample or test sections are designed to slip equally. As such, the measured displacements in the double pipe tests will be twice those in the fixed pipe constrained test as shown in the example result in Figure 2.2.4. It is important to be aware of this effect when comparing results from different tests.

When performing these tests for the purpose of comparison between different grouts and cable configurations it is important to record the following information and to maintain control over those parameters which are to remain constant:

- Cable Type
- Grout *W:C*, Curing time and if possible *UCS* (samples from same grout batch)
- Embedment length; Test type; Anchor length or free pull length
- Pipe material and dimensions (Borehole diameter and properties for field tests)
- Approximate pull rate
- Pullout load and the displacement during the test.
- Cable response notes (e.g. rotation, stick slip, cable strand rupture)

Testing Configurations for Cablebolts - Axial

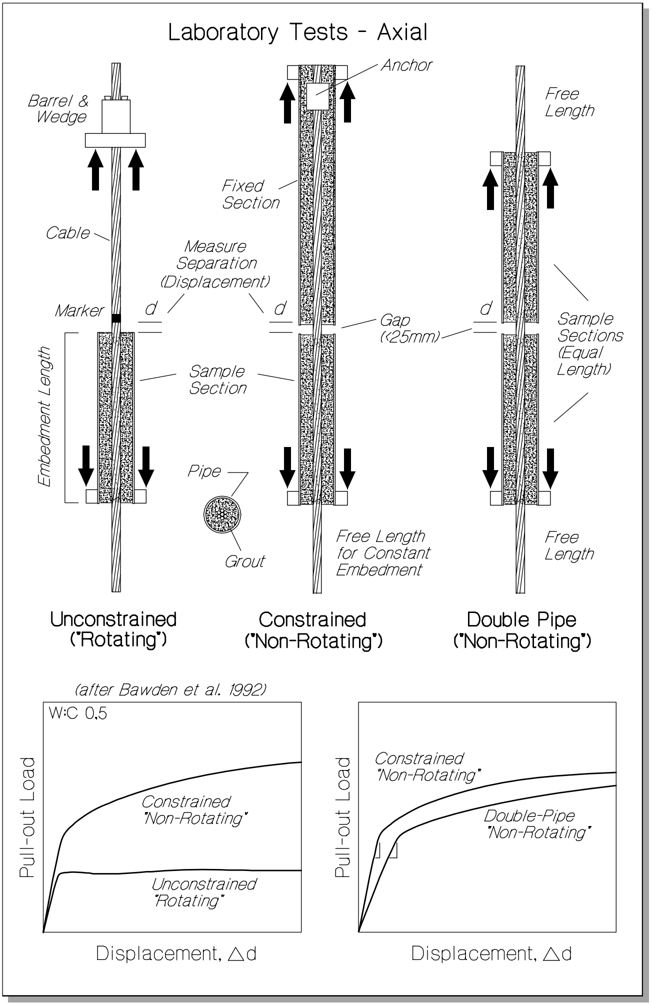


Figure 2.2.4: Three basic configurations for axial pull-out tests (laboratory)

In situ field tests can be carried out as shown in Figure 2 2.5. Unconstrained tests are documented in Maloney et al. (1992). Note that the downhole length is covered by a plastic tube, for debonding, except for the test section (test embedment length). The entire hole can be grouted if desired. Constrained field tests are more complex due to difficulties in constraining the down-hole cable length. A procedure for constrained tests is detailed in Bawden et al. (1992). Note that in either case, displacements will include cable (or loading rod) stretch.

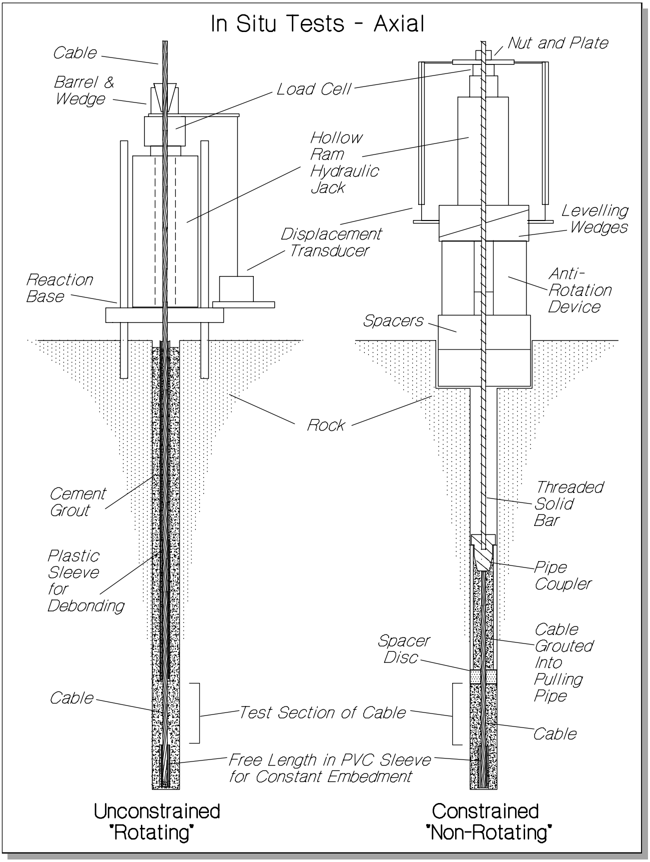


Figure 2.2.5: Axial field tests (after Maloney et al., 1992; Bawden et al., 1992)

Testing Configurations for Cablebolts - Shear

Direct shear (Windsor and Thompson, 1993) and combined axial and shear tests (Hyett et al., 1995) are complex and require specialized laboratory equipment. It is also a complex procedure to properly simulate the shearing and borehole conditions necessary for accurate results. Useful comparisons can be made, however, between different cable systems and between the performance (stiffness and load capacity) of cable strand with respect to loading angle. The actual performance of the cable is dependent on the sense of the displacement (shear, dilation and combined) and on the orientation of the cablebolt with respect to the test interface and the direction of motion.

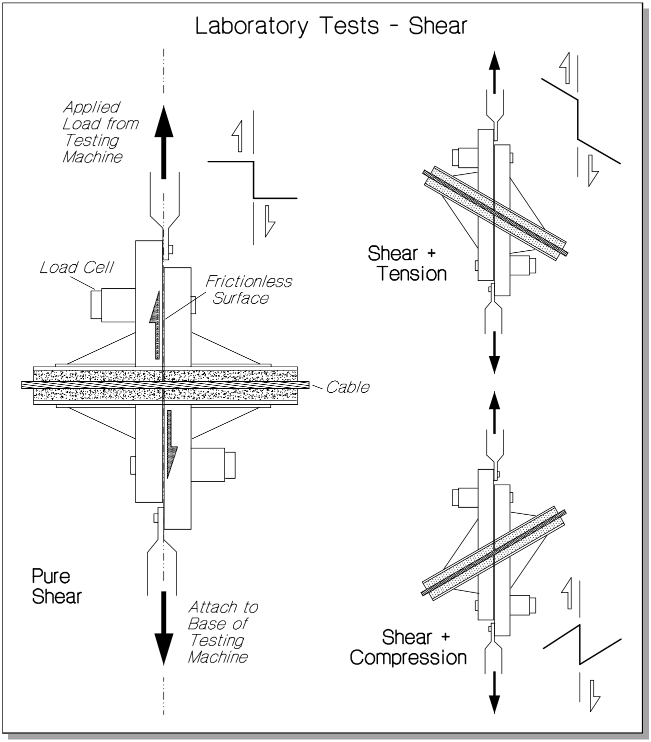


Figure 2.2.6: Direct shear and combination (shear + axial) testing for cablebolts (after Windsor and Thompson, 1993)

Figure 2.2.6 illustrates one configuration for direct-shear testing. In this arrangement the direction of motion is always parallel to the separation plane as would be the case on the basal plane of a sliding gravity block, for example (see Figure 2.2.3). The orientation of the cable, in this setup can be varied from 90 degrees (perpendicular to the surface) to 135 degrees (so that the cable is axially pulled in tension as well as sheared) to 45 degrees (the cable must first kink in compression before shearing). In this test, free dilation (aperture increase) of the sliding plane is prevented.

Figure 2 2.7 shows a fundamentally different type of shear test. In this test, the cable is always perpendicular to the separation plane. This is analogous to a cablebolt installed perpendicular to a laminated hangingwall. The testing frame allows for separation (aperture increase) to occur in addition to shear at numerous angles with respect to the plane (and to the cable). For example an angle of 45 degrees would represent a slab falling straight down from an inclined, laminated wall, inclined at 45 degrees. Bawden et al. (1994) describe some of the many important procedural details required for successful testing of this kind. In particular, it is essential to provide the appropriate confining boundary conditions to the grout column since this is a key parameter controlling shear behaviour.

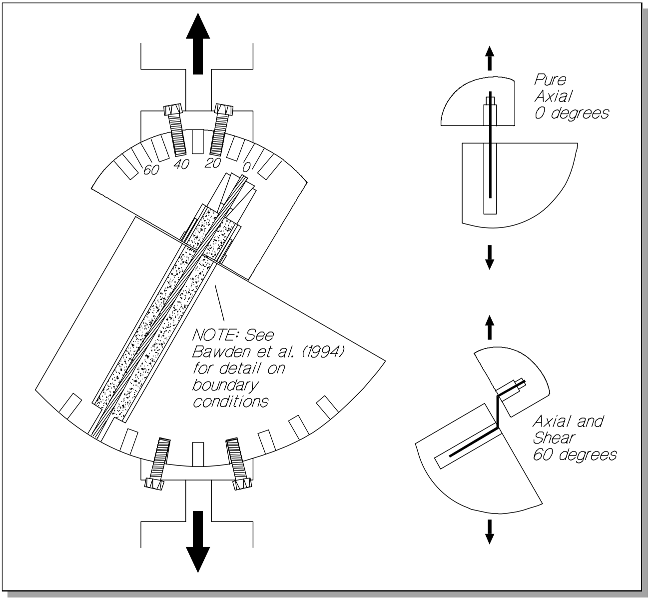


Figure 2.2.7: Combined axial and shear test (after Bawden et al., 1994)

2.3 The Cablebolt System

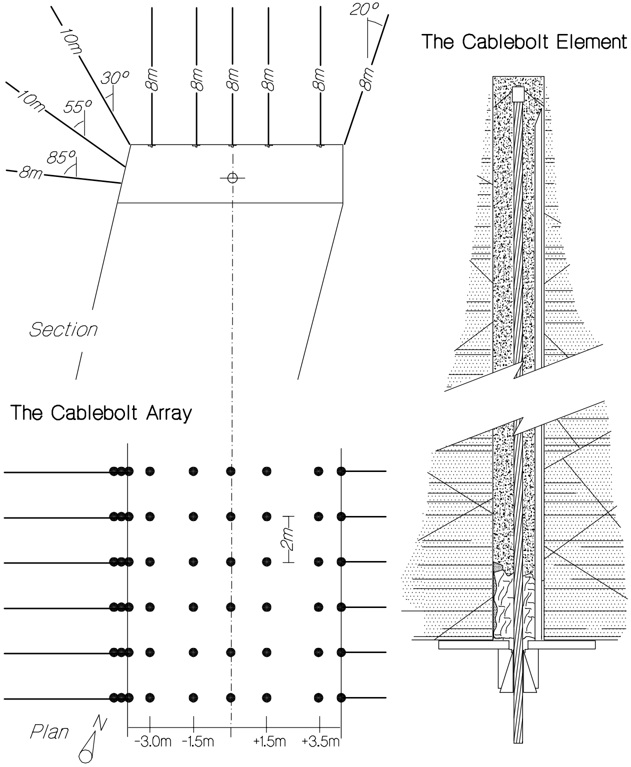


Figure 2.3.1: The cablebolt system (angles and spacings are examples only)

Figure 2.3.1 illustrates the makeup of the cablebolt system which is comprised of the cablebolt array and the cablebolt element itself. The overall performance of the cablebolt system is the result of a complex relationship of these components and of their even more complex interaction with the rockmass.

2.3.1 The Cablebolt Element

The individual cablebolt element is made up of several mandatory and several optional components.

The Cable Tendon

The steel strand, the set of paired or multiple strands or, in recent developments, the fibreglass wire cluster makes up the cable tendon. Most of this book deals with standard and modified (flared) cable configurations based on the 7-wire steel strand.

Grout

The grout forms the link between the cable and the rock mass. Chemical grouts for cablebolting have undergone some experimental use. However, this book will be primarily concerned with cement grouts.

Borehole

Some cablebolt elements are sensitive to the condition and diameter of the borehole and the properties of the rock surrounding the borehole.

Interface Mechanics

The mechanics of the interface between the cable and the grout usually determine the overall behaviour of the system. These mechanics are described in detail for the plain strand cable. The overall system capacity can be limited by the efficiency of the bond strength of the cable-grout interface which can be extremely sensitive to quality control, rockmass stiffness and rock stress change after installation. The various developments in modified cable geometries are primarily aimed at changing the mechanics of load transfer at this interface.

Surface Fixtures and Restraint Elements

Plates, barrel and wedge assemblies, and surface retention elements such as mesh and straps are important aspects of the cablebolt element. Economically practical cable spacings may not always be sufficiently tight to contain smaller surface blocks and wedges. Surface fixtures must perform this role.

Tensioning

The degree (or absence) of tensioning (pre- or post-grouting) can have an influence on the performance of the cablebolt system in fractured ground.

2.3.2 The Cablebolt Array

The behaviour of a cablebolt system is determined by the make up of the individual *cablebolt element* and by the *cablebolt array* as a whole. The components of the array are as follows:

Spacing or Density

A denser cablebolt pattern (a smaller spacing) will have a higher overall stiffness and load capacity, but will be subject to economic constraints. The ultimate ductility will not be substantially affected.

Face Pattern

Dominant joint orientations and block shapes may necessitate a particular geometric pattern in order to intersect as many free blocks as possible. Compatibility with face restraint systems such as straps may also dictate a particular pattern. Normally, for a constant density, small changes in spatial pattern will have a minimal influence on performance.

Length

The length of the cablebolts in an array should be determined by considering the required and actual capacity of the system and the height or thickness of any discrete geometric feature (i.e. wedge or broken zone) being supported. (For example, 20 metre cablebolts in a gravity-based design are unnecessary if the capacity of the system is equivalent to only 4 metres of rock).

Orientation

The orientation of an array is important when trying to optimize support efficiency in cases where the directions of loading and induced displacement (controlled by stress, excavation geometry, gravity and structural discontinuities) are known. Ideally, cablebolts designed for holding (gravity loading) should be aligned along the direction of displacement. Where applicable, to optimize reinforcement efficiency along shear planes (Section 2.8.3), cables should be oriented at 20°- 40° to a sliding surface (positioned to induce tension in the bolt).

Sequence and Timing

Timing of support installation and sequence with respect to mining can have an influence on the system performance by altering the displacements sustained by the system, the pre-cabling deterioration of the rockmass and the stress change and capacity change experienced by the cable system (Section 2.6.2).

2.4 7-wire Steel Strand

Modern cablebolts for hard rock mining applications are primarily based on the 7-wire steel strand originally manufactured for use in prestressing concrete in civil construction. Other configurations include wire rope, fibreglass tendon, and other polymer and composites.

Most of this chapter will be concerned with variations on the 7-wire steel strand cable. Before proceeding, a few definitions regarding the makeup of the steel strand cablebolt are required:

2.4.1 Definitions

Cablebolt element

A single, complete in-hole assembly, including all of the contents of the borehole (cable, grout, etc.), surface fixtures, the borehole specifications and properties, as well as the face restraint system.

Cablebolt

The steel component contained within the borehole (beyond the collar). This includes the steel strand or set of multiple strands in a single borehole and any modifications (bulbing, birdcaging, nutcaging, ferruling, buttons, etc).

Strand

Any length of finished material which comprises a number of wires (i.e. six) spun together in helical form around a centre wire as in seven-wire steel strand (six wires around a centre wire).

Wire

A single continuous length of steel, round in cross-section.

Inner or king wire

The centre wire in a strand which is straight.

Outer or helical wires

Six wires are wound around the centre or king wire and are heat treated (stress relieved) to form a continuous helical spiral.

Lay

The direction of the wire described as left or right hand lay. This has no effect on performance in the field but is a concern in testing when designing anti-rotation devices and when coiling cables for shipment.

Lay length

The axial distance along the strand required for an individual outer wire to return to its original radial position; also called the *pitch length*.

Flutes

The V-shaped helical grooves created along the strand as the six outer wires are wound around the king wire.

Nominal diameter

The diameter assigned in specifications for wire or strand. In the case of the strand this will be measured from the outermost extents of the cable cross-section though the centre of the king wire.

Nominal area of strand

The sum of the cross-sectional area of the individual wires.

Calculated mass of strand

The mass per length calculated from the nominal area and the unit weight of steel (taken as 7850 kg/m^3).

Gauge length

The length over which deformation and strain is measured in a test. Usually the central third or the complete portion of the total **free length** used in the test.

Coil Diameter

Steel cable can be shipped in a continuous length wrapped into a large coil. The inside diameter of this coil must be greater than a specified minimum to avoid disruption of the strand integrity (kinking and/or unravelling).

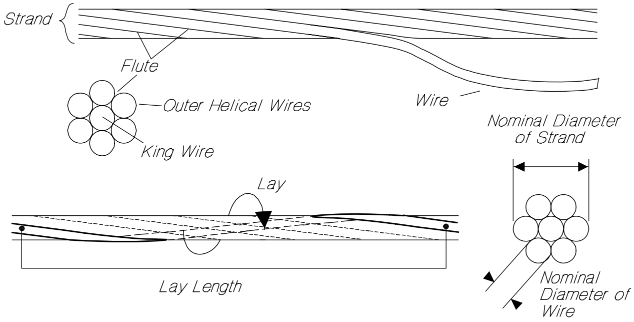


Figure 2.4.1: Geometrical properties of seven-wire strand

2.4.2 Strand Construction

Manufacture

The individual wires are manufactured from round wires, cold drawn from hot-rolled high carbon steel wire rod which has been treated to make it suitable for drawing. The steel should not have more than 0.060 % of sulphur and not more than 0.060 % phosphorus (AS1311 - 1987).

The wires may be plain with mill finish or indented. This indentation is to improve adhesive bond in concrete construction but is of dubious benefit in cablebolting applications.

6 wires are laid helically around a slightly larger (1.02 - 1.03 times larger) centre wire with a lay-length of 12 to 16 times the diameter of the final strand. After stranding, the strand is stress relieved by continuous heat treatment. This ensures that the helical wires maintain their form and reduces the time dependent relaxation or creep of the strand under load (Collins and Mitchell, 1991). The strand is then formed into coils (of greater than a specified minimum diameter to ensure cablebolt integrity) for transport and sale.

The strand may be subsequently *drawn* and *compacted*, resulting in a reduced cross-section and increased effective density of steel within the strand cross-section. This process also results in a smoother surface profile as the outer strands become flattened. While compacted or drawn cable has a higher tensile strength, it has been used in mining with little measurable benefit and is not advised for cablebolting due to the reduced mechanical interlock with the grout.

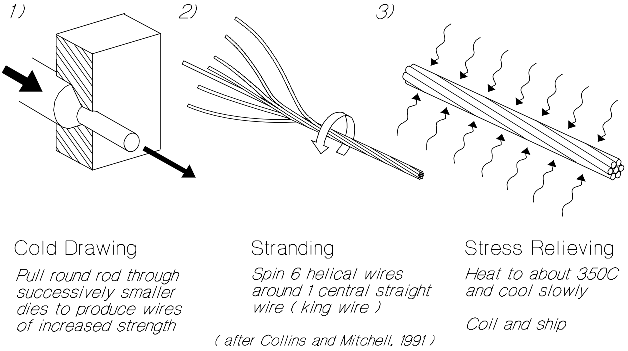


Figure 2.4.2: Manufacture of seven-wire, stress-relieved reinforcing strand

Key Quality Indicators

Some important properties of the strand which can be tested by inspection at the mine site in order of importance are:

- The strand should not carry on its surface or between the wires any lubricant, oil, rust or matter likely to impair its adhesion and friction with the grout.
- The wires should show no tendency to unravel when the strand is cut.
- The finished strand should be of uniform diameter (± 0.4 mm), of standard lay length or pitch and should be free from kinks (plain strand). The centre wire must be held tightly in place and show no signs of looseness.
- Drawn cables with flattened outer wires should not be used.
- The wires must be continuous with no more than one individual weld per 50 metre length of completed strand. The individual wires should be free of defects such as splits and surface flaws.

If any of these requirements are not met, it indicates a deviation from the accepted construction standards, and the strand should be returned to the manufacturer. Such deviations from the standards have been shown to result in serious degradations in pullout performance (Bawden et al., 1995) and steel strength (Thompson, A.G., 1993, pers. comm).

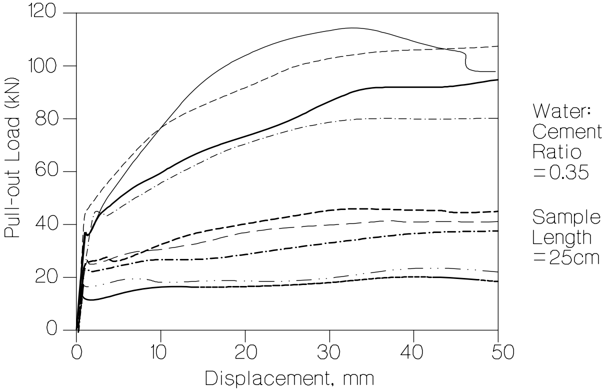


Figure 2.4.3: Variation in pullout performance under identical conditions for strands from six different manufacturers (after Bawden et al., 1995)

Strand Construction Standards

The construction specifications for the 7-wire strand used in cablebolting are covered by the following standards (or equivalent) relating to pre-stressing strand for concrete construction:

North America	(ASTM)	A416-80
Australia	(Australian Standards)	AS1311 - 1987

Even though these standards refer to application in reinforced concrete, they detail the necessary construction specifications to ensure optimum performance of the strand in mining applications. While several grades of steel are covered by these standards, the most commonly used product for cablebolt applications are:

Table 2.4.1: Standards for seven-wire strand (used for cablebolting)

ASTM A416-80	*AS1311 - 1987*
250	Regular Strand (not recommended)
* **270**	* **Super Grade** (normal or low relaxation)
	Extra-**H**igh **T**ensile **Grade** (normal or low relaxation)

* *Most commonly used for cablebolting applications*

Some basic geometrical specifications are as follows:

Table 2.4.2: Geometrical specifications for ***270 - Super / EHT Grade*** strand

Specifications (approximate - check standards listed above)	*Nominal Strand Diameter (from crest to crest of opposing wires)*		
	12.7mm (0.5 ")	**15.2mm * (0.6")**	18mm (0.7")
Nominal Area mm	99 - 100	**140 - 143**	199
Lay-Length mm	155 - 200	**185 - 240**	220 - 285
Calculated Mass kg / metre	0 775 - 0.785	**1.102 - 1.125**	1.560
Minimum Internal Coil Diameter mm	750	**750**	750

*** NOTE: All of the discussion in this handbook involves 15.2mm strand cablebolts. Do not use the design recommendations in this manual for other strand sizes without applying an appropriate adjustment.**

2.4.3 Strand Performance

Definitions

The axial performance of 7-wire steel strand is described by the following specifications (There are no standards for shear performance):

Breaking Load

The tensile load applied to a length of strand at the instant of rupture.

Proportional Limit

The load or strain at which the elastic behaviour of the cable deviates from linear.

Yield Strength and Proof Load

The yield strength is the tensile load applied to a length of strand to achieve a non-proportional (inelastic) strain of 0 2 %. The proof load (also called yield strength in ASTM standards) is similar to yield strength but is specified at a total extensional strain (elastic + inelastic) of 1.0 %.

Elongation:

The total strain ($\Delta L / L$) of a minimum specified gauge length of strand at the instant of rupture of one or more wires in the strand.

Elastic Modulus

The theoretical stress (load/area) applied to a unit cross-sectional area to achieve a unit elastic strain (100 %). The elastic modulus of carbon steel is 205-210 GPa. The modulus of the 7-wire steel strand (based on the nominal area of the strand) is somewhat less than this value due to the behaviour of the helical outer wires.

Elastic Stiffness and Normalized Elastic Stiffness

The elastic stiffness is simply the slope of a load / deformation curve and has units of MN/m. The normalized elastic stiffness is the stiffness of a unit length of material or load / strain. It is quoted in units of MN / (m/m) or MN.

Relaxation

Relaxation is defined as the reduction in load with time of a specified minimum length of strand held at a constant strain or elongation. This strain is achieved at an initial load which is specified as a standard percentage of breaking load. Relaxation is usually specified as percentage relaxation (drop in load as percentage of initial load) after a standard time interval. Relaxation is directly related to creep. Creep is defined as a rate of extension of a sample held at constant load.

Strand Performance Standards

Table 2.4.3: Approximate performance specifications for 7-wire steel strand

Specification (AS 1311 - 1987, ASTM A416-80) See Section 2 4 3 & Figure 2 4.4.	*Gauge * Length: mm or x LL (Lay-length)*	*Regular / 250 Grade*	*Super Grade Steel Strand*			*E H T / 270 Grade*
		Nominal Strand Diameter, mm				
		15.2	*12 7*	***15.2***	*18.0*	***15.2***
Breaking Load: Min. (kN)	*600 or 3 x LL*	230 / 240	184	**250**	338	**261 / 261**
Yield or Proof Load (kN) @ 1% Total Strain (elongation)	*200 or 1 x LL*	196 / 204	156	**212**	287	**222 / 222**
Elongation : Min. (Strain)	*600 or 3 x LL*	**3.5 %** (strain) @ Breaking Load				
Approx. Elastic Modulus (GPa)	*200 or 1 x LL*	**195 - 200** ** Below Proportional Limit				
Normalized Elastic Stiffness (MN)	*200 or 1 x LL*	26 - 27	19 - 20	**27 - 28**	39 -40	**27 - 28**
Relaxation Max. @ 1000hr 80 % Brk Load	*600 or 3 x LL*	LOW Relaxation Strand: **3.5 %**				
		NORMAL Relaxation Strand: **12 %**				

* All tests should have a minimum of 600 mm free length between end grips.

** Stiffness and modulus shown are for rotationally constrained case and with respect to nominal area. When free rotation (untwisting) is permitted, the initial stiffness is reduced by up to 25 %, rising to the constrained stiffness (tangent) with increasing strain. (Costello et al., 1976)

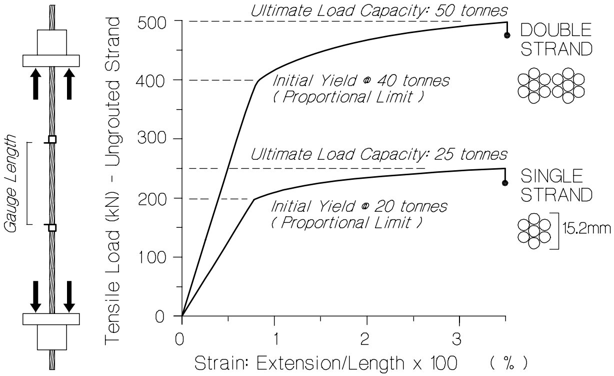


Figure 2.4.4: Minimum strand performance specifications for cablebolting applications

2.4.4 Strand Capacity Considerations

Cablebolt design tools such as rockmass classification, specialized empirical tools such as the Mathews/Potvin Method, and mechanistic analysis provide recommendations for spacing (or density) and length of cablebolts in regular patterns. Spacing recommendations are normally based solely on the tensile load bearing capacity of the strand while length specifications assure the ability of the cablebolt to traverse the reinforced zone and penetrate into stable rock.

NOTE: Most bolting system design tools assume perfect load transfer (very high bond strength), between the cablebolt and the surrounding rockmass, which can be a very erroneous assumption as discussed in Section 2.6. This capacity can be seriously impaired by ineffective bonding (interface shear strength) between the cable and the grout. These issues are discussed later in Section 2.6. In addition the ability of the cablebolt to withstand direct shear (guillotining) is less predictable than the tensile strength. It can be assumed that a 5 - 20 % reduction in capacity is possible in the field due to partial shear loading, although this is heavily dependent on the loading angle (tension + shear).

The stiffnesses listed in Table 2.4.3 and illustrated in Figure 2.4.4 refer to the performance of the steel strand only. The bond mechanics again govern the actual observed stiffness and displacement capacity of a grouted cablebolt in the field. This stiffness is relevant only when known debonded sections are present. In addition, the shear stiffness of cablebolts is significantly less than the response to tensile loading (Windsor, 1992; Bawden et al., 1994) and is extremely dependent on loading angle (Section 2 8).

Under ideal conditions, over a finite area of supported surface, load capacity and stiffness are directly related to the number of cablebolts installed. Doubling the strand density (number of strands per unit area), either by reducing the cable spacing (e.g. from 2mx2m to 1.4mx1.4m) or by using double strand cablebolts (two strands in one hole), normally results in a doubling of both the load capacity and the stiffness of the cablebolt system, although care must be taken when grouting double strand cables to ensure full encapsulation of both strands.

Double strand cablebolts with spacings equal or greater than those for single strand patterns do not improve the ability of the system to retain surface blocks or fragments between the cables. This failure mode must always be considered separately, particularly for cables on wide spacings (>1.5m). Surface restraint (rockbolts, screen) may be a necessary supplement to the cablebolt system.

Plating exposed and accessible cablebolts is always advisable if timing and economics permit. In addition to reducing the sensitivities (of plain strand in particular) to the influences of rock stiffness, stress change and quality control, plates also provide another measure of surface retention and safety.

2.4.5 Corrosion of Steel Strand

Corrosion of high carbon steel strand can be a serious problem in long term civil engineering applications. In mining, however, the incidences of cablebolt corrosion causing serious problems are rare. This is due primarily to the short time frame involved in open stope support in underground mining.

Corrosion problems observed by the authors in mining environments were typically in long term support in open pits where the groundwater was acidic or saline and in long term support in underground sulphide deposits. Cut and fill applications in wet conditions where fractured stope backs could remain (supported) for up to a year were notably susceptible to corrosion. Serious failure, due to corrosion and rupture of the strand, can occur in such applications.

The nature of corrosion is extremely complex and a fundamental discussion is beyond the scope of this book. It is the intent here to discuss some of the important factors involved in corrosion so that the engineer may assess the potential for problematic corrosion and take steps to prevent it or make the appropriate design allowances for it.

Most common refined metals are inherently unstable ionic materials composed of arrays of single atoms which possess a full compliment of electrons. Metals such as iron normally tend to give up electrons at room temperature (gold is a notable exception) and become involved in reactions leading to the formation of more stable compounds such as iron oxide or iron hydroxide (rust). The release of electrons is termed an anodic reaction and the acceptance of electrons a cathodic reaction. Both reactions must occur for corrosion to take place. Since metals such as the iron found in steel cable are normally willing to give up their electrons, it is normally the presence of a cathode which determines the corrosion potential.

The cathodic reaction (involving the consumption of electrons released anodically from the iron) can be made possible by the presence of an acid, sulphate, water and/or oxygen.

Corrosion of steel (iron) can be divided into four basic categories (Illston et al., 1979; Pohlman, 1987):

- Dry corrosion
- Wet corrosion
- Corrosion of immersed metals and alloys
- Induced or accelerated corrosion (includes influence of stress)

The following discussion is confined to corrosion of cablebolts and as such is incomplete as a comprehensive examination of general corrosion.

Dry Corrosion

Dry corrosion is an inevitable consequence of medium- to long-term storage of cablebolts in even the most ideal conditions. It involves the formation of iron oxide (Fe0) as iron atoms combine with atmospheric oxygen. Once the process initiates on a clean surface, it spreads fairly rapidly to involve most of the exposed surface. While Fe0 forms an adherent film on steel surfaces and can actually form an impervious layer, it can be vulnerable to cracking and as such fresh iron is constantly being exposed and the process continues. In the perspective of cablebolting in mining, however, dry oxidation is a relatively slow chemical process and is of only minor consequence. Light surface (dry) corrosion has been shown (Goris, 1990) to improve bond performance of cablebolts by up to 20% in ideal conditions, although **deliberate rusting of cablebolts is not advocated by the authors**. The process is accelerated by higher surface temperatures (e.g. if the cables are exposed daily, over long periods, to direct and intense sunlight).

Heavy surface rust on newly shipped cables is usually the result of exposure to moisture and subsequent atmospheric corrosion which can be very detrimental to the performance of the cablebolts.

Wet or Atmospheric Corrosion

In a wet or humid environment, the corrosion process is accelerated and can involve a wider variety of cathodic reactions. Water and oxygen become jointly involved in the cathodic reaction and result in other compounds such as $2Fe(OH)_3$, Fe_3O_4 (magnetite), or Fe_2O_3 (hematite). These compounds are much less adhesive then FeO and less likely to form a self-arresting film.

Corrosion products formed on cablebolts by wet corrosion are more likely to have a greasy feel as compared to the dry, rough texture of FeO film and are more likely to be associated with other film substances such as oils and additional moisture. These products are likely to have a detrimental effect on bond capacity of cablebolts. Clearly, unchecked corrosion reduces the cross-sectional area of steel in the cable and ultimately reduces the tensile capacity of the steel to unacceptable levels. Ductility and displacement capacity is also reduced (embrittlement).

The presence of water on the surface of the cablebolt also increases the potential for galvanic corrosion. The same wet corrosion cathodic reactions occur, accelerated by the presence of an electrolyte such as chloride, sulphate or hydroxide. Without electrolytes in a static solution, the corrosion process is self-limiting. Iron ions (e.g. Fe^{2+}) move into solution adjacent to the steel surface leaving behind free electrons ($2e^-$) in the steel solid. The concentration of iron ions in solution and free electrons in the steel creates an electrical potential difference which resists further dissolution of iron ions.

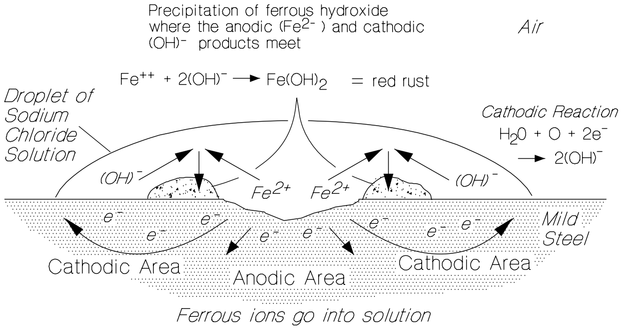


Figure 2.4.5: The mechanisms of rusting of mild steel (after Illston et al., 1979)

The effects of electrolytes in the surface water is best illustrated in the above example. A drop of water on the surface of the steel contains a dissolved electrolyte such as sodium chloride (which forms a solution of free sodium, Na^+, and chloride, Cl^-, ions). The presence of electrolytes permits the transport of iron ions as FeCl away from the corrosion (anode) site at the centre of the drop. At the same time, water and oxygen combine at the perimeter of the drop with the free electrons from the steel to form hydroxide ions (OH^-) balanced by Na^+ in solution. These move in the opposite direction to the FeCl generating a current (electron flow) in the steel supplying electrons to the drop perimeter as more iron ions go into the solution at the drop centre. Between the active centre (anode) and the drop perimeter (cathode) the iron ions combine with the hydroxide to form ferrous hydroxide. This in turn becomes a relatively stable and complex hydrated oxide known as rust. The sodium and chloride transport ions are freed to carry on the process. The cyclic nature of the process combined with the fact that the corrosion product (rust) is not deposited at the anode (as it is with dry corrosion) means that this form of galvanic corrosion is not self-limiting and can be very aggressive. This is particularly true in mining environments given the high concentration of chloride and sulphate ions in mine waters (Minick and Olson, 1987).

Moist corrosion is particularly enhanced by crevices such as those formed by the flutes of a cable. Crevices are particularly good at retaining moisture and the conditions are perfect for differential aeration with low oxygen supply at the tip of the crevice compared with the rest of the cable. If a weak electrolyte is present, an aggressive corrosion cell is thus generated. This corrosion is particularly detrimental as the corrosion product (rust) readily fills the flutes of the cable preventing the penetration of grout and seriously reducing the cable/grout interlock essential for cable bond strength.

Immersion of Metals and Alloys

It is the differential electrical potential between the anode (+) and the cathode (-) which is key to the moist corrosion example described above. This differential is primarily generated by the difference in oxygen availability between the edge and the centre of the water droplet.

Differential potentials can also be generated by the presence (and contact) of dissimilar metals immersed in an oxygenated electrolyte solution (Illston et al., 1979; Bryson, 1987). Corrosion induced by such a coupling can be extremely aggressive and can result from the designed use of dissimilar metals (steel cables with aluminum plates or anchors) or from the presence of cablebolts in a rich sulphide ore. Indeed, rock bolts in sulphide ore bodies have significantly reduced service lives (Hoey and Dingley, 1971; Gunasekera, 1992).

Corrosion cells can also be generated on cablebolt surfaces at the point where abrupt transitions in environment occur. These include differential grout coverage, for example, at the borehole collar, at penetrating cracks in the grout, where the cable crosses a local water table, or within voids in the grout column.

Oxygen (atmospheric or dissolved) is the critical component of the cathodic reaction discussed so far. The concentration of oxygen is therefore a critical factor governing the rate of corrosion. In aqueous environments with high levels of acidity or low pH, however, the hydrogen (H^+) ions in the acid solution react cathodically with the free electrons in the steel to form hydrogen gas (H_2). This reaction is countered as before by the release of iron ions from the steel and does not require the presence of oxygen. While oxygen concentration normally controls corrosion rate (loss of iron ions), the acid (H^+) reaction dominates below a pH of 4 and can become extremely aggressive (Figure 2.4.6).

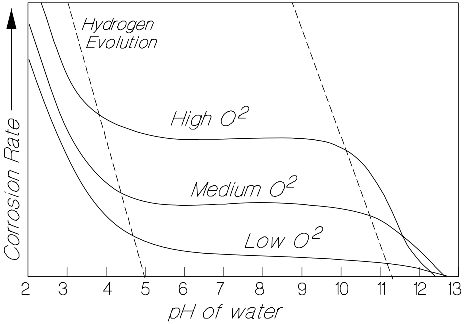


Figure 2.4.6: Corrosion rates for immersed carbon steel (after Bryson, 1987)

Although it is not as common as oxygen related corrosion, acid corrosion can pose a serious hazard to mine support (Gunasekera, 1992) due to its accelerated rate. Sampling of groundwater and/or mine water for pH is relatively simple so the risk can be easily determined. In Canada, mine water with a pH of 2.8 has been recorded in underground mines, and measurements of 3-4 are not uncommon (Minick and Olson, 1987). Acidic mine water can often be linked to the oxidation of sulphide ores (primarily pyrite and marcasite) resulting in the generation of sulphuric acid and pH levels as low as 1.5-2 (Gunasekera, 1992).

In addition, there are many species of bacteria which flourish in the underground environment and which greatly accelerate the breakdown of sulphides to form sulphuric acid. Different species are active with and without the presence of oxygen. Such bacteria can accelerate the production of acid in mine waters by a factor of four with a related increase in corrosion rate.

Accelerated Corrosion

Of primary consideration in cablebolting is the acceleration of any of these corrosion processes at points of excessive strain in the cablebolt. As steel is strained in tension or in shear across a joint in the rock by rockmass movement, or bent by improper plate installation, the susceptibility to all forms of corrosion increases. Any protective surface rust is cracked by such strain exposing fresh surfaces. Microscopic cracks formed in areas of high strain create corrosion conduits beyond the steel surface. In addition, the strained ionic bonding in the metal increases the potential for iron-electrolyte interaction and hydrogen embrittlement (Littlejohn and Bruce, 1975).

This so-called stress corrosion cracking is important because cables will tend to corrode much more rapidly in aggressive environments exactly when and where their mechanical integrity is most tested and is most critical. In the case of grouted cablebolts, load concentrations along the cable length are usually related to full cracking and separation across the grout column. This allows direct and focussed attack on the stressed steel by corrosive agents. Stress corrosion is often the final mechanism in cablebolt failure in corrosive environments.

Cablebolt Geometry Effects

In general, the high carbon steels used in the manufacture of cablebolt strand are more corrosion resistant than the steels used in conventional rock bolts. Nevertheless, certain features of the grouted cablebolt which increase its potential for detrimental corrosion include the presence of flutes (v-grooves), internal channels between the outer wires and the king wires, as well as the formation of concentrated corrosion sites at separation planes in the rock and grout. Voids and bubbles in the grout column also create potential corrosion cells.

Summary Recommendations for Corrosive Environments

Corrosion is rarely a problem in open stope cable support, simply due to the short service life involved. Cut and fill stopes can be open for up to a year or more and overhead cables should, therefore, not be allowed to corrode to unacceptable levels during this time. Fractured, sulphide ore bodies require special attention in this regard. Corrosion of cablebolts (and other steel support) in permanent mine openings can cause serious problems in terms of safety and rehabilitation. In addition to normal capacity reduction, corroded cables tend to become brittle and can suffer reduced effectiveness in dynamic loading situations. The factors which contribute to corrosion are often complex, are compounded in an underground environment, and are very difficult to combat in areas of high severity. Nevertheless, the following is a brief list of remedial measures for use when corrosion has been identified as a problem (Littlejohn, 1990; Gunasekera, 1992).

Cablebolt storage

- Store cablebolts in a dry location, preferably moving them underground to the working site only when required. Long-term storage outside, under the sun or exposed to the elements should also be avoided.
- Do not allow water to collect on the cablebolts. Corrosion will quickly fill the flutes reducing bond strength and potentially pitting the steel.

Installed cablebolts

- High humidity accelerates corrosion. Good ventilation at all times can help to reduce this factor.
- Use caution when installing cables in areas with flowing water.
- Avoid any use of cements, mixing water or admixtures containing chlorides, sulphides or sulphites.
- Grout voids and bubbles increase corrosion potential.
- Request that plates, barrels and wedges, and other fixtures are electrochemically compatible with the high strength carbon steel used in strand.
- Long rust stalactites growing rapidly from the ends of uphole cables indicates potentially severe strand corrosion up the hole.
- Sulphate resistant grouts are alkaline and can counteract acidic mine waters. The use of this cement does not permit the use of such waters for grout mixing.

Severe corrosion

- Epoxy-encapsulated cables are available for use in corrosive environments (Windsor, 1992). Note that such coatings may not be resistant to all forms of corrosion and that the coating must penetrate the strand, encapsulating the king-wire to prevent focussed corrosion down the centre of the strand.
- Galvanized cable would be of use against non-acidic corrosion.
- Grease can protect ungrouted lengths of cable (at the collar, for example).
- Other more costly measures such as cathodic protection are discussed in Littlejohn and Bruce (1975) and Littlejohn (1990; 1993).

2.5 Grout

Cablebolts are classed as continuously coupled, friction anchored devices. This means that there is a continuous coupling between the cable and the rock mass. This is achieved through the use of grout. While this handbook focusses on Portland cement grouts, there are several other material options:

Resins

Other more rigid grouted systems such as grouted rebar, can be effectively grouted with epoxy-resin grouts. Other chemical products such as urethane and other polymers have also seen limited use. The main problem with these chemical grouts when used with cablebolts seems to be with installation. It is difficult to spin a cable into a hole full of grout cartridges as is done with rigid rebar. Pumping and placing chemical grouts is often an unpleasant experience in an underground environment and may conflict with local safety regulations. Progress is being made, however, in these areas (Goris and Tadolini, 1993).

Shotcrete

The use of shotcrete for cablebolt grouting would have advantages in mines already using shotcrete and would reduce equipment requirements where the two support systems are used together. Hassani and Rajaie (1990) have pull-tested shotcrete grouted cablebolts in the laboratory and have obtained favourable results showing slightly reduced initial bond stiffness followed by comparable load capacity and enhanced residual strength at large displacements.

Portland Cement

Portland cement grout without aggregate remains the primary grout material. The design of the grout for cablebolt applications is often critical to success of the operation. Proper grout design is based on the following considerations:

Installation considerations:
- Installation method
- Cablebolt geometry (type)
- Mixing time
- Pumping rate, efficiency and limits
- Initial setting time

Performance considerations:
- Cablebolt geometry (type)
- Bond strength and load transfer mechanics
- Curing time
- Grout performance with time
- Environmental sensitivity (corrosion, etc.)

These requirements can be mutually contradictory in certain circumstances. It is important to reach an adequate compromise when designing a grout mix.

2.5.1 Composition of Cement Grout

The most common form of grout used in cablebolting, and indeed in over 90% of grouting and concrete operations (C.P.C.A., 1984), is ordinary Portland Cement. It is made from limestone and clay or other suitable materials, heated together and pulverized to form a powder (particle size in the range of 0 to 0.05mm) which is rich in calcium silicates (C.C.A., 1968; Mindess and Young, 1981; Hyett et al., 1992). Small amounts of gypsum (calcium sulphate) are also added to moderate the behaviour of the final product when mixed with water.

The final product contains approximately 45-60% C_3S, 15-30% C_2S, 6-12% C_3A, and finally 3-8% C_4AF, where C = CaO (lime), S = SiO_2 (silica), A = Al_2O_3 and F = Fe_2O_3 (Ferric Oxide). The relative content of these minerals controls the behaviour of the cement product (Mehta, 1986; Hyett et al., 1992) when mixed with water and allowed to hydrate. This hydration is a chemical reaction which evolves heat and forms new compounds and ultimately a strong solid mass. It is not a drying process and it is not reversible (**it is impossible to "reuse" hydrated clumps of cement**). Figure 2.5.1 below illustrates the contributions to strength and curing rate of the individual cement minerals. This gives an indication of the effects of varying the proportions of these minerals.

The grain size is equally important in determining behaviour. The rate of cement hydration (reaction with water) is a function of the *blaine* or total particle surface area within a unit mass of cement. The greater the surface area per volume of cement (i.e. the finer the grout), the faster the reaction (set).

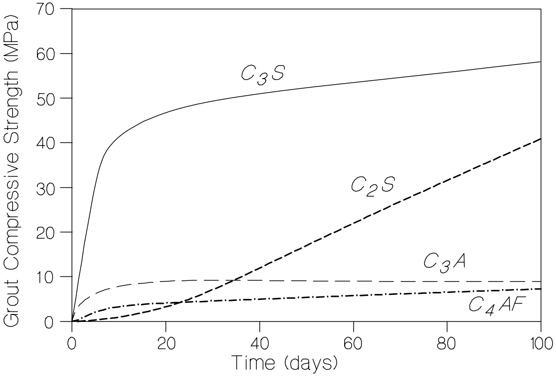


Figure 2.5.1: Compressive strength development in pastes of pure cement compounds (after Hyett et al., 1992 and Mindess & Young, 1981)

2.5.2 Varieties of Portland Cement

There are several types of cement which are applicable for use in cablebolting. They are all variations of normal or ordinary Portland cement. They contain the same basic minerals but in different proportions. There may be other acceptable variations which utilize various combinations of fly ash, blast furnace slag and other materials, although testing for strength and performance in the operating environment should be carried out. Most of the grout related discussions in this book focus on ordinary or normal Portland cement.

Ordinary or Normal Portland Cement

This is the most common and least expensive material for grouting. Only in special circumstances should it be necessary to use a different grade of cement. Typical concentrations of cement components in normal Portland cements such as the Canadian specification *Type 10* are approximately 50% C_3S, 25% C_2S, 12% C_3A and 8% C_4AF. The typical blaine of normal Portland cement is approximately 350 m^2/kg. This value can vary between manufacturers and accounts in large part for the variation in performance of otherwise identical batches of cement (Cortolezzis, 1991; Hyett, Bawden and Coulson, 1992).

Rapid-Hardening or High-Early Portland Cement

High-early cement (Canadian *Type 30*) contains a greater proportion of C_3S relative to C_2S and a smaller grain size (and therefore higher blaine - 450 m^2/kg). This gives it a higher early strength. The final strength (after 28 days) will not be greater than ordinary Portland cement, however, and in fact may be slightly less due to shrinkage and embrittlement. The finer grain size and higher initial reactivity may also lead to premature set and lumpy grout during storage in a humid mine environment. It is advisable to perform bond testing (Section 2.2.2) at lower water:cement ratios (<0.35) to ensure that grout disintegration due to early heat generation and expansion is not a problem. Such difficulties have been reported (Oliver, 1995, personal communication).

Portland Blast Furnace Cement

This cement is made by combining normal Portland cement with suitable granulated blast furnace slag. Concrete standards permit the maximum proportion of blast furnace slag (by weight) of 65%. This cement can be used in cablebolting with no appreciable reduction in cement quality after hydration. It may tend to gain strength more slowly and evolve less heat during hydration. In concrete applications it is often claimed to be more resistant to chemical attack, although this has not been investigated in cablebolting applications. The primary advantage to this type of cement is its reduced cost in regions where blast furnace slag is readily available.

Sulphate-Resistant Portland Cement

This cement contains a lower proportion of C_3A, and a higher content of C_4AF. The reduction in the former increases this cement's resistance to attack by sulphate compounds which can be abundant in some mine waters. Sulphate attack is a form of corrosion and can seriously degrade concrete strength with time. In short term applications, sulphate resistant cement would be required when the sulphate concentration in the groundwater exceeds 2500 parts per million. Greater than 1200 parts per million of SO_3 would suggest its use in long term installations (C.C.A., 1968). It is no more resistant, however, than normal Portland cement to acids or other dissolved salts which attack cement. Sulphate resistant cement is also suitable for use in high temperature environments at depth in mining where normal Portland cement may be predisposed to excessive cracking and shrinkage. Special low-heat cements are also available (C P.C.A., 1984) but these are usually expensive and of little advantage for cablebolting.

Low-Heat (Portland and Blast Furnace Slag) Cements

There has been some use of "low-heat Portland cement" and "low-heat blast furnace slag Portland cement" in cablebolting applications where the combination of internal rock temperature and the heat evolved during the hydration of cement has been thought to cause temperature related problems such as cement cracking and shrinkage. It is unclear whether this is a real problem in the temperature ranges encountered in rock. White, powdery grout in the borehole is an indication of excess heat generation. Sulphate resistant cements and regular blast furnace slag cements both evolve heat more slowly than normal Portland cements and are an adequate and more economical option in high temperature environments.

Silica Fume Portland Cement

The addition of silica fume, an extremely fine (smoke-sized) particulate by-product of the production of silicon metals, results in a higher strength and faster setting grout mix. The extremely fine particles and the high glass content of silica fume creates an enhanced hydration reaction, increasing strength and reducing porosity. As a result, this cement can provide higher cablebolt bond strengths (Hassani et al., 1992) and improved protection in chemically aggressive environments.

Sanded Cements

Some research has involved the addition of sand to the cement mix (Goris, 1990; Gendron et al., 1992). This increases the base strength and stiffness of the grout. It is unclear, however, whether this increases the performance (i.e. bond strength) of grouted cables.

2.5.3 Care and Quality of Cement and Water

Anhydrous Portland Cement, (Dry Powder)

The prime concern with respect to storage and transport of grout is prehydration. Due to moisture influx, the cement powder will begin to set (hydrate) and take on a hard or lumpy texture. Portland cement cannot be reconstituted once it begins to hydrate. Lumps will remain in the grout and adversely affect the strength and stiffness within the grouted borehole column and will therefore reduce cablebolt performance. In addition, lumpy grout can cause numerous problems during installation of the cablebolt. The following guidelines should be observed:

- 'Rapid-hardening' or `high-early' cement is particularly susceptible to prehydration and should not be used in mining applications unless maximum cable capacity is needed in less than 7 days from grouting.
- Plastic lined bags are not recommended due to condensation (and hydration) problems within the bags. Shrink-wrapped stacks of bags are preferable.
- Reduce the storage time on site - **Order small!...order often**!
- Upon arrival at the mine site, the bags should be checked. If "hard" bags are detected, the whole batch should be returned.
- Store the bags above ground in a dry warehouse until ready for use.
- Less than 3 days before use, transport underground on a raised flat and covered with a waterproof tarpaulin. Multiple tarps should have generous overlap.
- Do not stack bags higher than 1.5 m to avoid compaction and set.
- When grout bags are being emptied into the mixer, any hydrated lumps should be screened and discarded. Do not attempt to break up the lumps to fit through the screen for use in the mix.

Water

Concrete standards require that water for cement production be potable (drinkable). This will ensure purity standards required for adequate concrete quality. In cablebolt grouting operations, this is also the optimum condition. If this is not possible or practical (as is usually the case underground), then the following guidelines should be observed:

- Grout strength tests should be performed as described in the next section. Cement mixed with mine water should achieve in excess of 90 % of the 28 day strength of the cement mixed with distilled water. Set times should be monitored and compared for acceptability.
- The chloride content of the water should be low. Saline water will result in limited deterioration of cement strength but will cause unacceptable corrosion of cable steel.

2.5.4 Properties of Fresh Cement Paste

When anhydrous cement is mixed with a quantity of water, the resultant mixture is called fresh cement paste. This is the state in which the cement is pumped into the borehole for cablebolt grouting. When the phrase "pumping grout" is used in this and other literature, the reference is, of course, to the pumping of fresh cement paste. This section defines some of the important engineering properties of fresh cement paste.

Water:Cement Ratio (W:C)

This is the most important property of cement grout or paste. It is defined as the ratio of the mass of water (M_W) used to create the fresh cement paste mix, to the mass of anhydrous cement powder (M_C) used in the same mix. Remembering that the weight of water in kilograms is equivalent to the volume of water (V_W) in litres;

$$W{:}C = \frac{M_W(kg)}{M_C(kg)} = \frac{V_W(litres)}{M_C(kg)}$$

Water Content (W_c)

This is an alternative expression to describe the composition of fresh paste. This is defined as the volumetric percentage of water with respect to V_{CP}, the total volume of cement paste (V_C of anhydrous cement + V_W of water);

$$W_C = \frac{V_W(litres)}{V_{CP}(litres)} = \frac{V_W(litres)}{V_W + V_C(litres)}$$

and

$$W_C = \frac{1}{1 + \dfrac{\rho_W}{\rho_C \times W{:}C}}; \quad W{:}C = \frac{\rho_W}{\dfrac{\rho_C}{W_C} + \rho_C}$$

where $\rho_W = 1.0$ kg/l and $\rho_C \approx 3.15$ kg/l (Hyett et al., 1992) are the densities of water and anhydrous cement respectively.

Most of the discussion in this book refers to $W{:}C$ and not to W_C. The conversion is presented here for comparison with other literature.

Wet Bulk Density (ρ_c)

Wet bulk density is the mass per unit volume of fresh cement paste (freshly mixed and unset). This value is calculated by simply filling a container of known volume (say 10 litres) with fresh cement and determining the mass of the full container. Subtract the mass of the empty container from this volume. Divide the remaining mass (mass of the cement alone) by the volume of the container (1 litre = $0.001m^3$) to obtain the wet bulk density. There is a theoretical relationship between water:cement ratio, *W:C*, and the wet bulk density (Hyett et al., 1992):

$$W{:}C = 0.757 \times \ln(\rho_{cp} - 1.0) - 0.333$$

This relationship is adequate below a wet density, ρ_{CP}, of $2000kg/m^3$ (*W:C* greater than 0.39). However, due to air entrainment in thicker grouts, the relationship, which applies to fully saturated cement pastes, breaks down. Below a *W:C* of 0.33, there may be insufficient water available to completely saturate the cement grains (and achieve complete hydration) and/or air pockets can become permanently trapped in the thick paste as it flows. This relationship is also valid for hardened (hydrated) cement samples subject to the same constraints. The graph below shows acceptable bounds to be used when calculating *W:C* from ρ_{CP}, or vice versa. Note that the data shows a wide scatter. This should be kept in mind when using ρ_{CP} as a quality control measure for *W:C*.

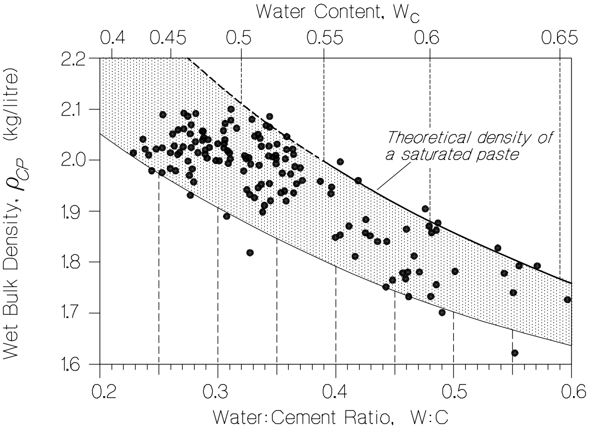


Figure 2.5.2: Wet bulk density of fresh cement paste (after Hyett et al., 1992)

Mixing and Cement Saturation

Dry anhydrous cement particles are inert until moisture adsorbs (bonds to the surface molecules) to the surface and initiates the formation of filament structures which in turn interact with neighbouring particles to form the physical structure of cement. This means that in order to achieve a uniform cement paste with optimum properties, thorough mixing is required. A shear type mixer is one which induces velocity differentials within the grout mix. This means that the grout is not simply spun in the mixing bin but undergoes radial and axial convection (with respect to the bin geometry). The higher the rate of shear, the more opportunity the cement particles have to adsorb hydration water. The most common type of shear mixer uses rotating blades to agitate the grout - water mixture. Key design considerations for grout mixers and pumps are described in Section 2.11.

Viscosity of Fresh Cement Paste

Viscosity is a quantity used to express the resistance of a fluid to internal dynamic shear - as occurs in mixing and pipe flow. Essentially, the higher the viscosity, the greater the resistance to mixing, handling or flow.

A material which has a low viscosity such as water requires little effort to transport at low to moderate pumping velocities (normally, internal shear increases at higher velocities). The movement, through a constriction of pipe, of a highly viscous material such as a thick grout, requires substantial pressure.

Plasticity of Fresh Cement Paste

Plasticity implies a resistance to static shear. Plastic grout pastes can maintain form at rest (or remain, for example, in an unplugged uphole). Grouts of *W:C* less than 0.375 exhibit a combination plastic-viscous or *Bingham* behaviour.

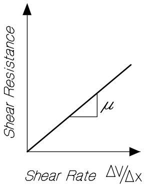
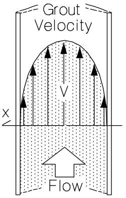
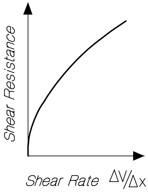
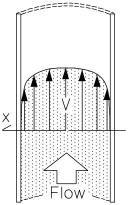
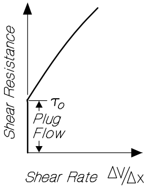
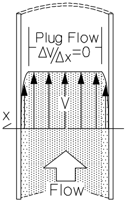
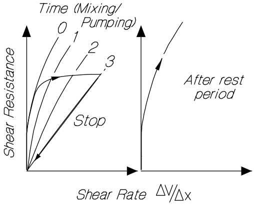
Hydraulic Behaviour of Fresh Cement Paste

Table 2.5.1 gives a brief definition of the various types of flow classifications (I.S R.M , 1991) for cement grouts used for cablebolting. This range of behaviour explains, for example, the designation of *W:C* ranges for:

- Toe-to-collar uphole grouting (*W:C*=0.3 to 0.35) where the grout must remain in an unplugged uphole once placed.
- The grouting of bulbed strand (*W:C* =0.4 to 0.45) where it is necessary to compromise between the strength of a thicker grout and one which will penetrate the bulbs.
- Grout loss problems and associated hazards associated with pumping thin grouts (*W:C* > 0.4) in fractured ground. Thicker grouts (Bingham or thixotropic) would slow and stall in a fracture.

Hydraulic Behaviour of Fresh Cement Paste (cont.)

Table 2.5.1: Key hydraulic classifications of grout flow

W:C	*Hydraulic Classification*		*Conceptual Shear Response*	*Pipe Flow Velocity Profile*
>0.50	▽ ▽ ▽ INCREASING VISCOSITY ▽ ▽ ▽	*Newtonian (Fluid) Flow* A linear relationship exists in laminar (parallel) flow between shear rate or velocity gradient and shear stress as defined by the viscosity constant, μ. That is, faster pumping takes proportionally more effort. Pure water is an example of a Newtonian fluid.		
0.50 to 0 375		*Pseudoplastic* As W:C decreases, the relationship becomes non-linear. Specifically, the spontaneous viscosity is high when the fluid is at rest and decreases with increasing shear rate or velocity gradient. As the mixing or pumping rate increases, further unit increases require smaller increases in effort.		
0 375 to 0.30		*Bingham (plastic)* At low W:C ratios, the grout behaves as a plastic solid at low shear. As internal shear stress increases beyond a threshold value, τ_o, the material behaves like a pseudoplastic fluid. The grout returns to a semi-solid (plug) as shear drops below τ_o. Once the grout stops flowing or mixing, it is difficult to restart.		
0.45 to 0.30 with admix- tures		*Thixotropic* This material can be either pseudoplastic or Bingham in the case of grout. (Thixotropic behaviour is often confused with Bingham). The term describes behaviour in which viscosity decreases with duration of agitation (shearing). The viscosity is restored to the original value after a rest period.		

Workability of Fresh Cement Paste

Closely related to the consistency, viscosity and hydraulic properties of grout is its workability or behaviour during the placement process. There are four main concerns with regards to workability which are often contradictory:

- The grout must be fluid enough (low viscosity) to allow it to be pumped some distance (typically 5m to 30m) along a grout tube and, where applicable along a length of small diameter breather tube. It has already been seen that this flowability and pumpability increases with increasing *W:C* ratio. *W:C* = 0.35 represents a lower limit for some commercial piston pumps while *W:C* = 0.3 has been successfully pumped and placed with more powerful progressing cavity pumps and larger grout hose. It is doubtful that grouts of *W:C*<0.35 can be reliably pumped back down a small diameter breather tube (< 13mm I.D.).

- The grout must be viscous or plastic enough to hold itself in a borehole against the pull of gravity in a top-to-bottom installation. It must also resist flow into open fissures intersecting the borehole. Viscosity and plasticity increase with decreasing *W:C* ratio. Grouts of *W:C* <0.38 fulfil the first requirement and are recommended for uphole installations where a breather tube and collar plug are not used. Grouts up to *W:C*=0.40 can be used in fractured ground providing that the cracks are not extreme in aperture.

- The grout must fully encapsulate the cable and must therefore be fluid enough to penetrate into the grooves of the steel strand cable or must fully penetrate the cage or bulb of modified (flared) strand cables. The ability to fully cover the cable surface and penetrate into the cable geometry increases with increasing *W:C* ratio. Grouts of less than 0.35 are not recommended for flared cables such as bulbed or birdcaged strand. Grouts with *W:C* below 0.35 may not be able to reliably encapsulate the surface geometry of a regular strand cable.

- The grout must resist initial set long enough to allow pumping and placement. Initial set results from a physical attraction between wet cement particles. It leads to an early stiffening of the paste and typically occurs between 10 and 60 minutes after initial mixing for the range of grouts used in cablebolting. This should not be confused with hydration which leads to the ultimate strength of the cement grout. A grout of *W:C*=0.3 will allow less than 15 minutes for placement, while a grout with *W:C*=0.45 will give up to 45 minutes of workability.

It will be shown in later sections that lower water:cement ratio grouts (0.35-0.4) are recommended to optimize the frictional cable/grout bond strength of the cablebolt. These thicker grouts can create slight problems in pumping and flow which can be easily corrected without increasing the *W:C* ratio to unacceptably high values. The following sections discuss these problems and their solution.

Pumpability of Fresh Cement Paste

The hydraulic properties of grouts profoundly influence the pumpability of grouts in underground applications. In particular, the transition from a pseudoplastic to Bingham state causes an abrupt change in pumping rates for many different types of pumps. While the absolute flow rates illustrated below are specific to the type of pump, the brand of pump, the power supply (electricity and voltage, air and P.S.I.), and the attachments (length of hose, etc.), the basic transition in flow characteristic between the water:cement ratios of 0.35 and 0.4 can clearly be seen and would seem to be independent of the pump used.

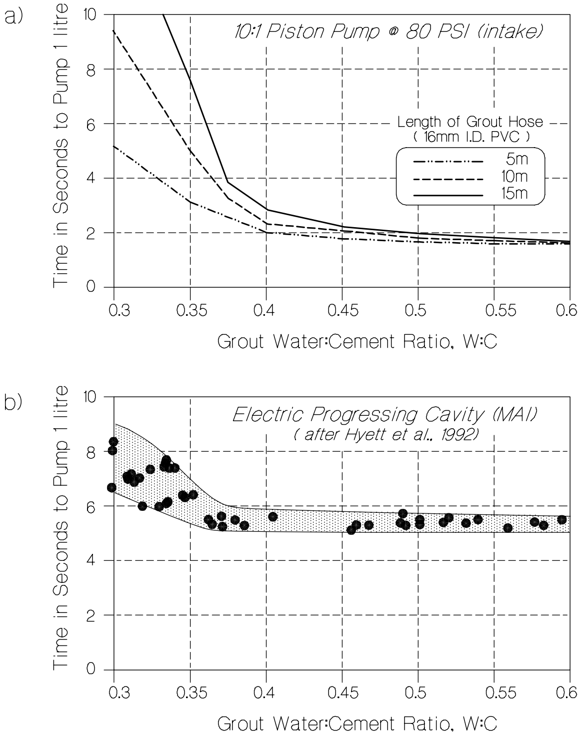


Figure 2.5.3: Typical flow rates vs *W:C* for a) piston and b) progressing cavity pump

Flowability in Grouting Tubes and Breather Tubes

Grout consistency and pump efficiency are often blamed for poor pumping (grout placement) performance in cases where the real culprit is the undersized grout tube or breather tube being used. Calling on some basic principles of hydraulics it is possible to understand why this is the case. Consider a Newtonian fluid with a viscosity of μ, being pumped through a tube of (inside) diameter, D, at a flow rate of Q (e.g. litres/sec). The pressure drop per unit length of pipe, $\Delta p/L$, or the minimum increase in pump pressure required to deliver grout through an additional unit length of tubing is given by (Streeter and Wylie, 1979):

$$\frac{\Delta p}{L} = \frac{128 \times Q \times \mu}{\pi \times D^4}$$

Note that pressure drop is a function of the inverse of diameter, D, to the power of 4. This suggests an enormous influence of tube diameter on pumping efficiency. This relationship is valid only for Newtonian fluids and for laminar (non-turbulent) flow. A grout of $W{:}C$=0.45 can be assumed to fit this description with minimal error. Figure 2.5.4 illustrates this theoretical relationship, using viscosity values back-calculated from pumping data by Goris (1990). A grout of $W{:}C$=0.4 is also analyzed in a similar fashion although the results are less valid due to increased grout plasticity. The graphs do, however, reliably illustrate that:

- Pumping rate decreases and/or pressure drop increases with decreasing $W{:}C$.
- A greater pump pressure is required to pump grout at a higher flow rate. At these higher pressures, a stronger tube may be required to avoid rupture.
- The tube diameter has the greatest influence on pumping capability. **The most effective means of improving overall pumping performance is to increase the breather tube or grout tube diameter** (up to a practical maximum of 20 to 25mm to avoid interference with cable-grout interface).

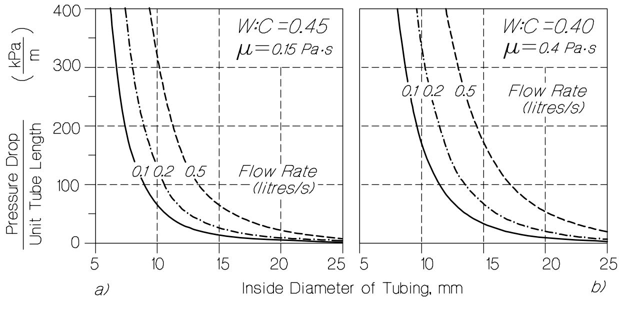


Figure 2.5.4: Theoretical grout pumping performance (efficiency)

Constrictions and Flow

Figure 2.5.4 on the previous page assumes that the flow channel (the tube) is free of constrictions and obstructions. In a real pumping setup and in the borehole itself, these constrictions and obstacles are common. For example, significant pressure drops occur whenever the conduit changes diameter. Unavoidable changes in diameter occur when the grout leaves the grout tube (into the borehole) and when it enters the breather tube. The latter can be severe. These pressure drops can be reduced by cutting the ends of both tubes at an angle instead of across the diameter allowing a smoother flow transition. The number of constrictions and diameter changes at the pump end should be avoided by matching the pump outflow and fittings to the grout tube being used. Where this is not possible, a tapered reducer should be used to reduce the abruptness of the change. Avoid rigid elbows in the outflow plumbing as well.

Flow in the Borehole

Thin grouts $W{:}C > 0.45$ possess near-Newtonian qualities (water-like) and can flow into fractures causing excess grout usage, and can potentially lead to grout voids, incomplete encapsulation of the cablebolt, and installation hazards due to roof jacking in laminated ground. Empty cablebolt holes have been observed in cases where thin grouts were used in highly fractured ground. Semi-thin grouts $0.38 < W{:}C < 0.45$ may not remain in an unplugged uphole containing plain strand cables (Toe-to-collar, grout tube installation) although grouts of $W{:}C = 0.4$ have been observed to remain in unplugged up-holes containing modified (nutcaged) strand. Initial trials may be required before implementing this option.

Flowing grout will take on excess water in a wet borehole resulting in an undesirable increase in $W{:}C$. This mixing action is likely to be more severe in collar-to-toe installations. Use thicker grouts in wet conditions to compensate.

Very thick grouts $W{:}C < 0.30$ are unlikely to penetrate and completely surround the flutes between the wires of the plain strand. In addition, air pockets will be incorporated into the grout column. This problem may be exaggerated when double cables are used. Thick grouts will also be unable to penetrate the "cages" of modified strand and will form voids around spacers and other fixtures. Semi-thick grouts $0.30 < W{:}C < 0.38$ may create difficulties in obtaining return flow through small breather tubes as discussed and may not completely penetrate modified strand geometries such as birdcaged and bulbed strand.

0.35 $W{:}C$ represents an optimum grout for single plain strand cables using a toe-to-collar, grout tube installation. Encapsulation problems have been observed when using this grout with double strand cables. **0.4** represents an upper bound for grout $W{:}C$ and provides an adequately strong grout with good flow properties for use in breather-tube installations and where modified strands are to be installed.

Bleeding of Grout

Bleeding of the grout occurs when the particles of cement settle down to the bottom and water flows up to the top of the grout column. The amount of bleeding increases with increasing water:cement ratio.

In cablebolt installations, the cablebolt acts as a wick, allowing the water to flow along the king wire, meanwhile preventing the cement particles from entering the space between the individual steel wires. The lack of a continuous, protected king wire along the length of modified geometry cablebolts should prevent some of the bleeding and grout settlement observed in tests on plain strand cablebolts. The section of the cablebolt strand which is within the water filled upper section of the borehole will have no load carrying capacity, and the effective embedded length of the cablebolt will be reduced from the design length. Excessive bleeding can disrupt the integrity of the grout column.

Table 2.5.2: Grout bleeding measured in laboratory tests by Goris (1990)

water:cement ratio	*Bleeding factor (m/m)*
0.3	0.016
0.35	0.033
0.4	0.063
0.45	0.096
Results of bleeding tests conducted with both single and twin plain strand cablebolts; bottom and sides of the cylinder impermeable.	
Bleeding factor: Loss of embedment per metre of installed cablebolt.	

A limited number of laboratory tests (using a typical paddle mixer and piston pump) conducted by the authors indicated that when both the upper and lower ends of plain strand cablebolts drained water from the grout mixture, the amount of settlement of the cement particles within the column was approximately double that found in tests where only the top of the cablebolt was free draining. The effect of lateral bleeding into a fractured rockmass was not considered.

The reduction of the embedment length of the cablebolt due to grout bleeding can be fairly significant, depending upon the water:cement ratio of the grout. For most cablebolt orientations, the bleeding of the grout reduces the length of the grout column at the non-working end of the cablebolt. In this case the effect of bleeding can be compensated for by increasing the length of the borehole. Use the following equation to approximate the increase in the hole length required.

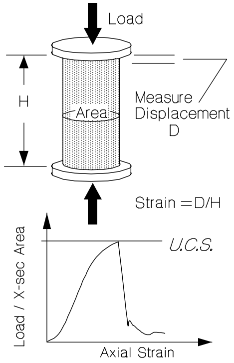
$$L_{design} = \frac{E_{mbedment}L_{required}}{1 - B_{leeding}F_{actor}}$$

2.5.5 Properties of Hydrated Portland Cement

In the cablebolt system, the grout serves to transfer load between the rock and the cable. The most important properties of fully hydrated (cured) cement paste which allow it to carry out this function are strength and stiffness. These properties are functions of water:cement ratio (*W:C*), cement composition, and elapsed time since placement.

Compressive Strength

The most convenient index for describing the strength of cement paste is its uniaxial compressive strength. A cylinder or cube of grout is loaded in a testing machine (A.S.T.M., 1984). The highest load achieved before destruction of the sample is the ultimate compressive strength. It is important to note here that this value depends to some degree on the testing setup and on the sample geometry. Cubes will give somewhat different results than cylinders and larger samples will give slightly different results than smaller samples. It is important to test in a consistent manner when making comparisons. The results illustrated in this section are obtained from cylinders with a height/diameter ratio of approximately 2.5.



The compressive strength of cement grouts is a function of time, composition, and of water:cement ratio. The dominant factor in determining the ultimate strength of hardened (hydrated) cement paste is the initial water:cement ratio (*W:C*) of the fresh cement paste. Figure 2.5.5 illustrates an ideal range of compressive strengths for different grouts prepared in the laboratory. However, the compressive strength of grout in a borehole in the field may be reduced up to 40% due to inadequate mixing caused by the limitations of the mixer and pump.

There is a clear and approximately linear trend towards increasing compressive strength with decreasing *W:C*. Note the increase in variability (scatter) below *W:C*=0.38. This is caused by a change in the hydraulic character of the grout which decreases the mixing efficiency. The reduced water content also means that less excess water is available for complete saturation and hydration of the cement particles. Cements at higher water:cement ratios incorporate unused water into their matrix after hydration, resulting in an increase in micro-voids and reduced strength.

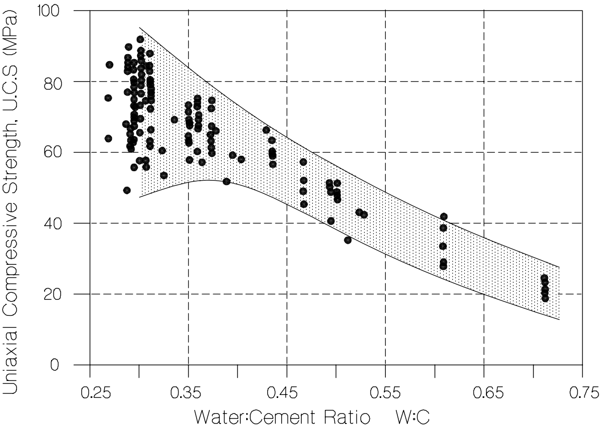


Figure 2.5.5: Uniaxial compressive strength of grout with respect to water:cement ratio (after Hyett et al., 1992)

It can be seen from Figure 2.5.5 that the range of *W:C* = 0.35 to 0.4 provides the optimum balance of strength and minimized variability.

In addition to the cement composition, the fineness of grind will affect the rate of hydration and therefore the rate of strength gain. The finer the grind, the more rapid the strength gain. High-early cements are typically of finer grind but generally result, however, in slightly lower long term strengths.

Mixing efficiency, chemical variability between cement brands, humidity during hydration and water quality will also influence cement strength.

Tensile Strength of Cement Grouts

Tensile strength of the grout is defined as the resistance to tensile stress or the resistance to being pulled apart. This parameter is of minor importance to the overall performance of the grout in cablebolting applications. There is a great deal of variability in even the most controlled laboratory testing (Hyett et al., 1992). The average tensile strength of cement grout of *W:C*=0.4 is approximately 4 MPa. There is a slight trend toward higher tensile strengths at lower *W:C*, but the inherent variability makes it difficult to specify a quantitative relationship.

Elastic Stiffness (Young's Modulus) of Grout

Along with compressive strength, the elastic stiffness of the grout is one of the most important measurable grout parameters affecting cablebolt performance.

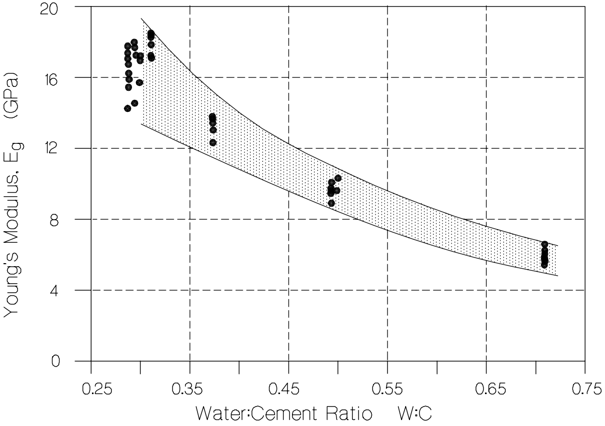
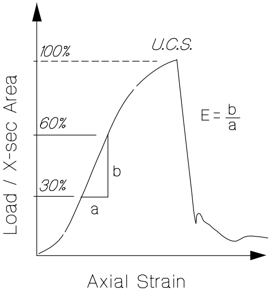


Figure 2.5.6: Elastic (Young's) Modulus of cement grout with respect to water:cement ratio (after Hyett et al., 1992)

Young's Modulus is a measure of elastic stiffness and is obtained from the slope of the graph of axial stress versus axial strain produced during a uniaxial compression test of the specimen. For the data shown in Figure 2.5.6, the slope is measured between points on the curve at 30% and 60% of ultimate compressive strength. The data shows a clear relationship which for practical purposes can be described as linear, with modulus decreasing as water:cement ratio, W:C, increases.



While thicker grouts give consistently higher moduli, the range *W:C*=0.35 to 0.4 again provides adequate results for cablebolting.

Effect of Curing Time

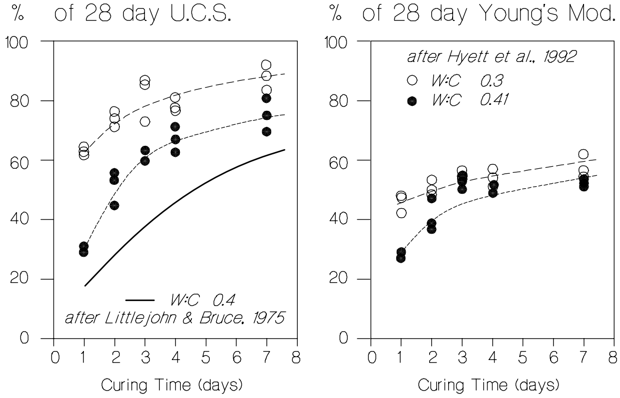


Figure 2.5.7: Increase in strength and stiffness with curing time for cement grouts

Normal Portland cement normally takes approximately 28 days (after mixing and placement to reach complete hydration (effectively 100%) and to obtain optimum strength and stiffness properties. In underground cablebolting, however, it is often necessary to bring the bolts (and therefore the grout) into service before this time. Testing of normal Portland cement (Figures 2.5 7 and 2.5.8) indicates the following trends:

- 40-70% strength gain after 3 days. Cables should never be brought into service (mined through) before this time although plating is possible after 24-48 hours.

- 80% strength gain after 7 days but with only 50% gain in stiffness. Mining at this time is not recommended but is possible when necessary due to scheduling constraints. The use of modified strand reduces the stiffness dependency and may permit faster cycle times when necessary.

- 100% strength and stiffness gain after 28 days. Unless otherwise noted, quoted values for cement strength normally refer to 28 day results.

Higher strength cements such as those containing silica fume can be used to obtain higher strengths (close to maximum) in 2-4 days. Test results involving cablebolts are limited in number but such grouts show promise (Hassani et al., 1992) for accelerating the cablebolting cycle time.

Effects of Curing time (cont.)

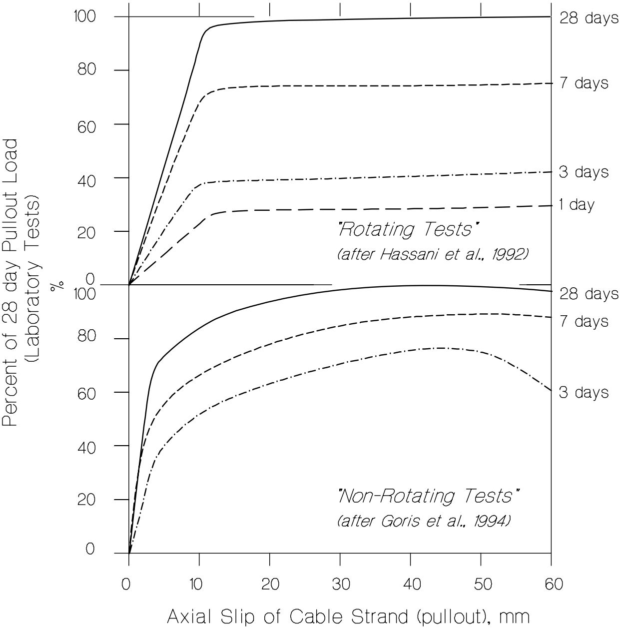


Figure 2.5.8: Increase in cablebolt performance (pullout) with curing time

Environmental Sensitivity of Cement Grouts

Cement grouts are susceptible to attack in highly acidic environments; pH<6 is considered aggressive for long term exposure (Littlejohn, 1993) although this is overconservative for mining purposes - most mine waters would fit this description. Sulphate resistant grouts have a high alkali content and can be used in these environments (Gunasekera, 1992). Sulphates in groundwater also react with the tricalcium aluminate in cement to form salts within the cement structure causing swelling and disintegration. Waters with > 0.5 g/litre of sodium sulphate or > 0.25 g/litre of magnesium sulphate are considered aggressive (Littlejohn and Bruce, 1975). Sulphate resistant cements or blast furnace slag cements provide some protection. If a problem is suspected, soak grout cylinders in samples of mine water for time periods corresponding to the expected service life. Check or test the samples to assess the impact of the corrosive elements in the water.

Blast Damage to Cement Grout

Little testing has been done to evaluate the potential for grout damage due to nearby blasting (say, in a production stope hangingwall). It is the opinion of the authors that serious disintegration of cured grouts (> 3 days) occurs only in very soft or highly fractured rocks. The grout is normally less stiff then the surrounding rock and as such will suffer less damage as a blast wave propagates through the rock. Relaxation tests of tensioned grouted cables in an open pit bench indicated no substantial reduction in cable bond as blasting encroached upon the test bench (Windsor, C.R. and Thompson, A.G., 1993, personal communication).

When it is necessary to blast nearby before the grout has cured for 3 days, Heilig and Espley (1993) conclude, based on a study of concrete damage guidelines and typical blast vibration data from mine sites around Sudbury, that:

> "*Any production blasting within a 24 hour period following grouting of the last cablebolt should not induce a level of vibration (in any direction) exceeding 200mm/s at the cable/grout interface......*Within 24 hours following grouting*, the minimum distance, in metres*, *required between production blasting and any newly installed cables is equal to* 5 times *the square root of the charge weight per delay expressed in* kilograms" and that for charge weights up to 750kg of explosive per delay; *"Any open stope production blasting within 100m of newly installed cables should not occur within a 24 hour period following grouting of the last cable"* and that no production blasting of any weight should occur within 30m of newly grouted cablebolts for the same 24 hour period.

Oriard and Coulson (1980) investigated the effects of vibrations on mass concrete and determined that vibrations below 100mm/s between 4 and 24 hours (minimum of 6 to 8 hours recommended) after concrete placement did not cause any deterioration. This recommendation is further supported by Esteves (1978) and Dowding (1985) although damage to the cable/grout bond is not considered. This guideline can be applied to cables near development rounds. The authors suggest a 24 hour limit for any larger blasts.

A good rule of thumb is that if the rock suffers blast damage (extensive induced fracturing), so will the grout. Minimization of blast damage is normally a priority goal in open stope and drift blasting. Modified (caged or bulbed) strand may reduce the influence of blast damage on cable performance.

Grout may be damaged, however, if the free end of the cable (exposed at the collar) suffers severe impact damage. This can occur in a cablebolted back during a crown blast, if drawpoint cables extend up into the drawcone, in ore passes, or in drift and sill walls where mobile equipment may impact the free end. The vibration can be transmitted down the steel and disrupt the cable-grout interface.

2.5.6 Cement Grout Specifications for Cablebolting

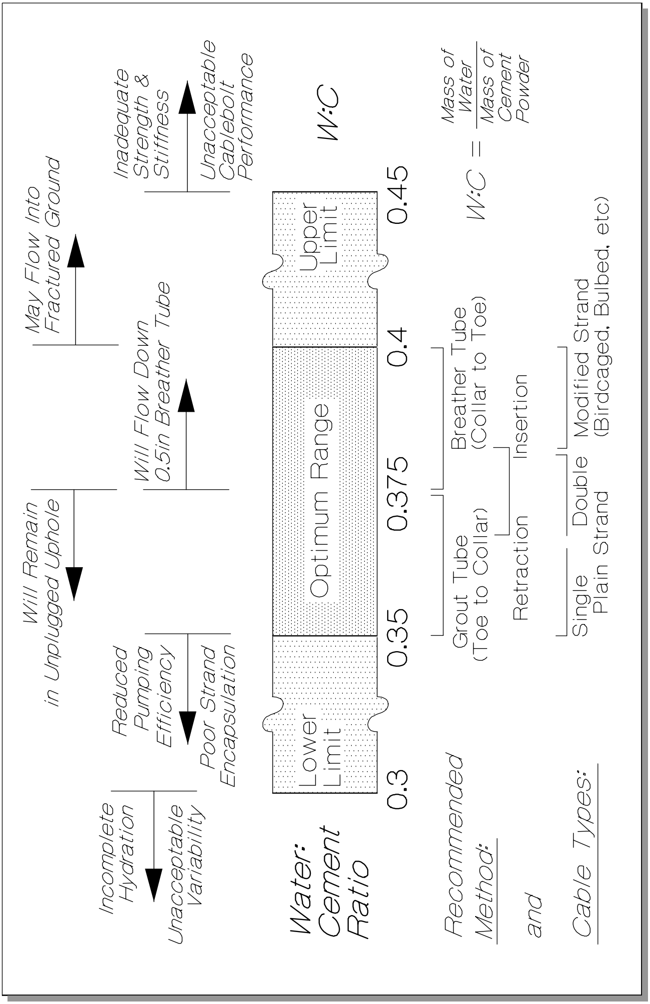


Figure 2.5.9: Summary of grout properties for cablebolting

2.5.7 Grout Admixtures

Grout admixtures are normally organic or inorganic chemical substances added to the grout mixture in small amounts (not exceeding 5% by mass of cement content) in order to physically alter the properties and behaviour of the cement paste. In most cases the additive alters the surface properties of the cement particles during the initial hydration process, then returning them to normal before final set. In cablebolting applications, admixtures can be used to:

- Improve pumpability
- Reduce segregation and bleeding
- Retard or accelerate the initial set
- Accelerate the rate of strength development
- Increase the ultimate strength
- Induce thixotropic behaviour
- Inhibit corrosion (grout and /or steel)

While there are hundreds of unique brand name additives currently on the market, these admixtures fall into the following major categories:

Plasticizers/Fluidifiers/Water-Reducing Admixtures

This class of admixtures allows the use of lower water:cement ratios (and hence higher strength and stiffness) without affecting mixing, pumping and flow characteristics. Conversely, these products can create a more pumpable grout without changing the *W:C* ratio. It is important to consider all workability requirements (including the retention of grout in the borehole) to ensure that the action of these admixtures to solve pumping limitations will not cause unforeseen problems in other aspects of installation and support performance. The effect of these products is usually minor, although excessive dosage can lead to variable setting times, excessive air entrainment and resultant loss of strength.

Superplasticizers/High Range Water Reducers

These admixtures perform the same function as plasticizing and water reducing agents, but they have a much more exaggerated influence on the hydraulic properties. These products should be used with care in cablebolt applications since improper use can have extreme and possibly undesirable consequences. Excessive dosage can lead to increased variability in setting time, excessive air entrainment, strength loss, severe segregation and unacceptable retardation of set and cure. These admixtures should not be used when installing cables in fractured ground and require tight collar plugging in uphole installations.

Air Entrainment Admixtures

Many admixtures perform this function in addition to other functions listed here. In concrete construction, air entrainment is used to generate billions of tiny bubbles in the hardened cement which subsequently act as pressure relief valves for freeze thaw action. Air entrainment also reduces segregation and bleeding and improves initial workability. There is reason to expect, however, that air entrainment may be detrimental to cable/grout interface strength when regular strand cables are used. The interface mechanics are sensitive to the microscopic integrity of the cement grout at the cable surface and air entrainment may be undesirable on this scale. Pull tests should be carried out before implementation.

Retarders

These admixtures are used to delay or retard the rate of set. These may be useful when pumping many holes from the same batch or where other installation requirements may cause long pumping cycles. High temperatures (> 30C) can lead to premature hardening of cement, leading to pumping delays, machinery breakdown and other problems. Retarders can improve the efficiency of the installation process in these environments. It is important to obtain detailed information on these (or any admixture) products. Some retarders may also behave as water reducers or air entrainers and some may reduce initial (3 to 7) day grout strength. It is important that ultimate strength is not affected. Pull tests may be necessary before proceeding. Excessive use of any retarder can lead to extremely long setting times and unacceptable bleeding or grout flow into fractured ground.

Accelerators

Accelerators are used to increase the rate of initial set and subsequent strength development. The use of accelerators is not advised in cablebolting applications. The most common accelerators contain chlorides and nitrates. Chlorides in particular (calcium chloride is a common accelerator) are extremely detrimental to reinforcement steel and of course, to cablebolts. One exception is the accelerator calcium nitrite which also acts as a corrosion inhibitor for encapsulated metallic elements. Overdose of accelerating admixtures can result in erratic setting and strength development, cracking and loss in ultimate strength. If acceleration of set is desired (in cold environments, tight operational schedules, need to plate immediately) it is more desirable to use lower water cement ratios, or "high early" (strength) cement.

Water Retention Admixtures

These reduce bleeding and water loss and improve the uniformity of hydration and strength.

Thixotropic Agents

These admixtures create a higher *W:C* cement which behave plastically at rest and at low pumping rates. Once in motion for a period of time, as in the mixing and pumping process, the grout behaves in a fluid manner. After cessation of pumping, or if grout penetrates into a crack (with subsequent decrease in flow velocity) the grout will return in a short time to a plastic state. This type of admixture may be desirable for top down grout tube installations with no collar plug or in highly fractured ground. Continuous mixing of the grout is important throughout the pumping process. The grout may achieve a false set around the pump intake or auger feed if mixing is not maintained.

Strength Enhancing Agents

These admixtures can be employed to achieve higher strengths in the cement. Agents such as silica fume increase ultimate strength in cement, improve cable/grout bond (adhesional and dilational) and can improve corrosion resistance of both the cement and the steel cable. There is little experience with this type of additive in cablebolting but its use is widespread in steel-reinforced concrete applications.

Corrosion Inhibitors

It is important to define the type of anticipated corrosion (Section 2.4.5):

- conventional (oxygen and electrolytic) corrosion of the cable steel,
- acid attack on both steel and cement, or
- sulphate attack on the cement.

There are a number of additives which can improve resistance to one of these forms of attack but which may be useless against another. Examples are calcium nitrite (an active corrosion inhibitor which acts to stabilize oxide films on steel), silica fume or any additive which reduces bleeding, shrinkage or air entrainment.

Conversely, some admixtures such as accelerators may contain compounds (chlorides, etc.) which accelerate corrosion and should be avoided.

Expansion Agents

These agents create an expansion in the grout during set and can compensate for shrinkage and bleeding. They may also increase immediate grout pressures, improving bond stiffness and frictional resistance to pullout. While some expanding cements may be appropriate for cablebolting, avoid admixtures which function by increasing the concentration of micro-voids which could impair the cable-grout interface strength. These grouts should be evaluated in confined pullout tests prior to use underground.

General Guidelines for Admixture Usage

Due to the literally hundreds of cement admixtures and brand names available on the market it is impossible to give guidelines for any particular product. The following are some general guidelines for usage.

- It is important to define the desired properties to the supplier. One option is to determine flow, workability and strength properties in terms of apparent water:cement ratio. For example, you may specify that you wish to pump at a $W:C$=0.45 efficiency, achieve the uphole stability of a 0.35 cement, while maintaining the long term strength of a 0.3 cement.

- Check into the experience of the admixture supplier with underground environments and with reinforcement grouting. If this is limited, the admixture must at least be approved for application in steel reinforced concrete works.

- **Always perform complete testing for an untried admixture** (pumpability, grout flow in tubes and in boreholes, grout strength and pullout resistance).

- Obtain complete specifications for the admixture and obtain complete usage instructions, including mixing sequence, dosages and safety considerations.

- Many admixtures are originally developed for uses beyond cement modification and suppliers may not have cement-related safety information. Many admixtures may increase the existing hazards of cement mixing (e.g. skin irritation) or create new hazards. Be attentive to complaints from crews when initiating an admixture program and perform a safety investigation beforehand.

- Most additives tend to negate the effects of others or may create disastrous consequences in certain combinations. Do not mix different additives and avoid mixing even if they perform the same function. Most suppliers have their own formula for an admixture and this may conflict with that of another supplier.

- Never exceed the recommended dosage and always follow the correct mixing sequence. Some agents must be added to the mixing water before the cement while others are added to the cement paste during mixing.

- Never use any admixture which contains agents which may enhance corrosion of steel. Calcium chloride (or any other electrolytic salts such as sulphates, sulphides or nitrates) must be avoided.

- Many admixtures are organic and have specific storage requirements. Others may be temperature or light sensitive during long term storage. Powdered additives should never be allowed to absorb moisture before use. Liquid admixtures should not be allowed to freeze. Do not use spoiled admixtures.

2.6 Load Transfer

Load is transferred between two separating zones of rock through tension in the cable strand. The cable-grout-rock interfaces must also bear this load transfer. Five modes of grouted cablebolt failure can occur (Figure 2.6.1):

(A) by rupture of the steel tendon
(B) at the cable/grout interface
(C) through the grout column
(D) at the grout rock interface
(E) through the rock surrounding the borehole.

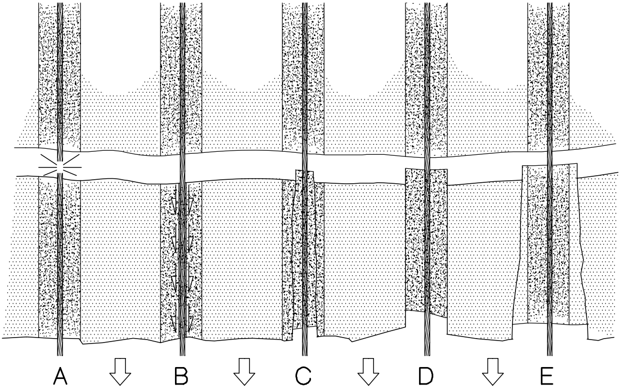


Figure 2.6.1: Possible cablebolt failure modes (after Jeremic and Delaire, 1983)

Strand rupture (A) occurs if the shear loads acting over the embedded surface area of cable exceeds the maximum ***tensile capacity*** of the steel strand. Steel strand capacity has been discussed in Section 2.4.

If strand rupture does not occur, it has been consistently shown (Stillborg, 1984; Yazici and Kaiser, 1992; Hyett et al., 1992; Goris, 1990; Reichert, 1991) that in hard-rock applications, failure will first occur along the grout/cable interface (bond failure) due to inadequate shear resistance or ***bond strength*** when modified (Section 2.9) or plain strand cables are used (B). Other modes of failure (primarily D & E) can occur in soft, weak rocks (Franklin, pers. comm; Carter, 1995) when modified strand cables are used to increase the bond strength.

2.6.1 Bond Strength

For cable installations in hard rock, cablebolt capacity is defined by the properties and strength of the steel tendon, and by the resistance to slip along the cable/grout interface.

The ultimate ***tensile capacity*** of a steel tendon under load is a standardized specification for the manufactured product. The minimum design specifications for 15.2 mm steel strand cables are 20 tonnes (~ 200 kN) at yield and 25 tonnes (~250 kN) tonnes at rupture.

The ***bond strength*** of a cable is defined as the resistance to slip (at the cable/grout interface) along a unit length or a unit surface area of cable. It is useful to think in terms of load/length required to cause slip. Convenient units are kN/m or the mass equivalent, tonnes/m. While the actual relationship between ultimate capacity and grouted length is not always linear, the concept of a normalized bond strength serves to simplify analysis and design.

Embedment length is a term used to describe the active length of grouted cable under unidirectional slip. Consider the simplest example of a slab of cablebolted rock displacing from the excavation face. It is apparent that there are two distinct cable segments to consider. The loading section includes the grouted length inside the slab (embedment length = L_1). The remaining length, L_2 (>> L_1 in this example), forms the anchor section. These two segments or embedment lengths can be considered separately to determine the maximum bond capacity for the system. In the absence of external influences or surface fixtures, it is clear that the shorter embedment length in the slab will dominate the overall behaviour of the system.

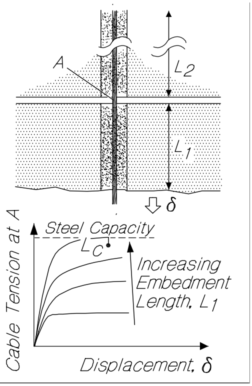


Figure 2.6.2: Embedment length and cable capacity

The total pullout load of a cable increases with active embedment length up to a limit defined by the steel capacity. For example, a cable with a bond strength of 98kN/m or 10 tonnes/m would reach a maximum pullout resistance of approximately 250 kN (25 tonnes) over an embedment (grouted) length of 2.5 m. For embedment lengths greater than this, the cable tendon would rupture during pullout. The minimum embedment length at which cable rupture occurs during pullout (e.g. L_C in above example) is called the ***critical embedment length***.

Critical Bond Strength

The minimum bond strength required over a unit embedment length to sustain a given load (for a given density of rock, for example) is called the ***critical bond strength***. If the actual (calculated in the following sections) or measured bond strength is less than this minimum, design adjustments will be necessary.

For example, cables spaced at 2 m x 2 m in a horizontal roof in a rockmass with a specific weight of 30 kN/m^3 require a critical bond strength of 120 kN/m (2 × 2 × 30) or approximately 12 tonnes/m. The case of a rock slab can be used to illustrate the concept of critical bond strength as shown in Figure 2.6.3.

Select the curve corresponding the unit weight of the rockmass and determine the critical bond strength for a given cablebolt spacing. If the actual normalized pullout strength (load/length) is less than this value, the rock will slide off the cable (bond failure) under gravity loading.

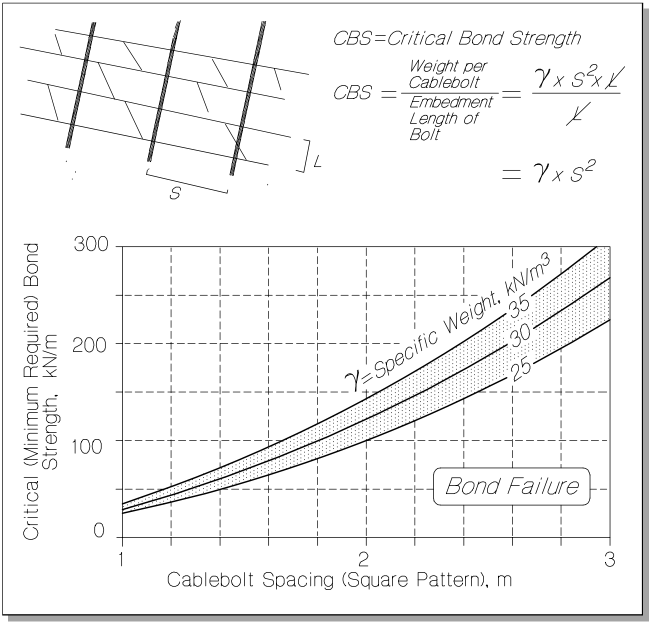


Figure 2.6.3: Critical Bond Strength, *CBS*, for cablebolts - gravity loading

2.6.2 Bond Strength of Plain Strand Cablebolts

Adhesion and Bond Strength

In the context of steel reinforcement of rock or concrete materials, adhesion describes a bonding mechanism (Farmer, 1975; Littlejohn and Bruce, 1975) in which a pseudo-chemical bond develops at the steel/cement interface which is brittle (no residual bond after rupture) and independent of confining pressure (stress normal to the interface).

Typically, for regular carbon steel and cement grouts with *W:C* in the range of 0.35 - 0.5, this adhesion or shear resistance is equivalent to 1 to 3 MPa. Over the surface area of a 15.2 mm diameter cable, this is equivalent to a capacity of 10 kN over a 20 cm length of grouted cable. Unfortunately, this adhesion is exceeded after less than one fifth of a millimetre of relative slip (Fuller and Cox, 1975; Hyett et al., 1992; Nosé, 1993). As such, it is unlikely that adhesion can act simultaneously over any appreciable embedment (grouted) length and rarely accounts for any significant percentage of the instantaneous pullout resistance (bond strength). In fact, as the cable is loaded and begins to slip at the cable/grout interface, a wave of localized adhesion failure propagates down the cable away from the loading site.

Adhesion is thereby rapidly removed from the system as this initial bond is broken and is not considered hereafter as a load transfer mechanism.

Slip, dilation, friction and bond strength

The helical, multi-wire nature of the cable surface creates a negative relief of equivalent geometry in the hardened grout. After adhesion is removed from the interface, the cable slips with respect to the grout annulus. If rotation of the cable during pull-out is prevented, a geometric mismatch occurs between the cable flutes and the corresponding grout ridges. This mismatch increases with increasing relative slip as illustrated in Figure 2.6.4.

As the grout ridges must ride up and over the cable wires, the grout compresses in the confined borehole and thus generates a normal pressure on the grout/steel interface.

Friction (pressure dependent shear strength) thus develops along this interface providing resistance to further slip. This interaction is called dilation. Dilation is limited in the extreme by the absolute scale (height) of the grout ridges. In reality, dilation pressures develop to the point where these ridges crush, reducing the maximum dilation to less than 0.1 mm for plain strand cable (Diederichs et al., 1993).

Dilation is the key to cablebolt performance and is a complex process which is dependent on grout stiffness, rock stiffness and grout strength. This relationship will be explored in the next section.

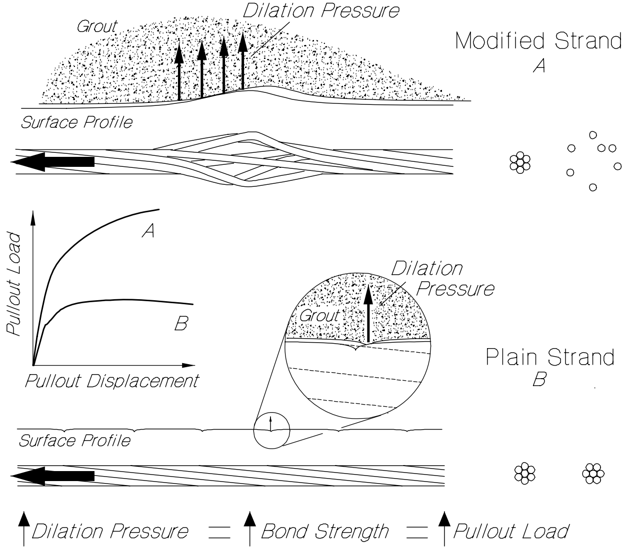
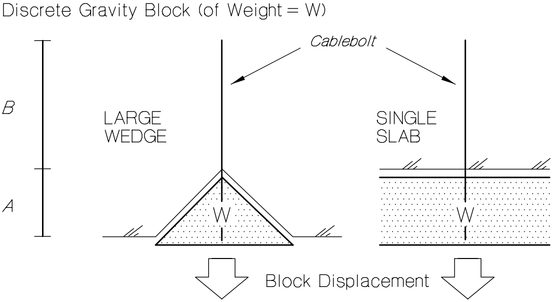


Figure 2.6.4: Dilation and bond strength: modified versus plain strand cable

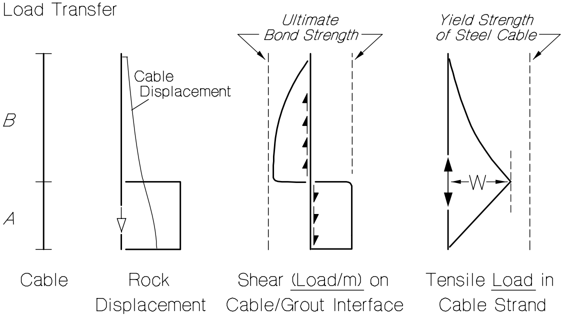
Bond Strength and Load Transfer

Before proceeding with a discussion of bond strength, it is necessary to understand the process by which load is transferred from the rockmass to the cable via the shear resistance at the cable-grout interface. As the rock slips with respect to the cable, shear stresses (load/unit area) are generated at the interface. As these shear stresses accumulate along the length of the cable due to the addition of incremental rock loads, the tension in the steel strand increases (for an unplated cable) from zero at the face to a maximum at some point into the borehole. Beyond this point (i.e. in the "anchor" section of the cable) the shear stresses act in the opposite direction and can be considered as negative. In this region, the loads accumulated in the bottom portion of the cable are transferred back to the rockmass and the cable tension drops back to zero at the upper end of the grouted strand. The following examples illustrate this concept.

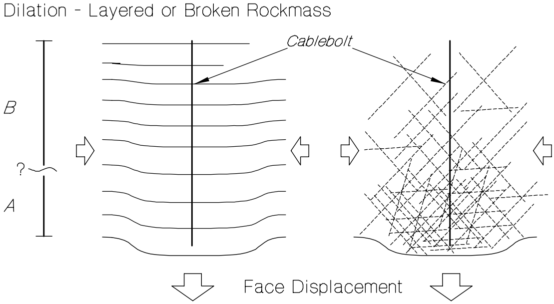
Load Transfer Example: Slab or Wedge



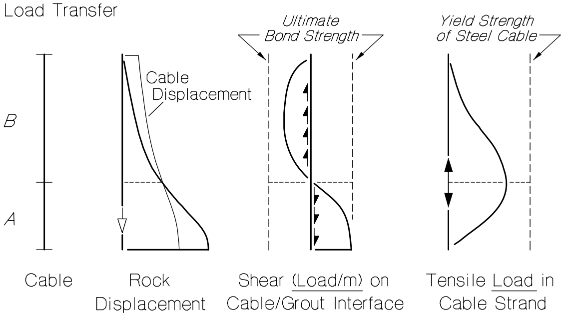
In this example, a slab or wedge of thickness, A (less than critical embedment length), displaces downwards under the influence of gravity. If the ultimate bond strength along segment A is less than the *critical bond strength*, the shear stress acting on the cable-grout interface in section A will become approximately constant as the slab slides along (and off) the cable. During slip, the tension in the steel cable rises linearly from zero at the face to a maximum at the separation plane between A and B. Segment A is called the *pick-up length.* Note that in the *anchor length*, B, the shear stresses act in the opposite direction as the cable tends to slip down with respect to the rock. Section B, in this example, is long enough to transfer the load from A back to the rockmass without significant slip (<10mm). The end of the cable in B may or may not displace at all, depending on the length of B (if A=B, the amount of slip will be equal). The tension in the cable returns to nil at the top of section B as all of the load is transferred back to the rock.



Load Transfer Example: Fractured Ground



The concept of bond stress and load transfer become slightly more complicated when dealing with a fractured rockmass, displacing under gravity or the influence of stress as in this example. Here the displacement profile (Bawden et al., 1995) of the rockmass is assumed to be non-linear, with maximum displacement at the face reducing to nil into the rockmass (at the top of section B). The boundary between the loading section, A, and the anchor section B, becomes undefined. In the lower section (A) the rock has displaced more than the cable (with respect to initial conditions). This generates slip and shear loading on the cable-grout interface and tension in the strand. At some point into the back, the relative displacement between the cable and the rock is zero. This is the neutral point (zero shear and maximum tension) and is the boundary between the pick-up length and the anchor length. Above this point the load is transferred back to the rockmass (Section B) as the shear reverses direction and the cable tension drops back to zero.



A Bond Strength Model for Plain Strand Cablebolts

Numerous models have been developed to explain the complex interactions that occur at the cable/grout interface, and to evaluate the influence of grout quality, rock properties and changes in stress in the rock surrounding the borehole (Hyett et al , 1995; Tan et al., 1993; Fuller et al., 1990). The following is a description of one such model, presented here without proof or mathematical detail to illustrate cable behaviour. Interested readers are referred to more comprehensive references (Kaiser et al., 1992; Yazici and Kaiser, 1992; Diederichs et al., 1993) for details on the formulation and application of this model.

Consider Figure 2.6.5 where the plain seven-wire strand slips past the grout interface. The grout ridges ride up and over the wires, compressing the grout annulus which in turn pushes against the borehole wall. This interaction generates a dilation pressure within the grout and upon the cable-grout interface. For the purposes of understanding the model, the cable is assumed to be a round bar. This bar expands in cross-section by an amount necessary to create a dilation pressure, *p*, equivalent to that generated by the actual cable. The expansion of this bar is represented by a radial displacement, *U*, at the surface of the steel (Kaiser et al., 1992).

Thick cylinder equations (Obert and Duvall, 1967; Popov, 1978) are used to simulate the combined cable-grout-rock system. The rock is approximated by a cylinder of infinite outer radius. In the case of a pullout test in the lab, the radius of the confining pipe is used directly.

The model, illustrated in Figure 2.6.6, incorporates the modulus of the grout and of the surrounding rock or pipe (the cable is assumed to be rigid) and relates a displacement (dilation), U_1, at the inner boundary with an associated increase in interface pressure, p_1 (slope $M = \Delta p_1 / \Delta U_1$). This pressure is related to bond strength through an interface friction angle ϕ_i.

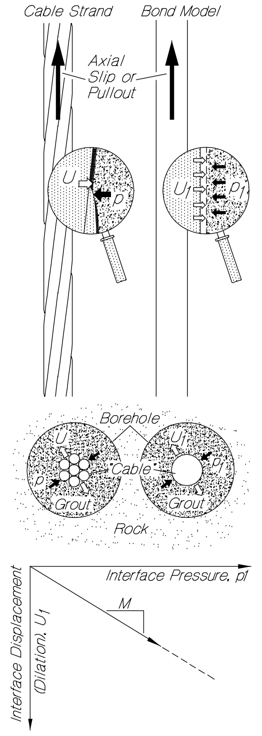


Figure 2.6.5: Bond model

A Bond Strength Model (cont.)

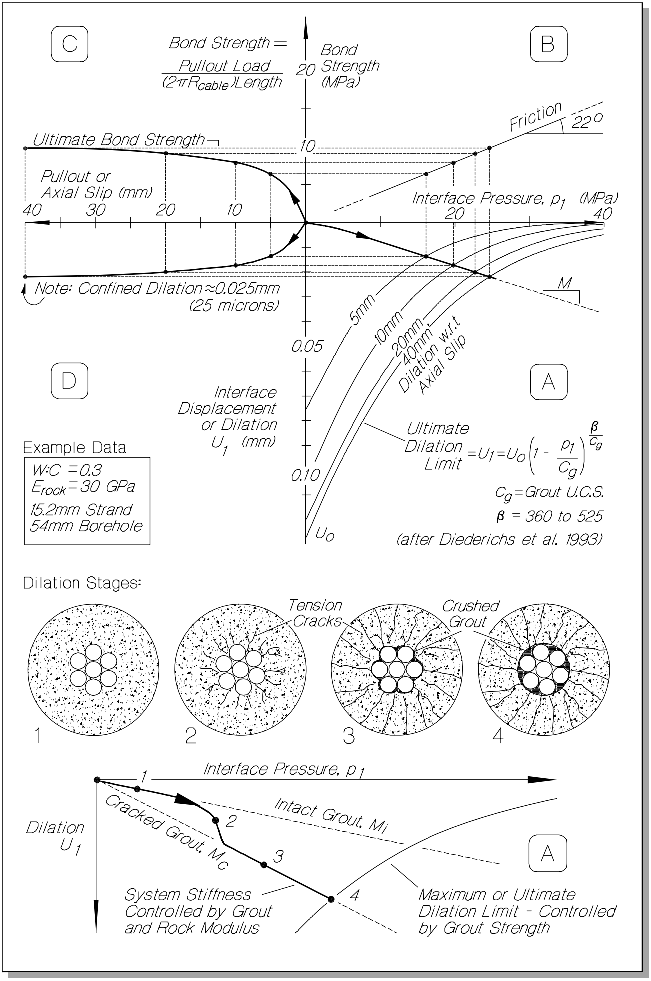


Figure 2.6.6: A model for axial pullout strength of plain stand cables (after Kaiser et al., 1992; Yazici and Kaiser, 1992; Diederichs et al., 1993)

The relationship between radial displacement and interface pressure is dependent on the elastic properties of the grout and rock and is linear for intact grout. Normally, a circumferential tension is generated in the grout annulus resulting in the formation of radial cracks. These cracks in turn reduce the effective stiffness of the grout annulus and therefore reduce the magnitude of dilation pressure at a given radial displacement. Once the radial cracks are fully developed, the relationship again becomes linear but at a lower (less stiff) value of M. At a limiting dilation, the dilation pressure at the tips of the grout ridges reaches a level equivalent to the uniaxial compressive strength of the grout causing crushing of the ridges. This dilation limit is controlled by the grout strength and is confining pressure dependent. The dilation pressure can be directly related to pullout resistance by a friction angle, ϕ_i, for the grout/cable interface. As pressure increases, so does the instantaneous pullout resistance or bond strength. In the model examples, bond strength is represented either as load/length (kN/m) or as shear load divided by the sample cable surface area (MPa). The bond strength corresponding to the ultimate dilation limit is called the ***ultimate bond strength.***

The dilation limit has been calibrated using test data (Diederichs et al , 1993) and can be used in the model to predict bond strength for any combination of key input parameters. An associated computer program, CABLEBOND (Diederichs et al , 1992) is used throughout the following sections to illustrate bond behaviour.

Interface Friction and Bond Strength

Laboratory research (Nosé, 1993) and back-analysis (Diederichs et al , 1993) indicate that the effective friction coefficient (shear resistance / interface pressure) between the steel strand wire surface and the grout is approximately 0.4, corresponding to an average friction angle of 21 to 23 degrees. This range has been independently verified by Hyett et al. (1995).

Light rust increases surface roughness and may increase this angle somewhat (Goris, 1990). Heavy rust or grease, however, may reduce this angle considerably, seriously impairing bond strength (LeClair, 1995; Lappalainen and Pulkkinen, 1982). Note that heavy rust which fills the cable flutes also reduces the potential for dilation and so has a compounded detrimental effect on bond strength. Paint sprayed on the cable serves as a debonding agent via the same mechanisms (Windsor and Thompson, pers. communication).

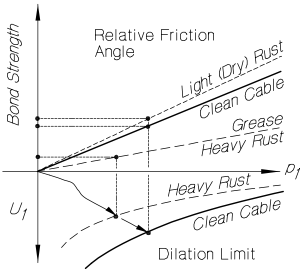


Figure 2.6.7: Interface friction angle

Cable Rotation and Bond Strength (Response Range)

Maximum dilation occurs when the cablebolt is rotationally constrained during pullout. When rotation is permitted, the strand will tend to take the path of least resistance as it slips past the grout interface. Rather than pushing the grout ridges up and out of the way, the strand will tend to "corkscrew" out of the grout column. This results in reduced dilation, interface pressure and pullout resistance (Figure 2.6.8.a). Tests which are not constrained will consistently give lower pullout resistances than constrained tests. The two results do, however, provide an upper and lower bound to actual cable pullout performance in the field (Figure 2.6.8.b).

In addition, it should be noted that even in tests where one end of the cable is constrained, the other end which is drawn in will still tend to rotate. Thus, as test lengths increase, more of the cable within the sample section experiences a rotational slip. Non-rotational bond strengths based on short test sections (less than 30 cm) are not valid for longer embedment lengths. The true strengths measured in laboratory tests are bounded by the ***upper bound*** (non-rotating) and the ***lower bound*** (rotating) strength limits predicted by the bond model (Figure 2.6.8.c).

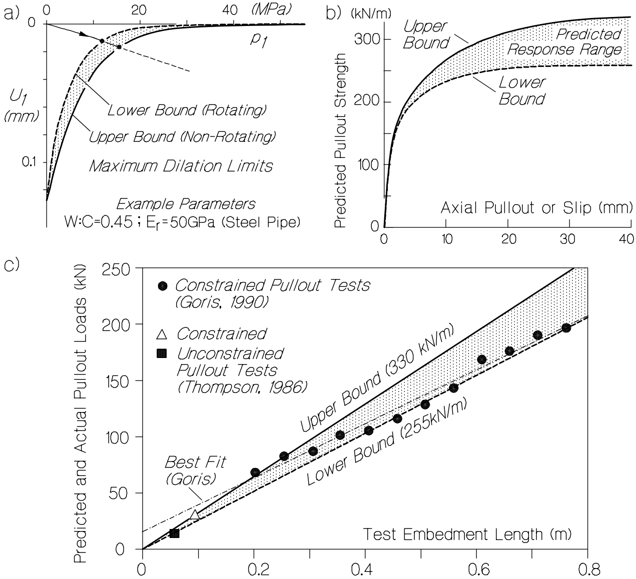


Figure 2.6.8: Cable strand rotation, embedment length, and bond strength

Interface Separation and Bond Strength

The maximum dilation (induced radial expansion) is normally less than 0.1 mm for plain strand cables and for moderate confinement can average 0.02 to 0.04 mm (Diederichs et al , 1993; Hyett et al.,1995). Even minor interface separation can, therefore, be significant. Separation can occur due to numerous influences:

Grout shrinkage and cable bond strength

If significant grout shrinkage occurs, then it is possible that the grout may pull away from the cable before any cable loading occurs. This separation must be closed before any dilation pressure can be generated. In the model, this can be viewed as dilation without pressure as shown in Figure 2.6.9. This results in a reduction in ultimate dilation pressure and consequently in reduced pullout strength. Shrinkage is a problem when high water:cement ratio cements are used or if grouting is performed in high temperature, low humidity environments.

Cable strain and bond strength

As a cable is loaded axially it experiences elastic axial tensile strain as well as an associated reduction in effective diameter. When a cable exceeds its yield strength (20 tonnes for a 15.2 mm cable) the rate of strain increases. In a heavily loaded cable, this plastic radial strain can be significant and can cause separation at the cable/grout interface, reducing the maximum available bond strength.

Inadequate quality of strand and bond strength

If the cablebolt strand is poorly fabricated, there may be inadequate contact between the outer helical wires and the inner king wire (Bawden et al., 1995). As the cable is loaded, these gaps will close, reducing the diameter of the cable strand, resulting in interface separation or reduced interface pressure. This and the previous two strength reducing mechanisms are modelled in exactly the same way - by an initial increment of dilation without an associated pressure increase as shown in Figure 2.6.9.

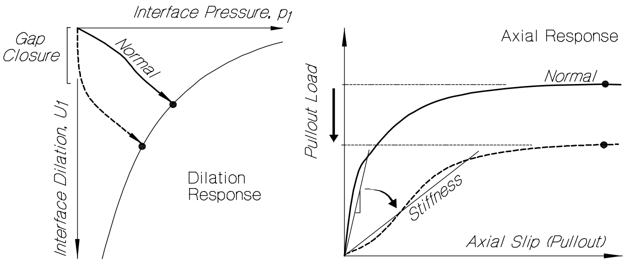


Figure 2.6.9: Dilation without pressure, caused by grout shrinkage or strand contraction

Borehole Diameter and Bond Strength

Borehole diameter has an effect on the overall system stiffness. This effect is, however, relatively minimal over the range of hole sizes currently in use for cablebolting. While smaller boreholes yield slightly higher bond strengths under ideal conditions, grouting difficulties arise at smaller diameters which negate this effect. The effect on ultimate bond strength (pullout resistance after approx. 40 mm of displacement in this case) is modelled in Figure 2.6.10 for two example combinations of grout quality and rock stiffness.

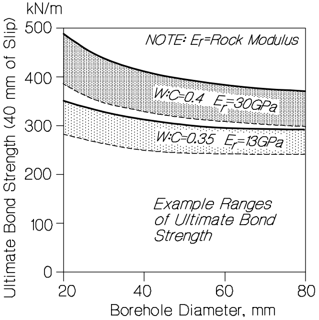


Figure 2.6.10: Effect of borehole diameter

Grout Strength and Bond Strength

As dilation of the cable/grout interface progresses during axial slip, the dilation pressure increases. As the grout ridges ride over the cable wires, the interface stresses become focussed within a decreasing contact area. Eventually, the grout ridges crush and further dilation is prevented. This point marks the theoretical limit of bond strength. This limit is stiffness dependent and can be expressed as a dilation limit curve in the bond strength model. The shape of this line is back-calculated from analysis of over 140 test results (Diederichs et al , 1993).

Figure 2.6.11 shows different dilation limits for different grout strengths. These strengths are related to grout water/cement ratio as shown in Figure 2.5.5. Clearly, increased strength results in increased maximum dilation pressure which in turn yields greater bond strength. Note, however, the practical difficulties (Section 2.5.4 and 2.5.6) inherent in the placement of thick grouts (*W:C* < 0.35).

Grout Stiffness and Bond Strength

Water:cement ratio also controls grout stiffness (Figure 2.5.6), which in turn affects the radial stiffness of the system (Slope *M* in Figure 2.6.6). Stiffer grouts lead to an increase in dilation pressure for a given radial displacement. This leads to an increase in ultimate bond strength as shown in Figure 2.6.11.

Note that the example pullout response curves in Figure 2 6.11 are for a specific borehole stiffness (equivalent to a moderately stiff limestone with modulus E_{rock}= 13 GPa) and that actual response will be dependent on the rock modulus (or pipe stiffness in the lab) as described in the next section.

Grout Quality (Water:Cement Ratio) and Bond Strength

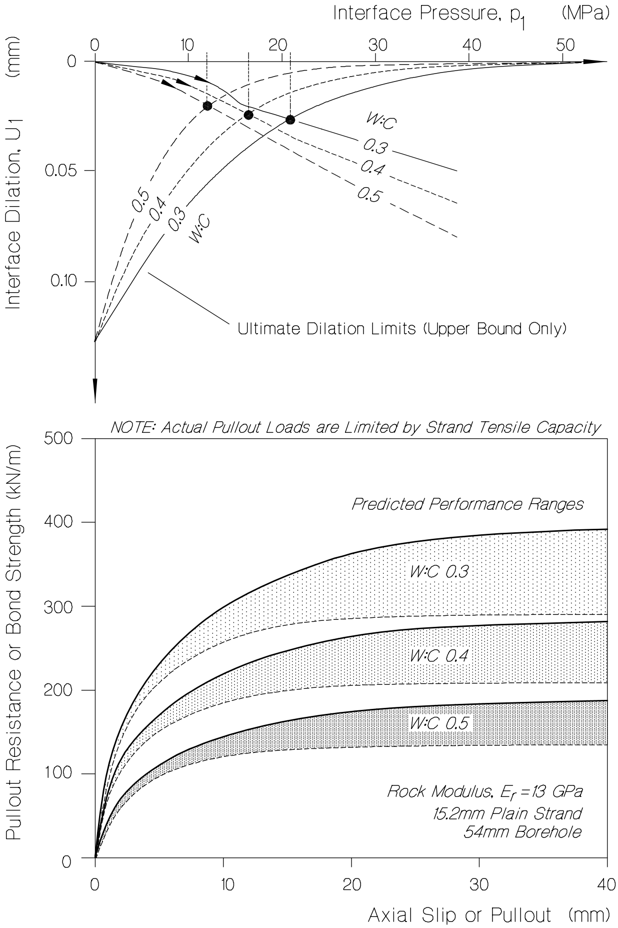


Figure 2.6.11: Influence of grout strength and stiffness as determined by water:cement ratio (after Diederichs et al., 1993)

Borehole Stiffness and Bond Strength

The overall radial stiffness of the system is defined by both the grout stiffness and the rock stiffness. The slope *M* of the model decreases with decreasing rock stiffness as shown in Figure 2.6.12. It should be noted that rock stiffness has a dramatic influence on bond strength when the modulus of the rock surrounding the borehole is close to or less than the modulus of the grout. In very stiff rocks, the grout modulus and strength are the critical parameters determining bond strength.

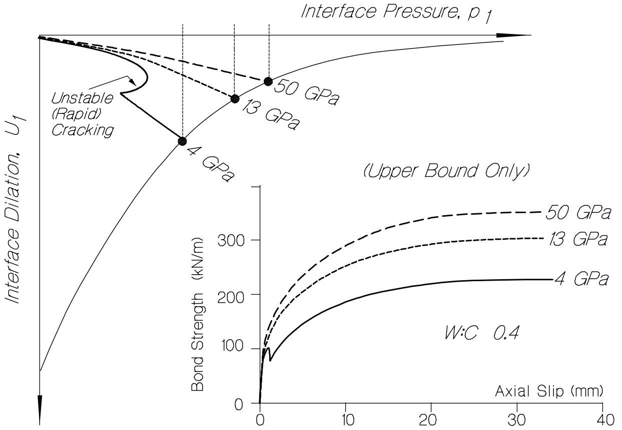


Figure 2.6.12: Influence of rock modulus (borehole stiffness) on system stiffness, interface dilation and bond strength (after Diederichs et al., 1993)

It is the stiffness of the borehole rock which is important to consider. Joints and fractures around the borehole can influence this stiffness. If the average fracture spacing is more than 5 times the borehole diameter or if the rockmass is moderately stressed, then it can be assumed that the intact rock modulus dominates the cable behaviour. For higher fracture densities or in low stress environments, it may be appropriate to use the rockmass modulus estimated from rockmass classification schemes. When the intact rock modulus is to be used, it is prudent to use 50-70% of the laboratory stiffness to account for borehole damage.

Rock stiffness can change during the service life of a cablebolt. As the rockmass is overstressed, creating more fractures or as existing fractures open, the effective rock stiffness can decrease, causing a drop in cable bond strength. This effect has been observed in the field (Hyett et al., 1992; MacSporran et al., 1992).

Rock Stiffness, Grout Quality and Bond Strength

Hyett et al. (1992) give the following relationship between the borehole parameters and the specifications for a laboratory pipe test (pullout):

$$\frac{2E_R}{(1+\nu_R)d_{BH}} = \frac{2E_P(d_O{}^2 - d_I{}^2)}{d_I(1+\nu_P)\{(1-2\nu_P)d_I^2 + d_O{}^2\}}$$

where:

E_R = Rock modulus

ν_R = Rock Poisson's Ratio

d_{BH} = Borehole diameter

E_P = Test pipe material modulus

ν_P = Test pipe material Poisson's Ratio

d_I = Inside diameter of Pipe

d_o = Outside diameter of Pipe

In Figure 2.6.13, ***ultimate bond strength*** is taken as bond strength (load/embedment length) at 40 mm of axial slip. Compare with Figure 2.6 3.

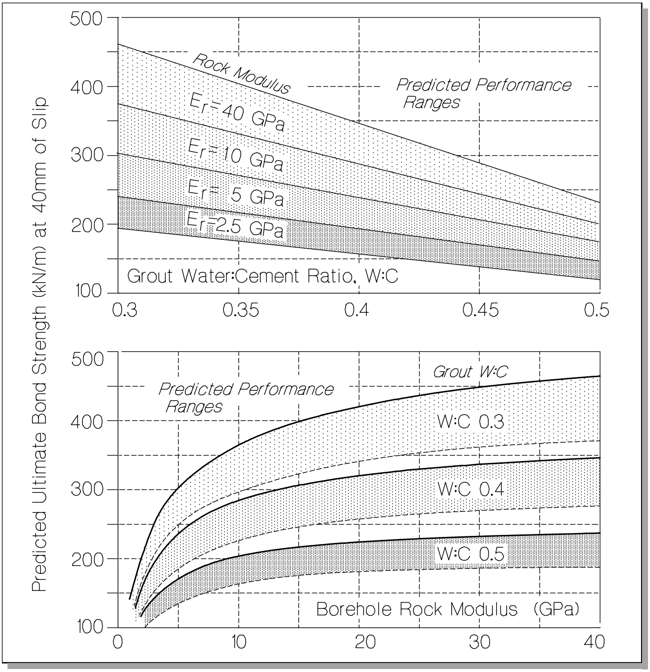


Figure 2.6.13: Ultimate bond strength as a function of grout quality and rock modulus; Note that actual system capacity may be limited by strand tensile strength

The model predictions in Figure 2.6.13 can be used with considerable confidence due to the fact that they are the result of a calibration process (Diederichs et al., 1993) incorporating over 140 pull test results spanning a wide range of key parameters (grout, confining medium, hole size, etc.).

As shown below, the upper and lower bounds (non-rotating and rotating, respectively) accurately reflect the range of performance encountered in testing and in field loading situations. The reader is referred to Hyett et al. (1992) and Goris (1990) for a comprehensive suite of laboratory and field test results.

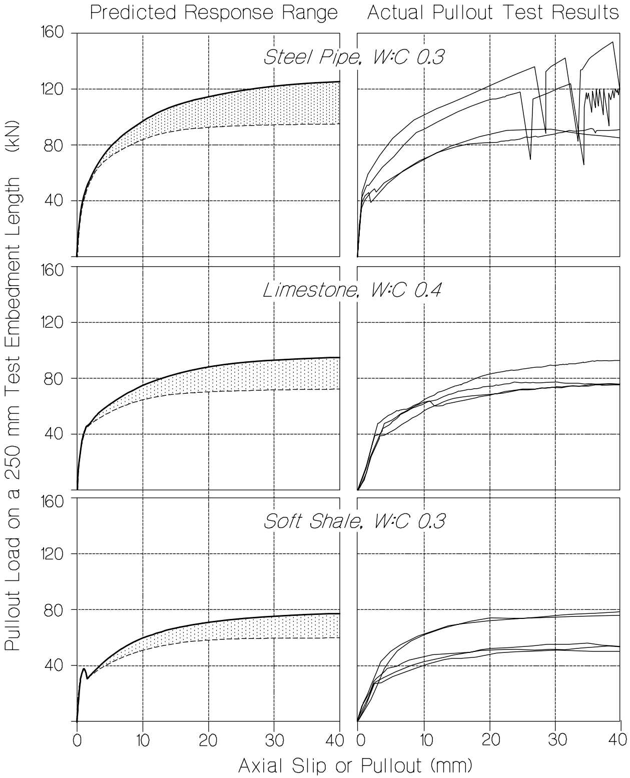
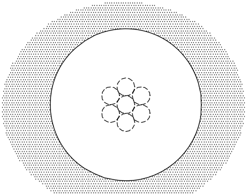


Figure 2.6.14: Comparison of model predictions (after Diederichs et al., 1993) with selected laboratory and field tests (after Hyett et al., 1992)

Stress Change and Bond Strength

Recent research (Kaiser et al., 1992; Diederichs et al , 1993; Maloney et al , 1992, Pieterse, 1993; MacSporran et al., 1992; Bawden, 1994; Hyett et al., 1995) has shown conclusively that stress change in the surrounding rockmass after the installation of a cablebolt can profoundly affect the bond strength of the cable. In short, stress increases cause an increase in bond strength while stress decreases can reduce the strength. In the latter case, it is possible in an initially stressed soft rockmass, that cable bond strength can be reduced to nil. Many cablebolt failures observed by the authors (Kaiser et al., 1992; Hutchinson and Diederichs, 1993) can be attributed to stress decrease across the installed cables.

To understand this mechanism, it is necessary to consider the sequence of cable installation. First a borehole is drilled in stressed rock. The borehole deforms inward as it is drilled. After creation, the borehole wall is radially unstressed. The cable is then inserted and grouted. The grout cylinder at this time is also unstressed but is in full contact with the cable and the rock (it is assumed that the cable tendon is of standard quality and there is no grout shrinkage).



Empty hole ready for Grouting
(Radial Boundary Stress =0)

During the service life of the cable in a mining environment, new excavations are created in the vicinity of the cable causing a change in the local stress field. The borehole responds with additional radial displacement, in general, contracting under increased stress or expanding under stress reduction. This time the grout is also influenced by these deformations and in turn, the conditions at the cable/grout interface are altered.

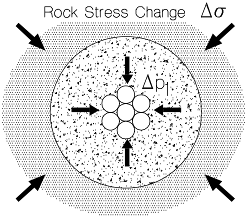


Figure 2.6.15: Stress change and interface pressure

Returning to the model, a stress increase in the rockmass causes a contraction of the borehole and a compression of the grout. If the cable is assumed to be comparatively rigid, this effect is modelled as an increase in cable/grout interface pressure without any dilation as in Figure 2.6.16 a). The result is an increase in maximum interface pressure (after dilation) for a given system stiffness and therefore an increase in ultimate bond strength.

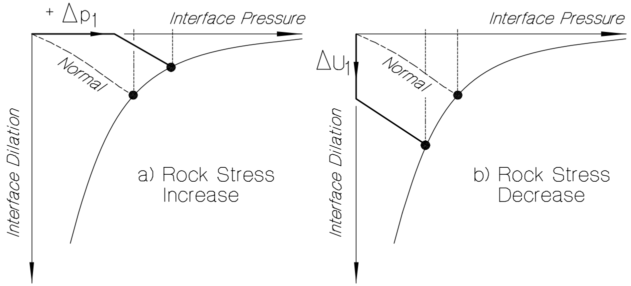


Figure 2.6.16: Conceptual influence of stress increase (a) and stress decrease (b) on bond strength of grouted plain strand cables (after Kaiser et al., 1992)

A decrease in stress in the surrounding rockmass results in an expansion of the borehole as the rock relaxes. As a result, the unstressed grout becomes separated from the borehole and/or from the cable. This separation must be closed before dilation pressure can be generated. This effect is modelled by a dilation without pressure increase as shown in Figure 2.6.16 b). If dilation pressure has been generated through previous cable slip, rock relaxation (stress decrease) will result in an instantaneous reduction in the interface pressure and a reduction in bond strength. The effect of stress change on an example borehole configuration is modelled by CABLEBOND (Diederichs et al., 1992) in Figure 2.6.17.

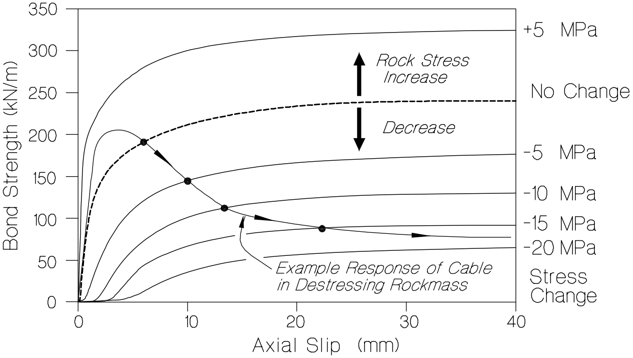


Figure 2.6.17: Example of the influence of stress change on predicted pullout load W:C = 0.4, Rock Modulus = 13 GPa (after Diederichs et al., 1993)

It should be noted that field research (Maloney et al. 1992) and recent independent work and bond strength modelling (Hyett et al., 1995) has confirmed this predicted behaviour.

The relationship between stress change in the rock mass and bond strength is also dependent on the relative stiffnesses of the rock and the grout as shown in Figure 2.6.18 at right. The grout modulus can be obtained from the water:cement ratio using Figure 2.5.6.

Example relationships for ultimate bond strength (lower bound bond strength after 20-40 mm of slip) for different rock moduli are given in Figure 2.6.19.

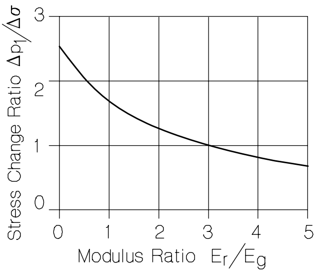


Figure 2.6.18: Influence of modulus ratio on stress change @ interface

In fractured rockmasses the rock modulus can be stress dependent. In general, rock stiffness will tend to decrease with decreasing stress. Softer rocks are more sensitive to stress change. The combined result of rock relaxation, therefore will be greater (a greater drop in bond strength) than is shown in Figure 2.6.19.

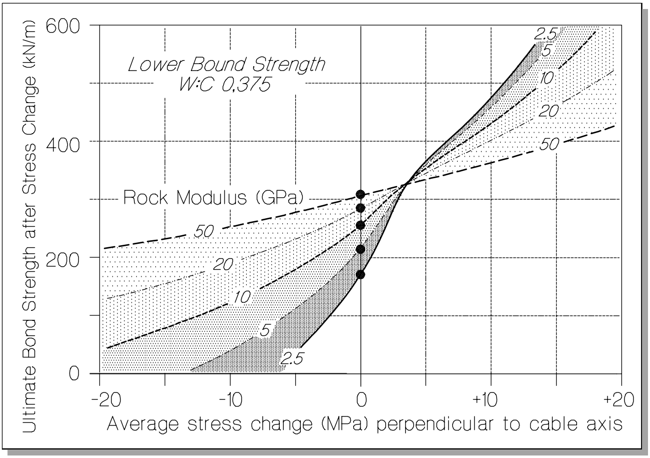
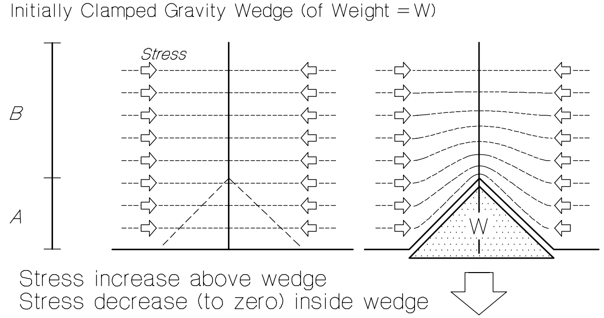
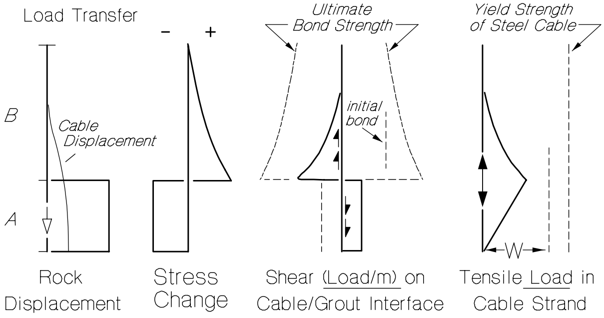


Figure 2.6.19: Effect of stress change and borehole confinement (rock modulus) on bond strength for a grout of W:C = 0.375.

Load Transfer and Stress Change



It has been shown that a modest stress reduction in the surrounding rockmass can have a significant and detrimental impact on cablebolt bond strength. This bond strength reduction translates into an overall capacity reduction by reducing the load transfer capabilities of the cable grout interface. For example, consider an initially clamped wedge as shown here. If the wedge is pinched out or becomes dislodged, the stresses in the wedge will drop potentially to zero resulting in a stress reduction experienced by the cablebolt as shown. If this stress drop reduces the interface bond strength below the ***critical bond strength*** (for the particular case being considered) the weight of the wedge cannot be transferred to the cable and carried up to the surrounding rockmass as intended and the wedge will fall. Note that the initial stresses have been diverted over the top of the wedge, resulting in a normal stress and bond strength increase in the anchor section.



Stress change and pullout stiffness

In addition to the ultimate bond strength, the initial pullout stiffness of a plain strand cablebolt is also affected by stress change in the rockmass (Hyett et al., 1995). A stress decrease in the rock causes a significant reduction in stiffness during the first 5-20 millimetres of slip as the induced gap between the grout and the cable is closed. Similarly, the system is made stiffer by an increase in average stress in the rock. The interface gains immediate frictional strength (without slip and dilation) due to increased normal pressure as the borehole contracts. These effects are clearly visible in the example shown in Figure 2.6.17. Note again that the magnitude of these changes is dependent on the relative stiffnesses of the grout and the local rock.

Calculation of borehole stress change

Note that the stress change referred to in this section is the average change in stress, $(\Delta\sigma_{x}+\Delta\sigma_{min})/2$, acting perpendicular to the borehole, using the stress state at the time of installation as a datum. The change in stress along the axis, $\Delta\sigma_{xial}$, of the borehole has a minor (and counteracting) effect. This effect is ignored in the simulations presented here. This simplification results in a maximum 3-5% error in bond strength calculation for hard rock applications.

Examples of Stress Change and Cable Performance

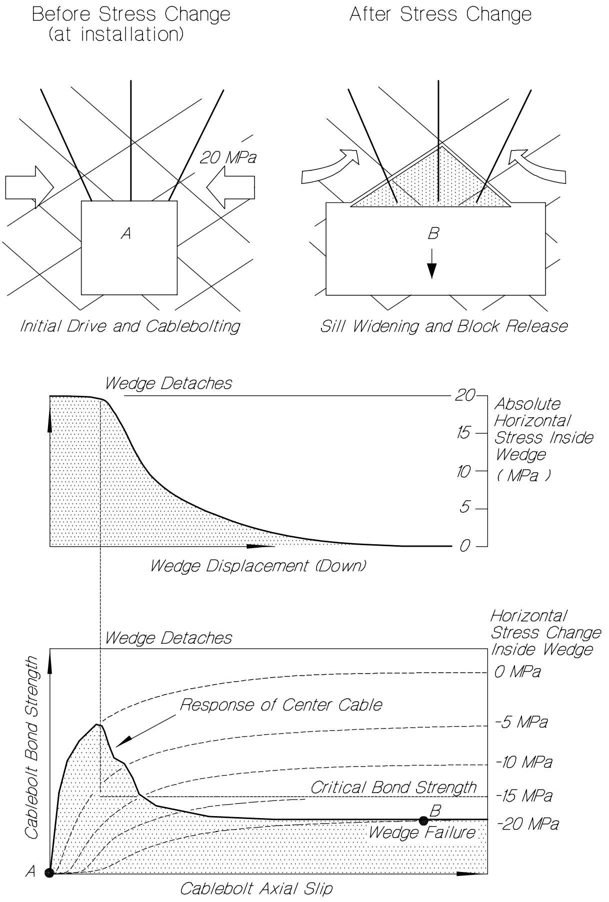
There are many circumstances which would result in a decrease in rock stress and in cablebolt bond strength reduction. Stress reductions in excess of 40 MPa have been measured across installed cablebolts (Maloney and Kaiser, 1991; Maloney et al., 1992). Even in very stiff rocks, such a reduction can be serious.

Since destressed and unconfined (i.e. unclamped) rockmasses are the most likely to require support, it is particularly alarming that the plain strand cablebolt is at risk of losing its bond strength and overall capacity at the very time when it is needed most.

It may be necessary to consider alternative cablebolt geometries such as birdcaged or bulbed strand which tend to show less sensitivity to stress change. The influence of stress change can also be reduced through sequencing. By properly timing the installation of cables it may be possible to reduce the stress decreases experienced after installation. It is also prudent to attach plates and surface anchorage to cablebolts where access permits. The plate and anchor make the "pick-up" section of the cable comparatively insensitive to stress change.

The examples given in Figures 2.6.20 to 2.6.22 are commonly encountered in mining. It is important to recognize these situations and to design accordingly.

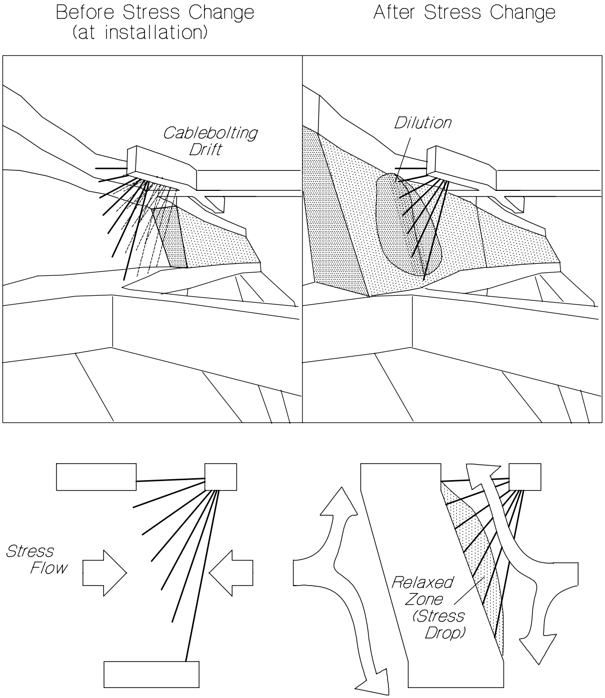
Stress Change Example - Wedge



Note: Bond failure under gravity loading occurs when actual bond strength drops below *Critical Bond Strength*

Figure 2.6.20: Wedge detachment, stress drop and bond strength loss

Stress Change Example - Hangingwall Stress Shadow



Example: Cablebolts installed in elevated stress front in advance of stoping. After mine-by, stresses across cables drop, accompanied by a significant reduction in bond strength resulting in hangingwall dilution as shown.

CABLEBOND (Diederichs et al.,1992) results at right show up to a 50% reduction in bond strength in lower hangingwall for cable array shown above.

Note that stress increases around cablebolt drift cause bond strength increases in anchor sections.

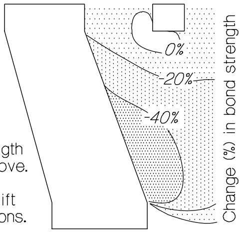
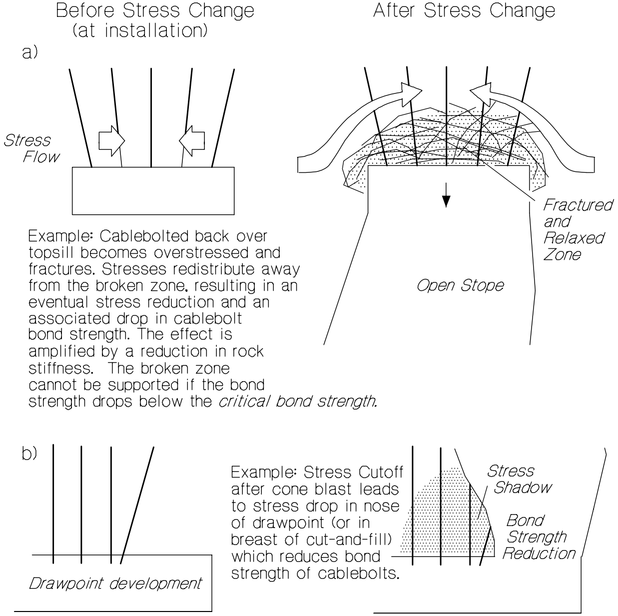


Figure 2.6.21: Mine-by stress shadowing and bond strength reduction

Stress Change Examples - Fracture Zone; Re-entrant Corners



Example: Cablebolted back over topsill becomes overstressed and fractures. Stresses redistribute away from the broken zone, resulting in an eventual stress reduction and an associated drop in cablebolt bond strength. The effect is amplified by a reduction in rock stiffness. The broken zone cannot be supported if the bond strength drops below the *critical bond strength.*

Example: Stress Cutoff after cone blast leads to stress drop in nose of drawpoint (or in breast of cut-and-fill) which reduces bond strength of cablebolts.

Figure 2.6.22: a) Stress fracturing, stress and stiffness relaxation and bond reduction
b) Creation of re-entrant corners (noses) and stress relaxation

Summary of Remedial Measures - Stress Change

Stress change occurs in every phase of mining. When potentially detrimental stress reductions are identified, the following options are available:

- Plate cables (with barrel and wedge anchor). This surface anchorage is not sensitive to stress change. Ensure that some length of cable (upper end) is reasonably unaffected by stress reduction. Otherwise pullout may still be a risk.
- Use modified strand cablebolts (Sections 2.6.3 and 2.9). These flared strand bolts are much less sensitive to stress change.
- Adjust sequencing to avoid installing cables in high stress zones (e.g. ahead of an advancing stope front) which will be subject to future stress reduction.

2.6.3 Modified Geometry Strand

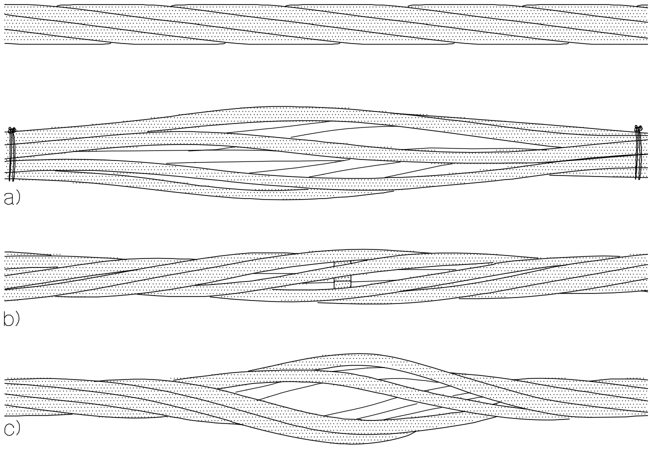


Figure 2.6.23: Commercially available versions of modified geometry strand (Canada):
a) Birdcaged cable b) Nutcaged cable c) Bulbed strand

While the plain strand cablebolt has seen many years of successful application in civil engineering construction and in mining, the acute sensitivity of the plain strand to imperfect quality control, stress changes and rock modulus reduction after placement creates difficulties in mining where these problems are common.

For this reason, various modified geometry cablebolts (modifications of the plain strand) have been developed over the years (summarized in Windsor, 1992) which possess reduced sensitivities to these elements and which in general possess enhanced bond strength and stiffness characteristics. Some of the more recent developments are detailed in Section 2.9.

In general modified cable strands possess enhanced dilational properties. That is, they serve to greatly increase the geometric mismatch between the cable and the grout, generating increased pullout resistance. Shear through the grout takes a larger part in the overall failure mechanism (Bawden and Hyett, 1994) resulting in higher bond strength and shorter critical embedment lengths (consistently less than 0.3 m required to break the strand during pullout).

Modified Strand - Flared Geometries

While some attempts have been made to improve the interface strength through the attachment of swaged (pressed) anchors (Schmuck, 1979; Goris, 1990), internal double-acting barrel and wedge anchors attached to the cable (Matthews et al., 1983), and through the use of single-acting cable wedges (Gendron et al., 1992) and internal ferrules similar to the nutcage (Windsor, 1990), the flared geometries illustrated in Figure 2.6.23 are the most common, commercially available configurations of modified strand (in North America and Australia).

The birdcage (Hutchins et al., 1990) is formed by unravelling plain strand and then rewinding the wires slightly out of phase with each other, creating a cage-like structure which is held together by wire ties. The nutcage (Hyett et al., 1993) is formed by unwinding plain strand, sliding a nut (or series of nuts) onto the king wire and then rewinding the strand, preserving as much as possible the original lay but creating a rigid bulb enclosing the nut. Finally the bulbed strand (Garford Pty. Ltd., 1990; Bawden and Hyett, 1994) is formed by clamping a section of plain strand between two hydraulic grips and crimping the intervening section to create a deformed bulb. This process, if performed correctly does not damage the strand and has the advantage of preserving the tight wind of the rest of the cable.

Modified strand will usually cost more than plain strand (since the plain strand is the raw material for their manufacture). Volume production, increased availability and reduced costs, combined with the increased performance where bond stiffness and bond strength are critical, have already made these products competitive alternatives to plain strand.

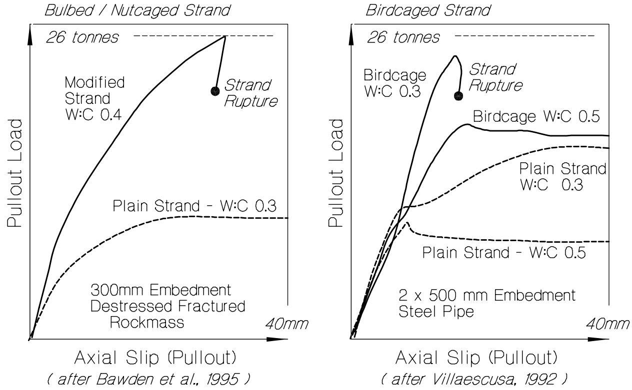
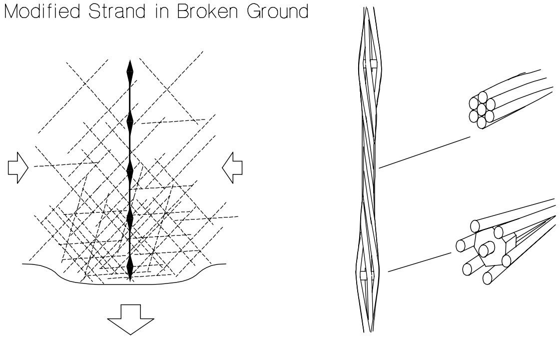
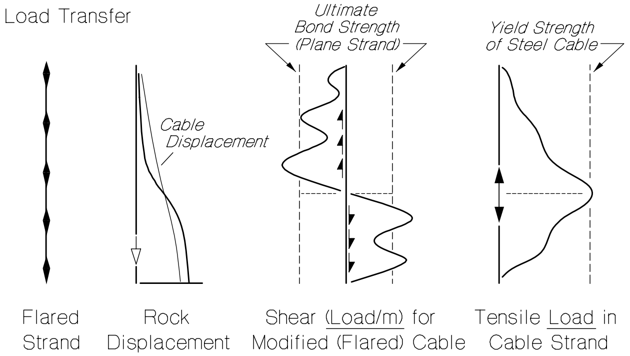


Figure 2.6.24: Examples of increased bond performance of modified strand (pullout)

Load Transfer and Modified Geometry Cablebolts



The flared elements of the modified strand serve as concentrated dilation and load transfer sites along the cablebolt. It has been shown that a single bulbed, caged or nutcaged element (node) is capable of generating full tensile capacity in the strand before bond failure occurs (Bawden and Hyett, 1994). Note that the modified strands are, in general, considerably stiffer in pullout than plain strand, generating and transfering loads over much smaller degrees of cable-grout slip. This property is desirable to reinforce fractured ground and to limit displacements.



2.6.4 Debonding

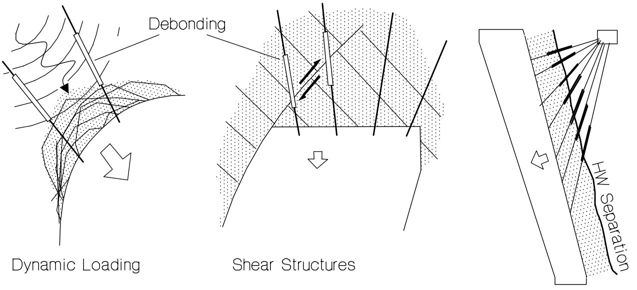
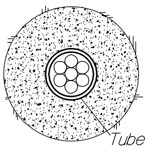


Figure 2.6.25: Example situations where debonded strand segments are desirable

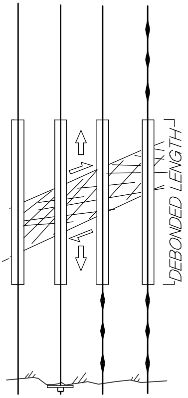


In highly stressed fractured ground, across mobile shear or delamination structures or in areas with the risk of dynamic loading from seismic activity, it is sometimes desirable to reduce the stiffness of the cablebolt system over a finite central length of strand while maintaining bond strength at the ends (Figure 2.6 25).

This is accomplished through the use of debonding. For plain strand sections this can be accomplished with varying degrees of efficiency through the use of paint, grease or plastic tubing. The latter is recommended as the more predictable method. Figure 2.6.26 shows the expected elastic and inelastic stretch (relative displacement) along varying lengths of debond.

Where debonding is used in fractured ground it may be advisable to plate the exposed end of the cable if access is permitted. In remotely installed hangingwall fans, it may be necessary to use specialized cables with a central portion of debonded plain strand between two ends of modified geometry or a modified loading section and plain strand anchor.

Such specialized strand (at right) can now be manufactured by cable suppliers on special order.



Debonded Length

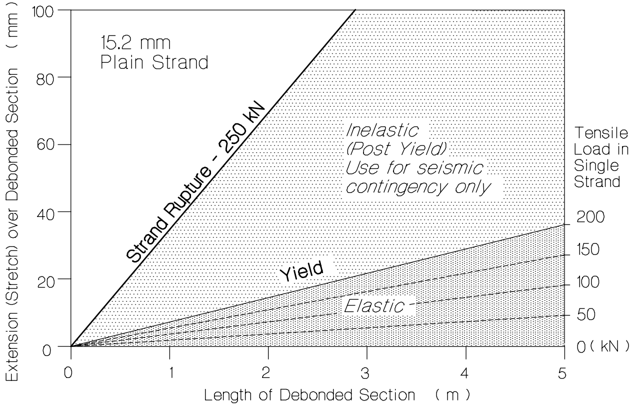


Figure 2.6.26: Supplemental displacement provided by debonded strand sections

The overall stiffness and therefore the total relative displacement in the cable will include the response of both of the embedded sections as well as the debonded length. The increase on displacement capacity (ductility) for a birdcage stand is shown in the example in Figure 2 6.27. Note that under dynamic loading, plastic cable strain may localize and reduce the available displacement shown here.

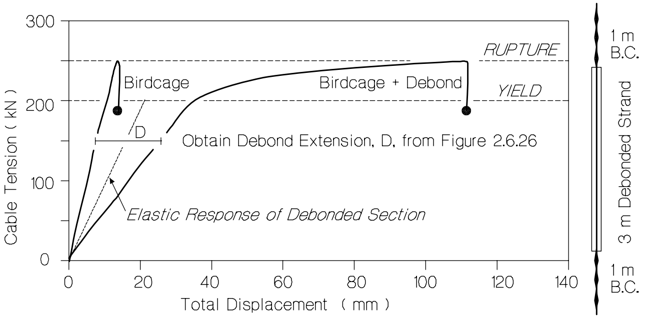
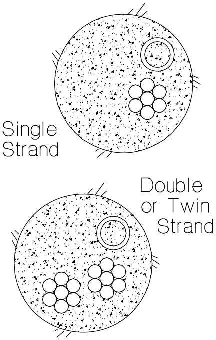


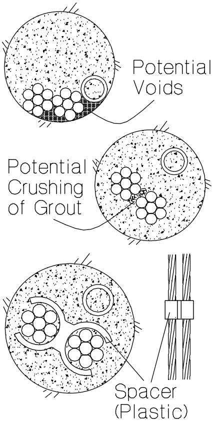
Figure 2.6.27: Stiffness reduction and increase in displacement capacity of a birdcage (B.C.) strand with 3m of debonding between two 1m embedded lengths

2.6.5 Double and Multiple Strand



The bond strengths and tensile capacities given in this section generally refer to single strand cablebolts; that is cablebolts consisting of a single grouted strand per borehole. It is common in underground engineering to employ the use of double or twin strand (two strands per borehole) and, in open pit and civil anchorage applications, to install more than two strands in a cluster. The primary motivation for doing so is the need to increase the tensile capacity of the steel strand (single strand capacity × number of strand per hole) for example, when strand rupture has been observed. It should not be assumed that the bond strength (load per unit length of twin plain strand) will be twice that of single strand.

While double strand cables can increase the bond strength (load per unit length of double strand cable) up to 100% (Goris, 1990) in laboratory testing, it is unlikely that double strand bolts can compensate for poor bond strength in the field to such a large degree. The factors which contribute to observed bond performance problems (grout voids, stress change, confinement, quality control) of single strand are likely to be exacerbated by the use of double strand. In particular, the interference between strands can lead to:



- Grout void formation between the strands during placement (see Section 2.12).
- Grout crushing between cables during pullout (Hutchinson, 1992), resulting in internal relaxation and reduced bond strength.

If spacers are employed (at right) at 1 to 2 m intervals along the double strand, then it is possible to rely on increased bond strength (up to 100%) in design calculations. Spacers maintain separation between multiple strands and allow grout to penetrate between the strands and the borehole wall. Full encapsulation is required for optimum bond strength.

Bond strength is unlikely to be a critical concern when multiple modified strand are employed with good quality control (Bawden and Hyett, pers. communication).

Double Strand Cablebolts

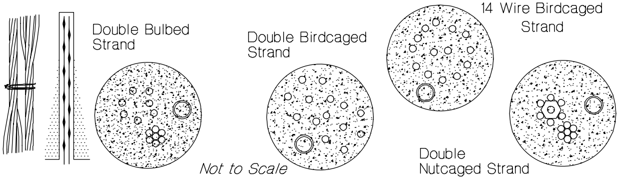


Figure 2.6.28: Modified geometry double or twin strand cablebolt configurations currently in use in underground and open pit mines

If it can be assumed that the bond strength is optimized in the multiple strand configuration, then the tensile capacity and the system stiffness per borehole is increased by a factor corresponding to the number of strand in the hole (Anderson and Grebenc, 1995; Villaescusa et al., 1992). Installation, experience and quality control constraints currently limit underground installations to double strand. Figure 2.6.29 illustrates the relationship between double strand usage and cablebolt spacing requirements or maximum system capacity for ***gravity loading.*** In essence, if it can be assumed that:

- the ground is competent enough to maintain surface integrity between neighbouring cablebolt holes,
- the bond strength has been optimized,
- the loading conditions remain the same;

it is possible reduce the number of cablebolt holes by as much as one half when double strand cablebolts are employed, significantly reducing installation costs. **Always exercise caution when expanding existing bolt patterns**.

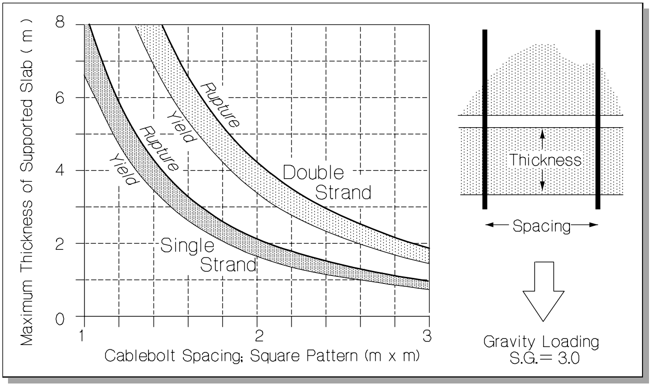


Figure 2.6.29: Influence of double strand use on cablebolt system capacity

2.6.6 Grout and Rock Shear Strength

In Figure 2.6.1, a number of possible shear failure modes were introduced for the cablebolt/rock system. The forgoing discussion has focussed on the cable/grout interface. For plain strand cablebolts in hard rock, this interface is critical. In weaker rocks or when modified strand is used, shearing may occur through the grout itself, at the grout/rock interface or through the surrounding rockmass, allowing the cablebolt and grout column to pull out of the rock mass. It is important in these situations to consider the available strength of these interfaces.

Grout Shear

The simplest and most conservative approach for assessing the shear strength of the grout column is given by Carter (1995) and Littlejohn and Bruce (1975). The strength per unit area is given as:

$$\tau_{G(ultimate)} = \frac{UCS_G}{\alpha}$$

where UCS_G is the grout unconfined compressive strength from Figure 2.5.5 and α is a factor which Littlejohn and Bruce (1975) specify as 10 for civil engineering applications. For single strand cablebolts or double strand cablebolts with spacers in mining applications, the authors of this handbook recommend a value of α ranging from 5 in very stiff (hard) rock to 10 in soft or fractured rock in order to account for grout confinement and frictional strength.

Normalized grout strength, T (allowable load/unit length of grout column) is;

$$T = \pi D \tau_{G(ult)} Q$$

where $Q \leq 1$ is a quality control factor which is equal to 1 for perfect quality control (Gerdeen et al , 1977). D is the diameter (in metres) of the relevant shear interface. For plain strand use the conservative value corresponding to the cable diameter (0.016m). Use twice the strand diameter for double strand cablebolts. Without considering quality control during grouting, a plain strand 0.35 $W{:}C$ grout (UCS = 60 MPa) in moderately stiff rock (α=6) would have a limiting grout capacity of $\pi(0.016)(60/6)(1)=0.5$MN/m. If this is compared to cable/grout bond strength values from Figure 2.6.13 of 0.3 to 0.4 MN/m, it can be seen that the cable/grout bond is critical.

For bulbed, birdcage or other modified strand, use the outer diameter of the deformed element (for example, the diameter of the bulb = 0.025m to 0.035m) as D in the above equation. For the example above, the grout capacity increases to 0.8 MN/m for the smaller bulb.

Grout/Rock Interface and Rock Shear Strength

After checking for grout shear capacity, check the shear capacity at the grout/rock interface, using the rock properties (Kenney, 1977; Goodman, 1980):

$$\tau_{R(ultimate)} = \frac{UCS_R}{\alpha}\left(\frac{\beta}{2\tan\left(45+\phi/2\right)}\right)$$

where UCS_R is the unconfined compressive strength of the rockmass, α varies from 1 for fresh rock to 2 for moderately weathered rock to 10 to completely weathered rock, and β varies from 0.3 to 0.9 for smooth to roughly drilled holes respectively. ϕ is the friction angle of the rock taken from 15° for weak clay rich or schistose rocks to 35° for competent granular or crystalline rocks (Barton, 1974). UCS_R is taken as;

$$UCS_R = \sqrt{s}(UCS_L) \quad \text{where} \quad s = e^{\left(\frac{RMR-105}{9}\right)}$$

and where UCS_L is the intact (laboratory) rock strength which can be estimated from Table 2.14.1 and *RMR* is the 1989 Rock Mass Rating from Section 2.14.4.

This shear strength value is again used in the equation:

$$T = \pi D \tau_R$$

to obtain an estimate of the shear capacity of a unit length of grouted borehole (MN/m) where *D* is now the diameter of the borehole.

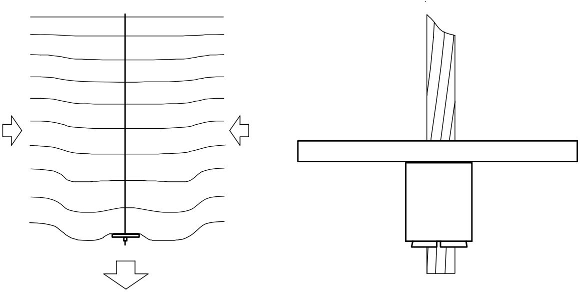
Littlejohn and Bruce (1976) recommend that the values of *T* calculated above should be divided by a safety factor of at least 2 to account for the simplifications and uncertainties involved.

It should be noted that grout/rock or rock shear is seldom the observed failure mode in hard rock mining applications where single or double strand cablebolts are used. When a larger number of cables are clustered in a single anchor as in open pit or civil construction applications, or in especially poor rock conditions, it is critical that the rock strength and associated pullout resistance be evaluated.

The calculations given here are grossly oversimplified but are normally adequate for design purposes in mining. The assumption of uniform load transfer leading to the unit shear capacity concept (MN/m) is a key point here. Many researchers including Farmer (1975), Aydan and Kawamoto (1992), St. John and Van Dillen (1983) and others have developed more rigorous approaches to the evaluation of annular shear behaviour for grouted tendons. Littlejohn and Bruce (1976) and Barley (1988) have also tabulated a variety of pullout test data.

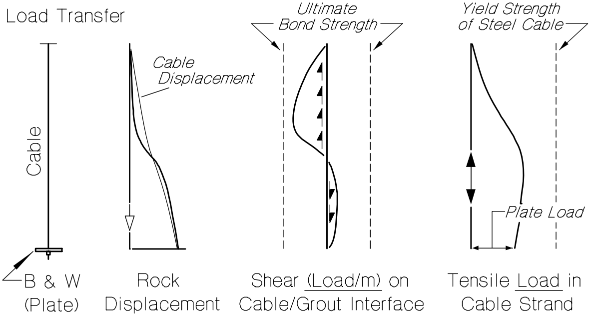
2.6.7 Load Transfer and Surface Anchorage

Plating (Barrel & Wedge Anchor)



Plates can be attached to the exposed ends of cablebolts using a barrel and wedge anchorage system. It is always beneficial to attach plates where access, timing and economics permit. This is particularly true for overhead installations and for plain strand cables. The direct rock-cable connection provided by plates and anchors reduces the dependence on bond strength. Bond strength near the face can be reduced by grout voids and due to the ungrouted length at the collar (breather tube installations). In addition, stress change and modulus reduction as the rock deforms will reduce bond strength where and when it is needed the most. If plates are attached, load is generated immediately and if the anchor is designed with a higher capacity then the strand, full tensile capacity of the cablebolt will be made available. The cable/plate/anchor system still relies on a functional anchor length up the hole to complete the load transfer process. For this reason, plates cannot be used as a substitute for quality control and good design.

Load Transfer



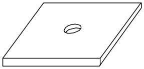
2.7 Surface Anchorage and Retention

It is often necessary, and when access permits, usually desirable, to complete the cable system by attaching plates to the end to provide a positive connection to the face. Screen or straps can complement the plate to retain small blocks between the cablebolts or between any intermediate rock bolts or rebars. Note that screen is usually not effective when anchored at points equivalent to normal cable spacings (e.g. 2m x 2m) and may require rock bolting between cables.

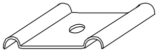
2.7.1 Plates

Plates can be used simply to attach screen or straps to the cable system. This is valid where the integrity of the face is such that little loading will develop near the exposed end of the cables. In this case, thinner plates may be used. Thinner plates can be deformed during manufacture to increase their retention capabilities or to increase the initial displacement in high-stress or dynamic conditions.

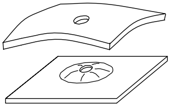
Where the plate is expected to form an integral part of the cablebolt load transfer system, as in highly fractured ground, thicker plates must be used which have pull-through capacities equivalent to the tensile strength of the cablebolt (200-250 kN).



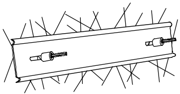
Standard Plate



Increased Retention



Reduced Initial Stiffness



Intercable Retention

Table 2.7.1: Typical dimensions and capacities of bolt faceplates (after Douglas and Arthur, 1983).

Working Load of Bolt (kN)	*Size of Plate (Length or Diameter) (mm)*	*Thickness (mm)*
80	125 to 150	7
150	150 to 200	10
300	**200 to 250 ***	**12 ***

* *Recommended for cablebolting in fractured ground*

Note that in order to prevent punch-through of the barrel and wedge anchor fitting, the hole in the plate should be only slightly (1 to 2 mm) larger than the diameter of the cable strand (15.2 mm). An exception is when the cable is angled with respect to the face. The edge of the hole must not pinch into the cable and therefore, must be larger than this limit. A bevelled or cupped washer should then be used to provide full bearing on the plate and to provide a reduced diameter to receive the anchor. If rounded barrels are to be used directly on a plate surface, the plates should have matching spherical insets to receive them.

2.7.2 Surface Anchorage - Barrel and Wedge

Cablebolts cannot be threaded to receive a locking nut for surface fixture placement and anchorage. Some suppliers have marketed cable elements with threaded end caps, pressed or welded onto the end of the strand, for affixing plates or straps. The conventional method, however, for surface fixture attachment and tensioning involves the use of a concentric barrel and wedge assembly which is activated by tension in the steel cable applied by a specialized jack.

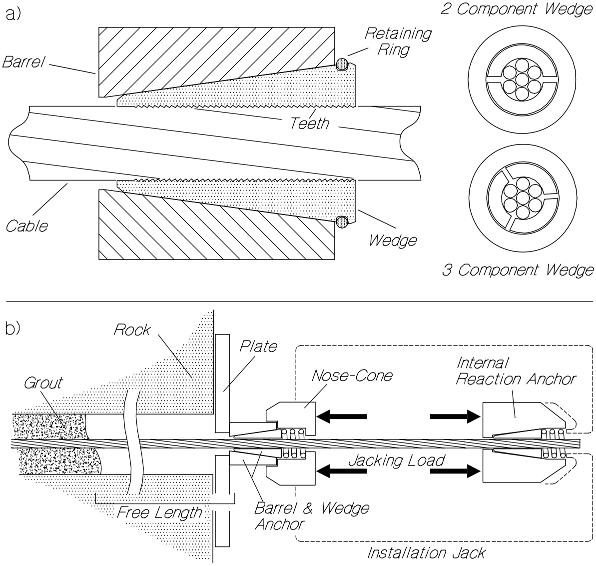


Figure 2.7.1: a) Typical Barrel and Wedge (B&W) anchor assembly for cablebolts
b) Schematic plating/tensioning configuration (after Thompson,1992)

A typical barrel and wedge anchor assembly is shown above. The conical wedge assembly can be composed of two or three components held together by a spring wire or rubber ring. The inside surfaces of the wedge are serrated with concentric rings of teeth to grip the cable. The wedges press against the barrel as they are driven inward by the jacking unit or pulled in by the relaxation of the tensioned cable. As long as there is residual load (tension) in the cable, the anchor is kept in place as the cable continues to pull against the wedges. The outer barrel completes the load transfer to the plate which bears against the rock.

Barrel and Wedge Anchor Specifications

There are currently no standards for barrel and wedge anchors in mining. For effective anchor performance, however, a few general manufacturing and inspection guidelines can be specified:

- The optimum wedge and barrel length is approximately 2.5 to 3.5 times the diameter of the cable (i.e. 35 to 55 mm).

- The wedges and barrel should be of the same length. If the wedges are inserted (without a cable) snugly into the barrel, both ends of the wedge cluster should be flush with the barrel. This will ensure the proper positioning when installed on a cable. When installed, the narrow end of the wedges should not extend past the back of the barrel while the wide end should not countersink.

- The curvature of the wedges should be equal to that of the inside of the barrel when installed on a cable. This means that without a cable, only the corners of the wedges touch the barrel. Incorrect curvature leads to wedge failure.

- The outer diameter of the barrel should be 65-100% greater than the diameter of the hole in the plate to prevent punch-through.

- Barrels with rounded backs (on plate side) improve plating performance where cables are not perpendicular to the rock face. Use rounded barrels only with matching plates which have a curved recess to accept the barrel. Rounded barrels on flat plates will result in point loading which can lead to plate failure.

- Three component wedges improve the efficiency and consistency of grip but impose additional cost and handling difficulties. Two component wedges are most commonly used in mining.

- The outer barrel must be made of a tough steel to resist splitting. There should be no risk of barrel splitting for cable loads in excess of 25 tonnes.

- The inside surface of the wedges must have hardened teeth (approximately 1 mm pitch) in order to bite into the cable steel, thereby achieving a positive grip. The teeth should show no tendency to crush or flatten during installation.

- The wedge teeth should be 0.4-0.6mm high. The tooth face towards the borehole should have a slope of approximately 30 degrees with respect to the cable axis. Away from the borehole the teeth should have a steeper, 60 degree slope. This ensures efficient jacking and high residual gripping capability.

- Do not use wedges from one batch with barrels from a different batch or a different supplier.

- The wedge (and inner barrel taper) angle should be between 7 and 9 degrees with respect to the cable axis. Most wedges are manufactured at the optimum angle of 7.5 degrees. Gripping performance degrades with higher wedge angles. Shallower angles will result in excessive wedge displacement during installation, require longer barrels and can lead to unpredictable performance.

- The barrel should provide sufficient thickness of steel adjacent to the wide end of the wedge - the tapered hole should not extend to the outer edge of the barrel. This will increase the risk of splitting.

- The wedge angle must be exactly equal to the barrel taper angle. This can be checked by inserting the wedges snugly into the barrel (without a cable). There should be no separation between each of the wedges and between the wedge corners and either end of the barrel. The wedges will spread when installed on a cable but the wedge angle will not change. The inside of the wedges should remain parallel with or without a cable in place.

Installation Considerations - Tensioning and Plating

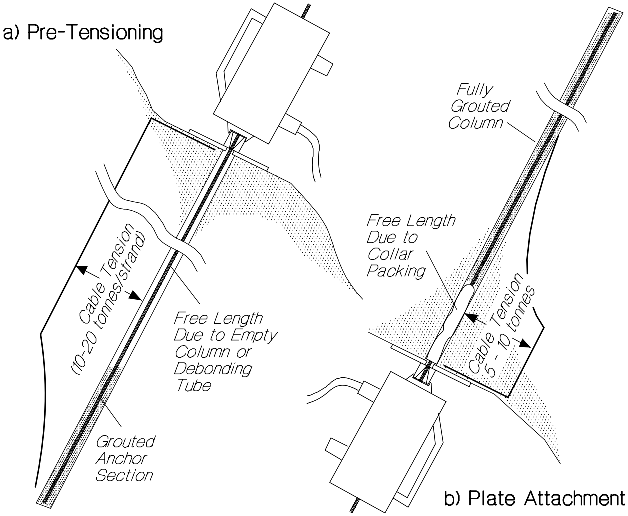


Figure 2.7.2: Cablebolt anchorage for: a) Pre- tensioning; b) Cablebolt plating

In civil engineering and in open pit mining applications, the barrel and wedge anchor is used with a jacking system to generate and to retain a preload in the cable of 10 to 20 tonnes per 15 2 mm cable. These cables would typically have ungrouted (or debonded) lengths of 4 to 10 metres between a grouted anchor section and the borehole collar and plate (the remainder of the hole may or may not be grouted after tensioning). The purpose of this procedure is to create an immediate active load at points in an array of cables, in order to directly improve stability by reducing displacement, enhancing joint interlock and by increasing normal pressure and thereby friction along potential slip surfaces.

This application differs significantly from the more common underground mining application - the attachment of plates or straps to the free end of cable bolts. The load generated in the steel serves primarily to maintain a frictional grip within the barrel/wedge/cable assembly at the collar. In addition, the active load on the plate provides limited compression to the immediate surface rockmass, slightly improving local stability. The cable loads generated in this application range from 5 to 10 tonnes. A drop in cable tension could result in a loosening of the barrel/wedge anchor and loss of the plate. The free length of *effectively ungrouted* cable includes any length of (non-grouted) packing at the collar and some unknown length of cable in the grout column adjacent to the collar (dependent on cable type, cable installation method, grout quality, load magnitude and time elapsed since grouting). Average effective free lengths can range from 0.5 to 2 metres.

The difference in free length and the magnitudes of the applied and residual loads are the most significant differences between the two applications and determines, for the most part, the type of jacking system to be used.

The load retained in the cable is critical for the effective gripping of the barrel and wedge anchor. If the anchor meets the desired specifications, this post-installation load is influenced by the jacking configuration, the jacking load, the effective free length of the cable, the surface character of the rock face and the presence of screen or soft backing behind the plate. If this load is inadequate, the anchor may loosen during blasting or seismic activity. Improperly installed anchors have been observed by the authors to fall off or creep down the cable under dynamic loading or by static load generated by near-surface rockmass displacement.

A number of jacking configurations are in current use. The type of jack can profoundly affect the quality of the anchor installation. Research performed by the CSIRO (Thompson, 1992) has shed considerable light on this issue. The results of this work are summarized here with respect to plating applications. Note that configuration A in Figure 2.7.3 is recommended for underground cablebolt plating applications.

Jacking Configurations

There are several options for the application of the reaction load between the jack and the anchor assembly (*Pw* is the percentage of the total jack load, *Pt* which is applied directly to the wedges and represents the main difference between the four configurations).

A. 100% of the jacking load applied to the wedges. In this case, the wedge load, *Pw* = 100% (of the total jack load).

Recommended for Plate Installation

B. 100% of the jacking load applied to the outside of the barrel with no load applied to the wedges (*Pw* = 0%).

Highly Undesirable

C. Load applied to the barrel with a fixed spacing nose cone partially loading the wedges (0% < *Pw* < 100%).

D. Load applied to the barrel with a spring loaded nose cone applying partial (and consistent) loading to the wedges (*Pw* ≈10%).

Recommended for Pre-Tensioning

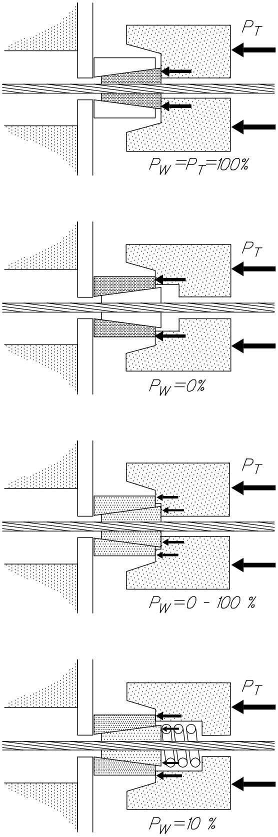


Figure 2.7.3: Jacking Configurations (after Thompson 1992)

Three key considerations determine the correct reaction configuration:

- The percentage, *Pw*, of the total jacking load which is applied directly to the wedges during the installation process.
- The relative magnitude of peak cable load, *Pmax*, generated behind the anchor (in the borehole) compared with the total load generated by the jack.
- The residual load, *Pres*, remaining in the cable after the jack is removed.

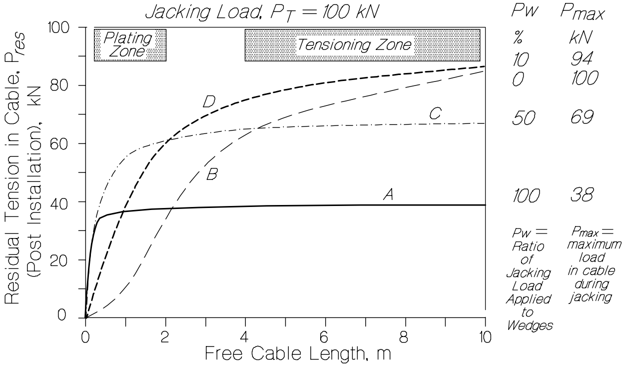


Figure 2.7.4: Theoretical relationships between initial wedge load, *Pw* (% of total *Pt*), peak load, *Pmax*, during installation and the residual cable load, *Pres*, for a nominal jacking load, *Pt* of 10 tonnes (after Thompson, 1992)

When the reaction load is applied directly to the wedges (A: *Pw*=100%), the wedges lock into the cable and barrel before the peak load in the jack is reached. This lock-in prevents additional load transmission to the cable behind the anchor. Instead, the jack continues to load only the length of cable between the anchor and the jacking grip. The peak cable load behind the plate will be 30 to 50% of the peak load registered by the jack even for very short free lengths (Line A in Figure 2.7.4). Nevertheless, when the jack is removed the load in the cable behind the plate remains relatively constant with little relaxation. This is because there is little or no additional draw-in of the wedges after jack removal.

Jacking units currently available with spring-loaded nose cones apply a maximum of 1 tonne (10 kN) onto the wedges while applying a total of 10 tonnes to the anchor assembly. In this case, when only a partial load is applied to the wedges (*Pw*=10%) the peak load in the cable behind the anchor approaches the maximum total load registered by the jack as shown in Figure 2.7.4 for longer free lengths. This is a substantial increase in efficiency (Line D).

With this second jack there is, however, the increased probability of additional draw-in of the wedges after the jack is removed. If there is a long free length (4 to 10 m) this draw-in is accommodated with little relaxation of load in the cable. If the free length is shorter than 3 metres, this draw-in can result in significant load relaxation. In addition, the nose cone spring in current models has been observed to fatigue, causing *Pw* to drop to 0%, and lose its effectiveness over time and extended usage (Windsor and Thompson, pers. communication).

It should be noted that jacking systems which apply 100% of the load solely to the barrel (*Pw*=0%) are totally unacceptable for plating applications (Line B). The residual load in the cable can approach zero in this configuration. In tensioning applications with longer free lengths, the results can be unpredictable.

A nose cone with a calibrated recess to apply delayed loading to the wedge can be effective for tensioning applications (Line C). When plating cables with short free lengths, strict dimensional tolerances between the nose cone and the anchor are required. It is advisable to purchase the jack and anchors from the same supplier in this case, and to ensure perfect matching between the two.

Recommended procedures

Now that the basic mechanisms involved in the jacking process have been examined, it is possible to recommend the appropriate jacking system to ensure that the capacity of the anchorage is maintained or that the desired pre-tension in the cablebolt is achieved:

Plating and surface fixture attachment (Free length <2 m)

Jacks which apply full load to the wedges during installation are recommended for barrel and wedge anchorage of surface fixtures where a residual cable load of 4 tonnes is required. Note that using this method, a nominal jacking load of 8 - 12 tonnes (2-3 times the desired residual load) is required due to the inherent inefficiency of the system. Inadequate loads lead to anchor failure.

Tensioning (Pre- or Post) (Free length > 4 m)

Jacks which apply a partial load to the wedges by means of a nose cone spring are recommended for tensioning applications where high residual loads are required and most importantly, where long free lengths are present. If the spring fatigues with use (as is the case in current designs) then the wedge pressure will approach zero and the consistency of the installation will degrade and may not be acceptable. The spring should not be compressible by hand - it should exhibit only a slight give. Maintenance of the spring is especially critical if such a jack must be used for plating or tensioning of cables with less than 4 metres of free length. In this case the rigid nose cone (*Pw*=100%) is preferred.

Key plating and tensioning tips:

- **Never allow oil, dirt or significant rusting** to occur on barrels and wedges before installation. Ensure that the cable segment receiving the anchor is clean.

- Recommended tension load for plate or strap attachment is 4 tonnes. **A rule of thumb is that 8 -12 tonnes of nominal jack load is required** to ensure this minimum residual cable tension regardless of the installation method used.

- **Do not confuse jack pressure with jack load**. Ensure that the gauge is clearly marked. Ensure that the pressure to load conversion factor (i.e., the area of the ram in the jack) is clearly affixed or marked on the jack for later reference. Clearly specify the jacking limit in the same units as the gauge. It is confusing to specify cable load in tonnes if the jack gauge is in p.s i. or kPa.

- **Do not exceed yield strength** of the cable strand (20 tonnes) during tensioning. Premature rupture can initiate at the wedges at loads close to yield.

- When plating, a light bending of plates is desirable as a quality control measure (post-installation rebound should be minimal). Significant bending (>5 degrees) indicates underdesigned plates or excessive jack loads and should be avoided.

- When plating against pre-installed screen, jack the cable until the plate touches the rock. The screen cannot provide the necessary rigidity to ensure anchor integrity. Blast vibration will shake the anchors loose.

- When installing plates against rough rock surfaces, it is desirable to apply sufficient jacking load to crush the tips of sharp surface ridges in order to establish a more positive contact with the surface. These small rock edges may otherwise be destroyed by blast vibrations of rock creep, reducing the residual load in the cable and impairing the load capacity of the anchor. Thick stiff rubber backings may also be used to dampen blast vibrations where problems are observed (failed plate anchors) and to allow for limited face displacement.

- It is useful to affix strong reflective tape or a paint stripe on the cable 5 mm from the top of the wedges after installation, to serve as a draw-in indicator.

- Do not attempt to attach B&W anchors to modified strand unless a suitable length of plain, unmodified strand is left at the collar end of the cablebolt (discuss this with the supplier). Allow enough unmodified length outside the hole for anchoring and jacking as well as at least 0.5 m inside the borehole.

- When plating a double strand cablebolt use two anchors if necessary. Avoid the practice of bending the second strand behind the plate. This leads to an imperfect contact between the plate and the rock. The preferred option is to recess the second cable, exposing only a single strand to be anchored.

- When installing cables over screen, it is preferable that the screen not be cut to accommodate the cable and plate. Instead, place the plate and anchor over the screen, mating the retention system to the support system.
- When screen has been previously stretched flat across a large depression in the rock surface making tight placement of a plate impossible, the screen should then be cut to allow placement of the plate and anchor on the rock surface.

Anchors under dynamic loading

Under severe dynamic loading, such as heavy blasting or rock burst (seismic) conditions, a properly tensioned barrel and wedge anchor with sufficient free length behind the plate should perform adequately. In the authors' experience, there have been instances, however, where the anchors lose their grip under severe vibration. In some cases, the barrel and wedge anchors can be ejected under high velocity and can constitute a safety hazard (Seldon, 1996, pers. communication).

In these conditions, the first recommended option is the use of domed or curved plates (Section 2.7.1). The dome must be strong enough to withstand the initial tensioning/plating load or 10-12 tonnes but should be designed to collapse under more than 18 to 20 tonnes. This will provide a seismic displacement contingency.

The use of double anchors on single cables may also provide a "belt and suspenders" solution, providing that the dynamic loading is not enough to break the cable strand (Figure 2.7.5). The first anchor will be thrust onto the second anchor forcing the wedge to grip.The disadvantage of this system is that it will be difficult to achieve sufficient residual load P_{res} in the outer anchor due to the lack of free length.

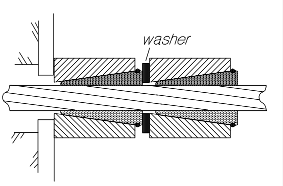


Figure 2.7.5: Double B&W anchor

Another alternative which has not been widely used and which should be tested before adoption, is the use of a swaged (pressure-fitted) steel anchor (Section 2.9.3) in place of a barrel and wedge. Quality control is difficult to achieve with this system but the displacement characteristics of such an anchor would be ideal for plating in dynamic loading conditions. In static load pull tests (Goris, 1990), the swaged anchors have been shown to yield at a constant load slightly below the yield point of the strand. This characteristic is ideal for dynamic support. It is not possible to apply any tension to the plate with a swaged anchor and so the system will be very soft, applying no initial load to the rock surface.

Steel straps used for support in bursting conditions should have slots through which the cables are installed. The strap can slip with respect to the cablebolts to accommodate rock displacement during blasting or seismic disturbance.

2.8 Shear Loading of Cablebolts

While most of the research and testing of cablebolt capacity deals with axial loading (pullout), actual conditions in the field normally include at least some component of shearing. For this reason, recent research (Windsor, 1992; Windsor et al., 1988; and Bawden et al., 1994) has focussed on the response of various cable configurations to different modes of shearing. Section 2.2.2 describes the testing apparatus used to perform these tests. The following discussion is a brief summary of cablebolt response and is not intended as a comprehensive reference.

2.8.1 Direct Shear

Windsor (1992), Windsor and Thompson (1993) and Windsor et al. (1988) describe a testing device for grouted cablebolts subjected to direct shear (Figure 2.2 6). The cables can be oriented with respect to the shearing surface to simulate pure shear (noted as 90°), shear + tension (135°) and shear + compression (45°). Note in Figure 2.8.1 that cables oriented at 135° degrees to the direction of shear, such that tension and shear are mobilized, give the stiffest results. Cables which experience compression, kinking and shear (45°) have very soft responses and reduced capacities.

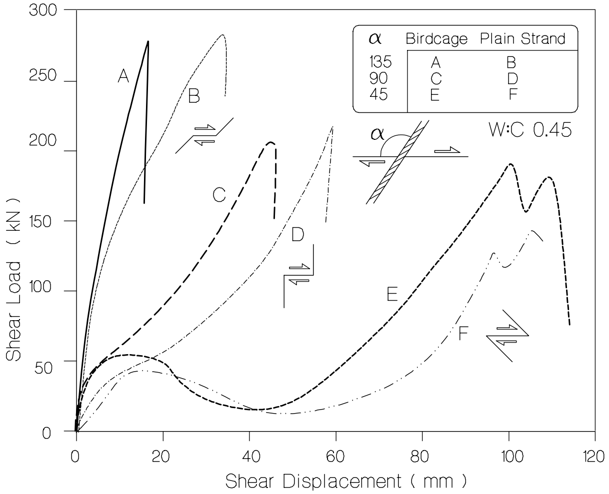


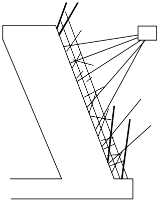
Figure 2.8.1: Typical results from direct shear tests of cablebolts (after Windsor and Thompson, 1993; Windsor, 1992; Windsor et al., 1988)

2.8.2 Oblique Loading - Shear

Consider the hangingwall block examples shown in Figure 2.8.2. In a simplified gravity loading scenario, the block moves down under its own weight. Cablebolts are installed perpendicular to the hangingwall to support the block. The relative components of shear and axial loading experienced by the cablebolts will depend on the angle of the hangingwall. For a horizontal surface, the loading will be purely axial. Shearing of the cable increases with increasing inclination of the wall and of the separation plane.

Bawden et al. (1994) present preliminary results from an extensive testing program using the apparatus shown in Figures 2.2.7 and 2.8.2 to investigate this scenario. A summary of these results is shown in Figures 2.8.2 and 2.8.3. Note that the ultimate capacities of the strands does not show significant reduction with increased shear. This is likely due to crushing of the unconfined grout at the separation plane. For steep angles of loading > 45 degrees, the tendency toward axial pullout is reduced as shearing becomes dominant. In short embedment lengths this gives the impression of increased capacity.

In shear, the system stiffness over the first 10 to 20 mm of slip is reduced (Bawden et al.,1994). This is a significant finding and can explain the inability of low angle cablebolts to effectively reinforce sloughing stope walls. Such cables are designed to prevent beam delamination (axial displacement) but are less effective, for example, when the hangingwall is undercut and displacements become vertically downward. If undercutting is suspected, high angle (closer to vertical) cables should be included in the array as shown at right.



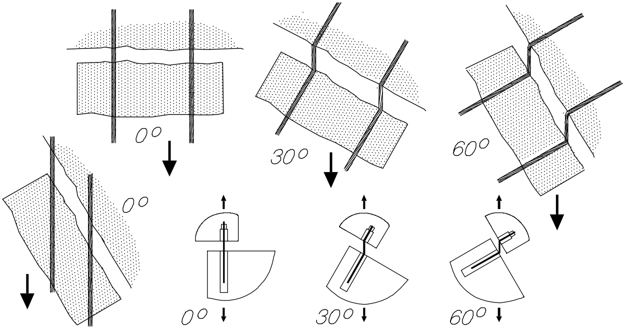


Figure 2.8.2: Analogous field conditions corresponding to oblique axial/shear testing

Oblique Loading - Results

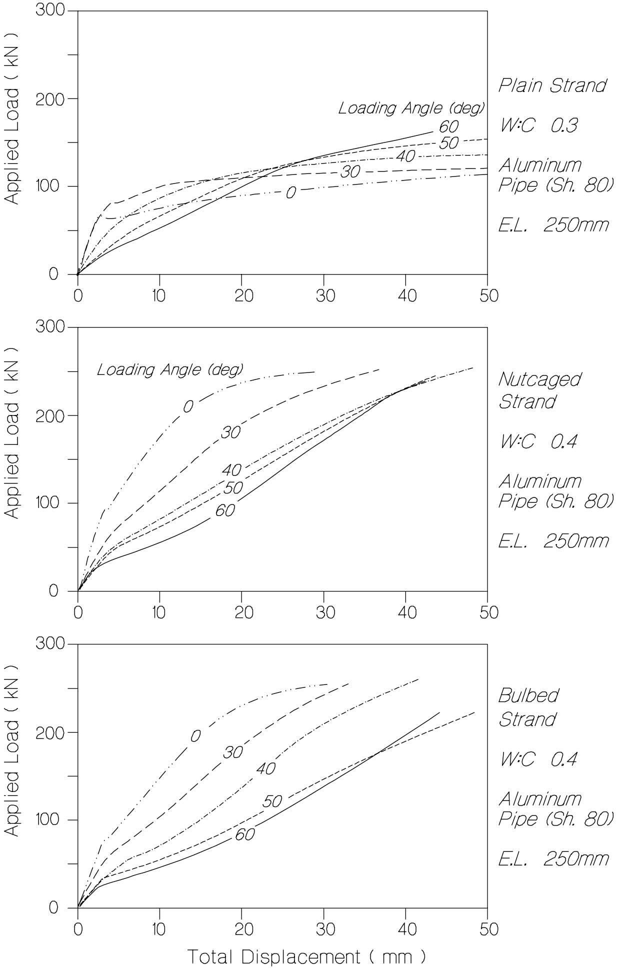


Figure 2.8.3: Example tests results - oblique loading. (after Bawden et al., 1994)

2.8.3 Cablebolt Orientation

A few simple conclusions pertaining to the optimization of the cablebolt installation angle can be drawn from the discussions in Sections 2.8.1 and 2.8.2.

- Cablebolts installed across a confined shearing surface (sustained contact during shear) perform best when oriented at an acute angle to the direction of the discontinuity and to the direction of shearing. In this case an angle of 30 to 60 degrees is recommended. Ensure that the orientation is chosen such that the shearing immediately induces stretch and not buckling in the cable strand. This will optimize cable behaviour and will also induce confining load on the shearing and dilating discontinuity, effectively increasing the frictional resistance. If this orientation is not practical then follow the next point:

- Cablebolts installed across any surfaces which experience normal separation (i.e. are pulled apart) should be installed parallel to the direction of displacement, regardless of the orientation of the discontinuity.

It is therefore important to understand the nature of the displacement across a discontinuity and to orient the cablebolts accordingly as in Figure 2.8.4.

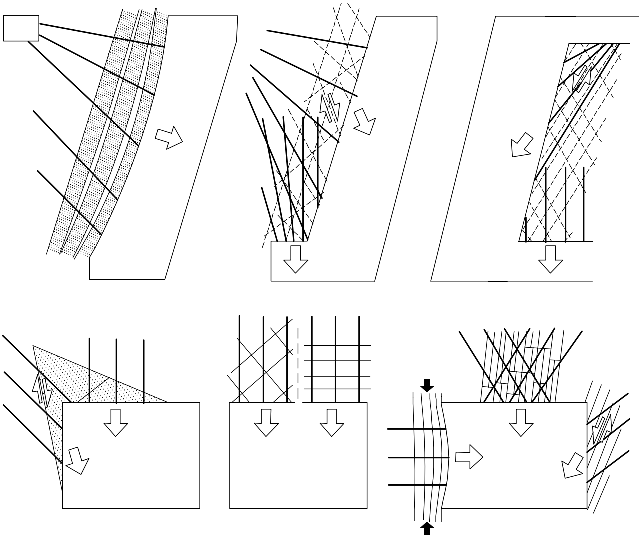


Figure 2.8.4: Optimization of cablebolt angle

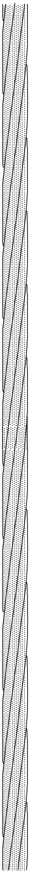
2.9 Cablebolt Strand Alternatives

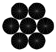
The following section summarizes the engineering properties and installation recommendations for a number of the cablebolt strand options currently available. Many of the options discussed are readily available from suppliers in North America, Australia and elsewhere at an increasingly competitive cost. While 15.2mm plain strand cable has formed the basis of much of the discussion in this book, one purpose of this discussion is to illustrate many of the detrimental sensitivities inherent in this device. While there have been hundreds of successful applications of plain strand cable in civil construction and in mining, the diverse and ever-changing underground environments typical of hard rock mining warrant the consideration of a wider toolbox of cablebolt options.

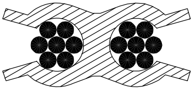
Table 2.9.1: Strand summary: (* indicates a detailed summary in the following section)

Plain Strand - 15 2 mm *	The basis of all other steel strand alternatives
Modification by Coating	Increase corrosion resistance
Epoxy Coated *	- *Outside of the cable is coated*
Epoxy Encapsulated *	- *Epoxy penetrates internal spaces in strand*
Modification by Unwinding	Increase Bond Strength & Stiffness
Birdcaged Strand *	- *Disassemble wound plain strand and rewind out of phase to create an open cage*
Birdcaged 14-wire *	- *As above, but created with two plain strands unwound and rewound coincidentally*
Modification by Inclusion	Increase Bond Strength & Stiffness
*Nutcaged Strand**	- *Disassemble plain strand; Rewind tightly while inserting hexagonal nuts at intervals over the king wire*
Ferruled Strand	- *Inclusion is a rounded ferrule instead of a nut*
Modification by Deformation	Increase Bond Strength & Stiffness
*Bulbed Strand**	- *Grip intact plain strand in hydraulic device and symmetrically kink the wires into a flared bulb Strand between bulbs is undisturbed*
Modification by Attachment	Increase Bond Strength and Stiffness (Locally)
Buttons or Swages *	- *Clamp, press or weld cylindrical steel "buttons" onto strand at prescribed intervals*
B&W anchors	- *Install pairs of conventional barrel and wedge anchors such that the wedges of the opposing units press against each other and lock onto the cable*
Modification by Debonding	Eliminate bond along partial strand length to increase displacement capacity
Tubing	- *Insert tubing over debonding length*
Paint, Grease, Coatings	- *Apply paint or heavy grease to strand prior to grouting*
Double or Multiple Cables *	All of the above options can be used in tandem (double strand) or in combination (e g. one bulbed strand and one plain strand in each hole). Single modified strands can include differently modified segments.

2.9.1 Plain Strand







Recommended Grout: 0 35 (Grout Tube) 0 38 (Breather-Tube)

Minimum Hole Diameter (Sct.2.10):	Single	48 mm	48 mm
	Double	64 mm	64 mm

Recommended Applications:

Moderately blocky ground with limited potential for relaxation after installation. Semi-ductile interface allows for moderate dynamic loading. Effective in uphole installation with plates.

Capacity Notes: (references: see Sections 2.4 to 2 6)

Initial Stiffness
- 50 % of Pullout Load after 1 to 2 mm; 80 % after 10 to 20mm

Displacement Capacity
- Dependent on embedment length.
  In the event of pullout (no strand rupture), high residual strength is maintained for 40 to 80 mm. Otherwise strand rupture occurs within 20mm. Double strand increases stiffness (up to 100 %).

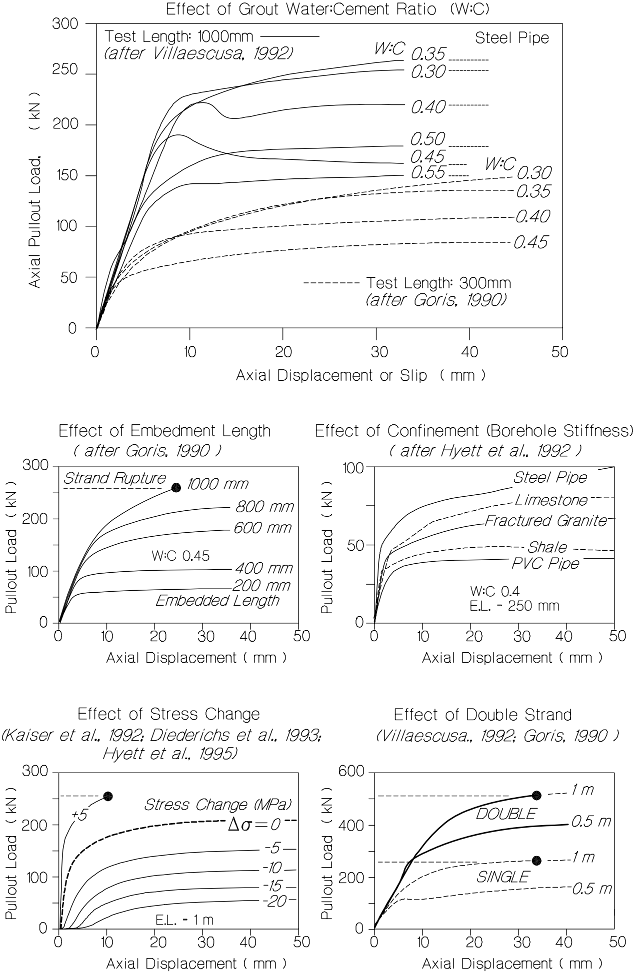
Load Capacity
- Pullout load with respect to embedment length ranges from 20 to 35 tonnes/m (1 tonne ~ 10 kN) for recommended grout range but is highly sensitive to stiffness and stress change.
- Tensile capacity: Yield = 20 tonnes/ strand,
  Rupture = 25 tonnes / strand.

Sensitivity
- Highly sensitive to reduction of rock modulus below 10 GPa.
- Sensitive to stress change - Relaxation of rockmass can reduce bond strength to near zero if severe.
- Quality control is necessary with respect to grouting and storage.
- Quality and surface condition (clean) of strand critical.

Advantages:
- Readily available, inexpensive and easy to install
- Can be shipped in continuous reel
- Will fit into a smaller hole than modified geometries
- Easily fitted with plates and surface anchors
- Relatively ductile system

Disadvantages:
- Extreme sensitivities as noted
- Lowest bond strength and highest critical embedment length

Plain Strand



Effect of Grout Water:Cement Ratio (W:C)

Effect of Embedment Length (after Goris, 1990)

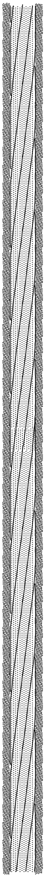
Effect of Confinement (Borehole Stiffness) (after Hyett et al., 1992)

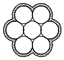
Effect of Stress Change (Kaiser et al., 1992; Diederichs et al., 1993; Hyett et al., 1995)

Effect of Double Strand (Villaescusa., 1992; Goris, 1990)

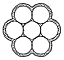
Figure 2.9.1: Performance summary for plain strand cablebolts

2.9.2 Epoxy Coated/Encapsulated Strand





Epoxy coated cablebolt



Epoxy encapsulated cablebolt

Recommended Grout: 0 35 (Grout Tube) 0 4 (Breather-Tube)

Minimum Hole Diameter (Sct.2.10):	Single	48 mm	48 mm
	Double	64 mm	64 mm

Recommended Applications:

Same as for plain strand.
Provide corrosion protection in aggressive environments.
Epoxy coatings should be certified for the type of corrosion expected (e g. acid, electrolytic), for flexibility of coating, abrasion resistance, etc. Epoxy should have embedded grit.
Epoxy *encapsulated* cables have epoxy filling the internal voids in the strand. These channels in *coated* strand can otherwise provide concentrated corrosion sites in aggressive environments.

Capacity Notes: (ref: Goris, 1990; Goris et al., 1994; Littlejohn, 1993; Windsor, 1992; Dorsten et al., 1984)

Initial Stiffness
- 70 % of Pullout Load after 1 to 2 mm; 90 % after 10 mm

Displacement Capacity
- Dependent on embedment length.
- Comparable with plain strand although coating may rupture after large displacements (> 100 mm) permitting concentrated corrosive attack in aggressive environments.

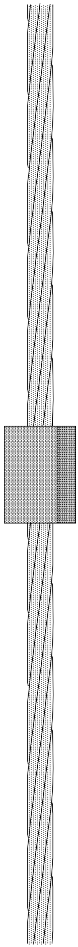
Load Capacity
- Pullout Load up to 30% higher (with grit coating) than plain strand
- Tensile capacity: Yield = 20 tonnes / strand
  Rupture = 25 tonnes / strand

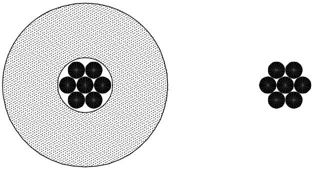
Sensitivity
- Improved corrosion protection.

Advantages:
- Installation procedure same as for plain strand
- Corrosion resistance and long service life
- Slightly improved bond performance

Disadvantages:
- Cannot be plated unless stripped
- Expensive

2.9.3 Swaged/Buttoned Strand





Recommended Grout: 0.4 (Grout-Tube*) 0 4 (Breather-Tube**)
Preferred Method for: *Downholes **Upholes

Minimum Hole Diameter: Allow for button and grout / breather tube

Recommended Applications:
Support across known joint surfaces
Anchorage for debonded cable sections
High relaxation zones

Capacity Notes: (ref: Goris, 1990; Goris et al., 1994;Schmuck,1979)

Initial Stiffness
- Controlled by embedment depth and rigidity of button fixture.

Displacement Capacity
- In the event of pullout (no strand rupture), high residual strength is maintained for 40 to 100 mm. Otherwise strand ruptures within 10-20mm. Buttons can be designed with yielding limit for ductility.
- Button can be used to provide positive anchorage around a debonded segment.

Load Capacity
- Pullout increased 150% for button depth >100 mm (Fig. 2.9 3)
- Pullout load reduced from plain strand if button depth < 50 mm
- Tensile capacity: Yield = 20 tonnes
  Rupture = 25 tonnes.

Sensitivity
- Sensitive to button depth (placement with respect to fractures) If proximity to fracture <50mm, grout is pushed out of hole
- Sensitive to button fixture quality control
- Sensitive to button spacing and attachment method

Advantages:
- Bond is less sensitive to stress change and confinement
- Provides positive anchorage at button
- Cablebolt can be easily plated

Disadvantages:
- Sensitivity to button placement
- Large borehole required
- Sensitive to voids on lee side of button (w.r.t grout flow)

Epoxy Coated Strand

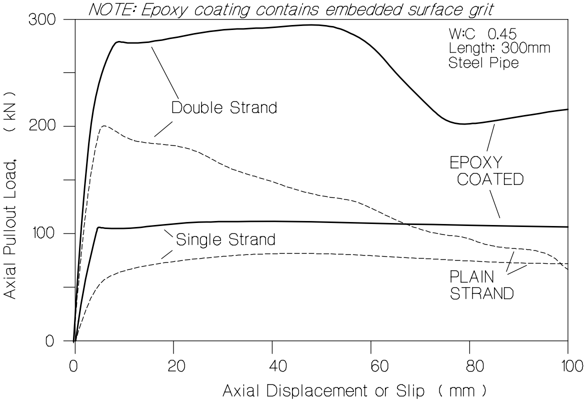


Figure 2.9.2: Pullout response for epoxy coated strand (after Goris, 1990)

Strand with Buttons

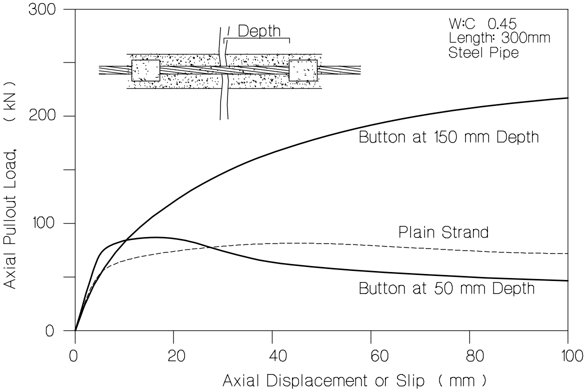
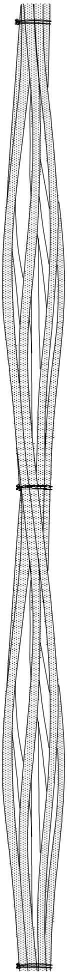
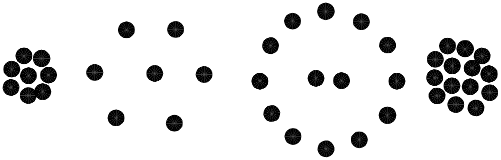


Figure 2.9.3: Pullout response of strand with buttons or swages (after Goris, 1990)

2.9.4 Birdcaged Strand





Recommended Grout	0.37 (Grout-Tube*)	0.4 (Breather-Tube**)
Preferred Method for:	*Downholes	**Upholes

Minimum Hole	Single:	64 mm	57 mm
Diameter:	Double:	76 mm	76 mm
(Sct. 2.10)	14-wire:	76 mm	70 mm

Recommended Applications

Highly fractured ground
Ground with potential relaxation after installation
Stiff system for immediate load response

Capacity Notes: (ref: Goris, 1990; Goris et al., 1994; Hutchins et al., 1990; Cortolezzis, 1991)

Initial Stiffness

- Double the initial stiffness of plain strand

Displacement Capacity

- Variable
- Can be high in low W:C grout

Load Capacity

- Pullout load strength in short embedment lengths 35 to 80 % higher than plain strand.
- Strand ruptures at 20 - 22 tonnes due to eccentricity of loading on individual wires

Sensitivity

- Birdcages should be firm to the grip. Loose birdcage reduces effectiveness and increases installation difficulty.

Advantages:

- Only slightly more expensive than plain strand
- Stiff, strong system
- Birdcage has no central channel to contribute to grout bleeding

Disadvantages:

- Cannot be plated unless plain strand section is left at end.
- Difficult to insert and handle
- Cannot be shipped in a continuous coil
- Requires larger borehole
- Response in partial or full shear may be unpredictable due to uneven loading of wires
- Cannot be installed with standard automatic cablebolt pushers

Birdcaged Strand

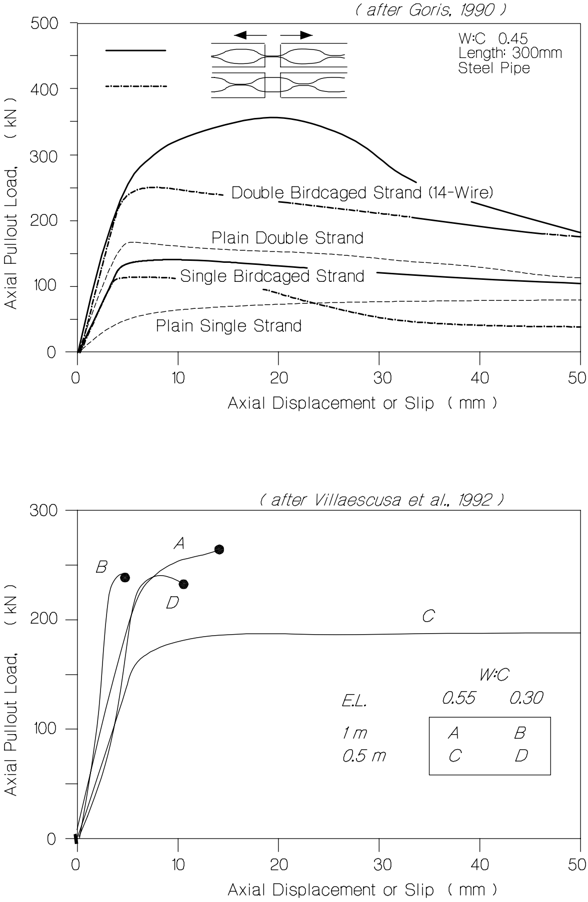
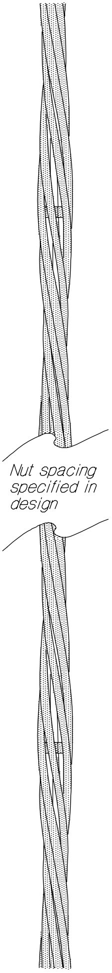


Figure 2.9.4: Performance summary for birdcaged strand

2.9.5 Nutcaged Strand



Recommended Grout		0.37 (Grout-Tube*)	0.4 (Breather-Tube**)
Preferred Method for:		*Downholes	**Upholes
Minimum Hole Diameter (Sct.2.10):	Single:	51 mm	48 mm
	Double:	57 mm	51 mm

Recommended Applications

Highly fractured ground
Ground with potential relaxation after installation
Higher ductility than birdcage and bulbed strand

Capacity Notes: (ref: Hyett et al., 1993; Bawden and Hyett, 1994)

NOTE: Recommended nut size 12 - 16 mm

Initial Stiffness

- Stiffness for embedment lengths of 300 mm are at least 100 % greater than plain strand. Larger nut gives higher pullout stiffness.

Displacement Capacity

- In moderate confinements at embedment lengths of 300 mm, cable rupture occurs at 25 - 30 mm for 15 9 mm nut and at 40 to 50 mm for 12.7 mm nut.

Load Capacity

- Pullout loads 100 - 200 % greater than plain strand (for 300 mm lengths) depending on grout and confinement.
- Tensile capacity: Yield = 20 tonnes
  Rupture = 25 tonnes.

Sensitivity

- Sensitive to nut size. Maximum recommended = 16 mm
- Sensitive to reduction of rock modulus below 10 GPa. Strength still 2 to 3 x plain strand in these conditions

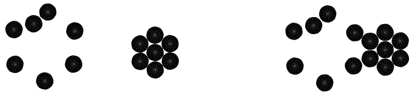
Advantages

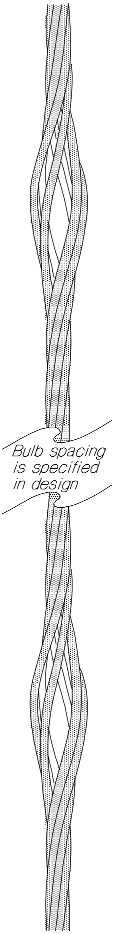
- Reasonable ductility (adjusted through nut size)
- Small hole size

Disadvantages

- Cannot be manufactured in continuous coils at this time
- Failure mode inconsistent with large nut sizes

2.9.6 Bulbed Strand





Recommended Grout 0 37 (Grout-Tube*) 0 4 (Breather-Tube**)
Preferred Method for: *Downholes **Upholes

Minimum Hole	*Single:	64 mm	57 mm
Diameter (Sct.2.10):	**Double:	57 mm	51 mm

*35 mm bulb used above for singles; **25 mm for doubles

Recommended Applications

Highly fractured ground
Ground with potential relaxation after installation

Capacity Notes: (ref: Hyett et al., 1995; Bawden et al., 1995; Garford, 1990; Stjern, 1995)

Initial Stiffness
- Approximately double the stiffness of plain strand
- 50 % pullout load after 2 to 5 mm; 100 % at 20 mm

Displacement Capacity
- Limited but consistent. Wire rupture initiates at 20 to 30 mm.

Load Capacity
- Pullout loads are close to strand capacity for 300 mm embedment lengths for *W:C* = 0.4 and a range of confinements.
- Wire rupture initiates at around 24 tonnes

Sensitivity
- Mildly sensitive to reduction in rock modulus below 10 GPa.
- Grout should fill bulb structure for efficient load transfer
- Increased bulb diameter >35mm= unpredictable pullout loads

Advantages
- Inexpensive and easy to install
- Can be shipped in continuous reel
- Can be easily customized to fit plates or debonded sections
- Bulb spacing and diameter can be specified to suit application

Disadvantages
- Poor grouting could lead to minimal improvement over plain strand (Recommend *W:C* = 0.4).

Nutcaged Strand

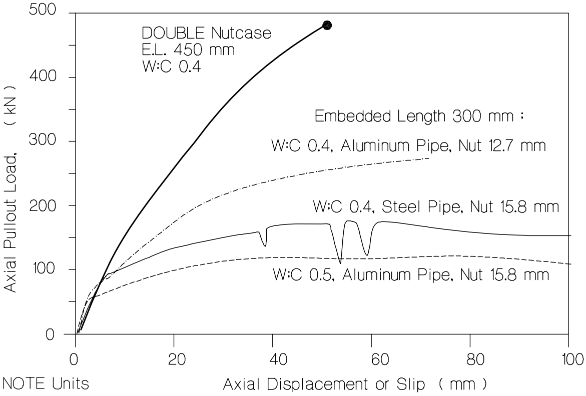


Figure 2.9.5: Sample pullout performance of nutcaged strand ("nutcase" after Bawden and Hyett, 1995; Hyett et al., 1993)

Bulbed Strand

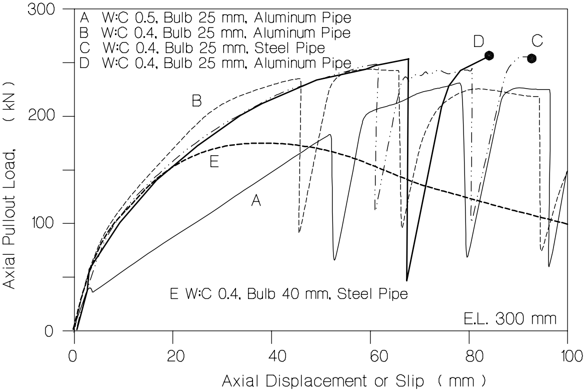


Figure 2.9.6: Sample pullout performance of bulbed strand (Garford Pty. 1990; Bawden and Hyett, 1995; Hyett et al., 1995)

2.9.7 Combination Strand

The strand configurations on the previous pages can be combined in parallel (Stjern, 1995) or in series as shown at right.

Parallel combinations are double strand cablebolt elements intended to combine the beneficial characteristics of two different strand types such as plain and bulbed strand. A birdcage and plain strand combination would, for example, also facilitate plating of a stiff modified element. Unfortunately, in many cases, the stiffness characteristics of the two strands may not be compatible and the stiffer strand will rupture before the other has a chance to carry significant load. The capacity of the system may not, therefore, be comparable to double strand and in fact may be closer to that of a single strand. This configuration is not recommended for standard use in most mining applications.

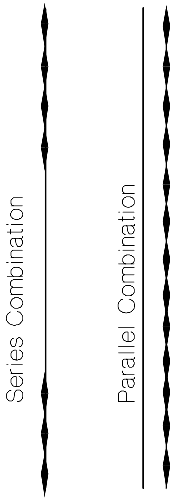


Figure 2.9.7: Combinations

Series combinations can be fabricated to give different bond characteristics along the cable. A single strand with a debonded length (Section 2.6.4) and with birdcaged, bulbed or buttoned end segments would serve to provide a soft and dynamically resilient connection between two strong and stiff anchorages for use in fractured ground subject to seismic disturbance. The modified end lengths would provide reliable bond strength to maintain integrity of the near face rockmass and to ensure adequate anchorage while the debonded or plain strand segment would accommodate dynamic displacement or excessive fault shear as shown in Figure 2.9 8.

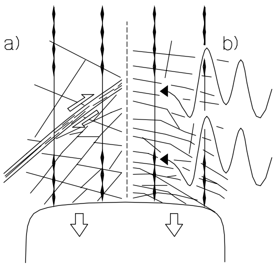


Figure 2.9.8: Series Combinations
a) Fault shear
b) Seismic loading

2.9.8 Strand Selection

The logic for selection of strand type can be summarized as shown in Figure 2.9.9 which describes various operational and engineering considerations.

Strand Selection

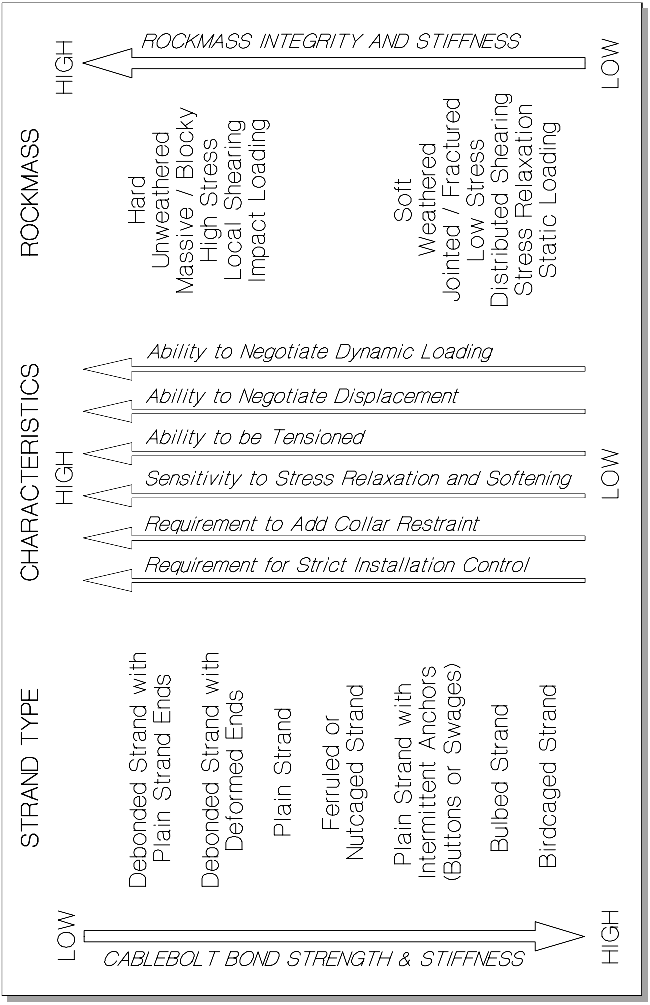


Figure 2.9.9: Cablebolt strand selection logic (modified after Windsor, 1992)

2.9.9 Strand Alternatives: Fibreglass Cablebolts

In an effort to develop a cuttable lightweight cablebolt which would provide an alternative to the conventional steel strand and its modified derivatives, several researchers in Canada (Pakalnis et al., 1994; Mah et al., 1991; Mah, 1994, Peterson et al., 1992), along with partners in the mining industry investigated the use of fibre composite strands (Figure 2.9.10.a). After testing a four strand product made from expensive European strand (Type A in Figure 2.9.10.b) and obtaining promising results, they developed a more economical product called the DAPPAM bolt (Type B in Figure 2.9.10.c).

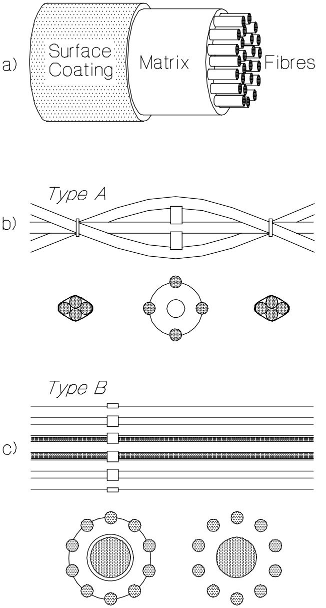


Figure 2.9.10: Fibreglass cablebolts (after Pakalnis et al., 1994)
a) Strand construction
b) Type A c) Type B

This bolt is composed of 10 fibreglass strands on a circular spacer which has a grouting tube pre-installed down the centre of the assembly. The numerous individual fibres in each strand are encased in a matrix. The composite strand itself is then coated with a surface material which can be smooth, rough, laced with fibre or permeated with grit. The surface coating can be specified when ordering and is critical to the strength of the interface with the chemical or cement based grout. The effect of various surface coatings on the pullout bond strength in Portland cement grout (W:C=0.35) is shown in Figure 2.9.11.

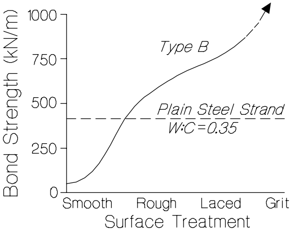


Figure 2.9.11: Bond strength vs surface type

Corrosion of fibreglass composites can be a problem in certain environments (Reinhart and Clements, 1988). The chemical makeup of the matrix and surface coating must be selected to provide adequate protection against alkaline Portland cement or against the acidic conditions found in many mines.

The Type B bolt (Pakalnis et al., 1994) weighs approximately one quarter as much as a single steel strand cablebolt of the same length. This can result in a significant increase in productivity. Installation trials by Pakalnis et al. have shown comparable overall installation costs (including drilling) of approximately CDN $20.00/m. The bolt's main advantage is it's cuttability. This can lead to higher productivity in cut and fill operations and where automated excavators are employed. In addition, the bolt tends to disintegrate when blasted, removing the hazards of steel cables in the muck stream. They are not recommended, for the same reasons, as support under dynamic loading.

The Type B bolt has a design tensile capacity of 290 kN (Figure 2.9.12) although tests have shown breaking loads of up to 400 kN. The shear capacity is not, however, comparable to steel strand (Figure 2.9.13). The pullout performance of a single Type B bolt is comparable to a double (twin) plain steel strand (Figure 2.9.14).

These bolts are currently available in North America and are being used in numerous operations. They can be installed using similar equipment and techniques to steel strand cablebolts. The composite should be chemically tested to ensure compatibility with the milling and refining process.

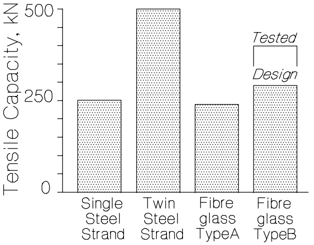


Figure 2.9.12: Tensile capacity comparison (after Pakalnis et al., 1994)

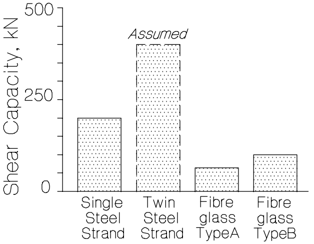


Figure 2.9.13: Shear capacity comparison (after Pakalnis et al., 1994) (Plain strand; Windsor and Thompson, 1993)

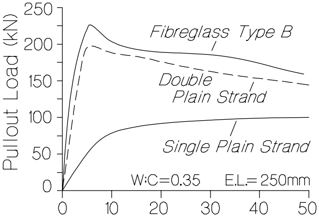


Figure 2.9.14: Pullout performance (after Pakalnis et al., 1994)

2.10 Installation Configuration

The "configuration" of the installation includes the grout water:cement ratio, cablebolt installation method and borehole diameter.

2.10.1 Grout Mix Design Selection

There is an optimum grout mixture for each cablebolt type which represents a compromise between high grout strength (lower grout *W:C*) and good grout flowability (higher grout *W:C*). Specification of the proportions of cement and water in the grout mixture depends upon the type of cablebolt, whether plain or modified, that has been specified in the design. A summary of the factors that should be considered is given in Figure 2.5.9.

Plain strand cablebolts should be installed with maximum strength grout to maximize the bond strength. Since grout strength increases with decreasing water:cement ratio mixtures, the lowest possible water:cement ratio grout that can be readily mixed and pumped, and which will have sufficient water for complete hydration should be used. The grout mixture specified in design for plain strand cablebolts should therefore be between 0.3 and 0.35 *W:C*. If surface fixtures will be installed on the working end of the cablebolts, the grout water:cement ratio can be increased to 0.4.

The performance of modified geometry cablebolts relies on grout completely filling and supporting the cages of the bolts. To ensure that the grout will be fluid enough to flow into the cages, a grout mix design of 0.4 water:cement ratio should be specified in design. Wetter grout (*W:C* > 0.4) will definitely flow into the cages, and will be easier to mix and pump, but will have reduced strength, leading to reduced load carrying capacity of the modified geometry cablebolts.

For additional information regarding flow properties and bond strength relative to grout water:cement ratio, the reader is referred to Sections 2.4 to 2.6.

In summary, the *optimum* grout water:cement ratios are:

0.35 *W:C* for plain strand cablebolts,

and

0.40 *W:C* for modified geometry cablebolts.

2.10.2 Cablebolt Installation Method Selection

Several cablebolt installation methods are used. A brief introduction to these methods has been made in Chapter 1, and a summary of their advantages, disadvantages, and equipment requirements is made in Tables 2.10.1 to 2.10.3. The selection of the cablebolt installation method depends upon the flow characteristics of the grout mixture specified in design, and the orientation of the borehole. The installation method must result in full encapsulation of the cablebolt wires and complete filling of the borehole with design quality grout.

For uphole cablebolt installations, the breather tube or either one of the grout tube installation methods can be used. Grout of $\geq$ 0.375 *W:C* will flow downward under its own weight. Therefore this type of grout will not remain in an unplugged uphole, and must be installed using the breather tube method. On the other hand, thicker grouts of *W:C* $\leq$ 0 375 flow only when pumped. This thicker grout will remain suspended in an uphole and so can be installed using the grout tube method. Very thick grouts $\leq$ 0.35 cannot be effectively pumped using small breather tubes (I.D. < 12mm). Attempting to do so will result in excessive pumping pressure, which is likely to rupture the grout tube, crush the breather tube, or blow out the collar packing. Larger breather tubes (12-20mm) may be used with *W:C* = 0.35 to 0.375 as required provided that they are always filled with grout. The grout tube methods utilize thick grouts exclusively (*W:C* $\leq$ 0.35).

Downhole cablebolt installations are grouted using one of the grout tube methods described in Chapter 1. Grout of *W:C* 0.3 to 0.45 can be specified for downhole installations, so long as it can be easily mixed, can be pumped through the longest grout tube, and is compatible with the cablebolt type. Grouts of *W:C* > 0.45 are never recommended due to insufficient strength.

2.10.3 Borehole Diameter Specification

The borehole diameter specified in design must be large enough to allow easy insertion of the cablebolt and tube(s) into the borehole, but as small as possible to minimize drilling and grouting costs. Suggested *minimum* borehole sizes for different types of cablebolt elements are shown in Figures 2.10.1 and 2.10.2. Figures 2.10 3 and 2.10.4 can also be used to assist in the selection of the borehole size for different combinations of cablebolts, tubes and spacers. The borehole should be large enough to provide adequate space for easy insertion of the cablebolt element (strand, tubes and spacers) into the borehole and grout flow around the element. If the cablebolts stick or jam in the hole during placement, the tubes can be kinked or crushed. In this case use a larger diameter borehole. In a very fractured rockmass, the pieces of rock surrounding the borehole may shift, making it difficult to insert the cablebolt into the hole. In this case, the borehole diameter should be increased, and the cablebolt should be installed and grouted as soon as possible after the hole is drilled.

Table 2.10.1:

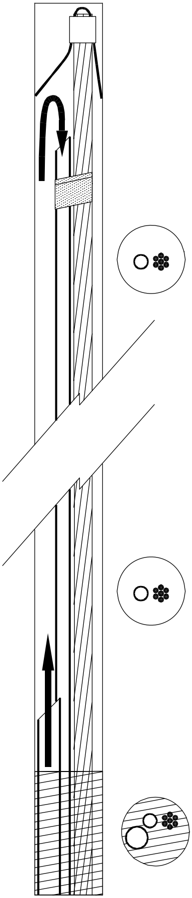
Breather tube installation method (for upholes only)		
	Grout flow	Upward against gravity. Grout flows upward in the borehole, and then returns to the collar of the borehole inside the breather tube.
	Grout mix design	0.4 water:cement ratio.
	Grouting materials	1 metre of ≥3/4" inside diameter (I.D.) grout tube inside the hole at the collar + ≥1/2" I.D. breather tube to the end of the borehole. Collar packing materials are required.
	Associated cablebolt hardware	Plain strand and all types of modified geometry cablebolts can be grouted with this method.
	Grout pump selection	A piston pump is commonly used with this method. A progressing cavity pump can also be used.
	Advantages	The flow of the grout upward inside the hole, against gravity, results in the formation of a complete grout column without voids (except in very fractured rock; see below). This method should be used for grouting multiple strand or modified geometry cablebolts, so that the grout will flow completely around the wires of the cablebolts. The breather tube is of smaller diameter than a grout tube, and so a smaller borehole can be drilled.
	Disadvantages	The encapsulation of the cablebolt can only be guaranteed if grout returns along the breather tube to the collar of the hole. Blocked or crushed breather tubes or flow of grout away into the rockmass through fractures will prevent the return of grout down the breather tube, leaving a void in the column. The pressure required to force the grout back down a small breather tube may lead to burst grout tubes, crushing of the breather tube, or blow out of the collar packing, and hence a partially ungrouted column. Collar packing and borehole grouting are usually done on two passes, increasing labour costs.

Table 2.10.2:

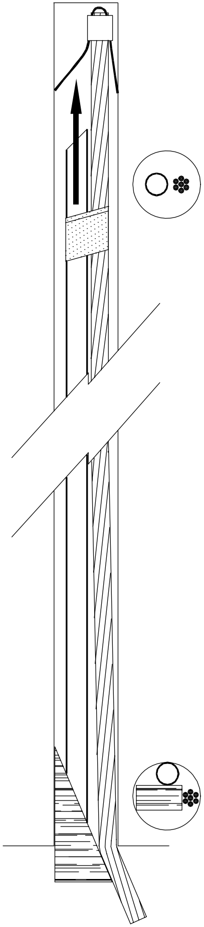
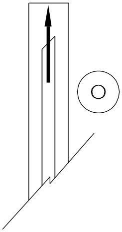
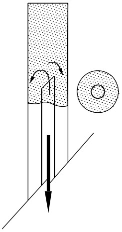
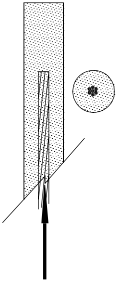
Grout tube installation method		
	Grout flow	Downholes: Upward against gravity. Upholes: Up inside the grout tube, and then downward with gravity inside the borehole.
	Grout mix design	Downholes: Any water cement ratio (0.3 to 0 45) that is compatible with the type of cablebolt being installed. Upholes: 0 35 water:cement ratio.
	Grouting materials	≥3/4" inside diameter (I D.) grout tube to the end of the borehole.
	Associated cablebolt hardware	Single plain strand cablebolts work best with this grouting method in upholes. Usage of this method with modified geometry or multiple strands may result in encapsulation problems, in which portions of the column or the cages are not filled. This can be investigated with pipe pumping tests (Section 2.12).
	Grout pump selection	A continuous feed or progressing cavity pump should be used with this method. Ideally a piston pump should not be used with this method, since the grout front progresses in a "stop-start" fashion, likely preventing formation of a uniform grout flow front. This may introduce voids into the grout column or result in freeze-up of grout in the tubes preventing complete grouting of the hole.
	Advantages	Grout with *W:C* > 0.38 will not remain in upholes, but will run out. The incompletely grouted cablebolt holes should then be evident during subsequent inspection. The higher strength of the thicker grout pumped with this method will increase the cablebolt capacity.
	Disadvantages	Full encapsulation of the cablebolt is not guaranteed, and may only be partial if the grout front separates into "tongues". If the grout falls in "blobs", but remains inside the uphole, the ungrouted hole may appear to be full of grout.

Table 2.10.3:

<table>
<tr><th colspan="3">Grout and Retract installation method (cable inserted before or after grouting)</th></tr>
<tr><td rowspan="7">
Grout tube inserted to toe of hole

Grout tube retracted as grout is pumped

Cablebolt is inserted into grout filled hole</td><td>Grout flow</td><td>In this method, the grout flows within the grout tube to the desired position in the borehole. If the tube retraction is too slow, the grout will flow a short distance within the borehole.</td></tr>
<tr><td>Grout mix design</td><td>Downholes: Any water:cement ratio that is compatible with the type of cablebolt.
Upholes: 0.35 water cement ratio.</td></tr>
<tr><td>Grouting materials</td><td>≥3/4" inside diameter (I.D.) grout tube to the borehole end. The tube is retracted and can be reused.</td></tr>
<tr><td>Associated cablebolt hardware</td><td>The authors' experience with this method is limited to plain strand cablebolts. However, it is thought that the pressure created in the grout column as the cablebolt is inserted into the hole should result in full encapsulation of multiple or modified geometry strands. Full grout encapsulation can be investigated using pipe pumping tests (Section 2.12).</td></tr>
<tr><td>Grout pump selection</td><td>A progressing cavity pump should be used with this method. A piston pump can also be used with this method, so long as it is powerful enough to pump grout to the end of the longest borehole in the pattern.</td></tr>
<tr><td>Advantages</td><td>Grout with $W:C > 0.38$ will not remain in upholes, but will run out. The poorly grouted, empty cablebolt holes should then be evident during subsequent inspection.
The higher strength of the thicker grout that can be pumped with this method increases cable capacity.</td></tr>
<tr><td>Disadvantages</td><td>Voids can be left in the borehole if the grout tube is retracted too quickly, or if the grout is too wet and falls in "blobs" down inside the hole. Some grout will be displaced from the hole as the cablebolt is inserted. However, if grout continues to fall or flow from the hole after the cablebolt has been placed, there are likely to be voids in the column.</td></tr>
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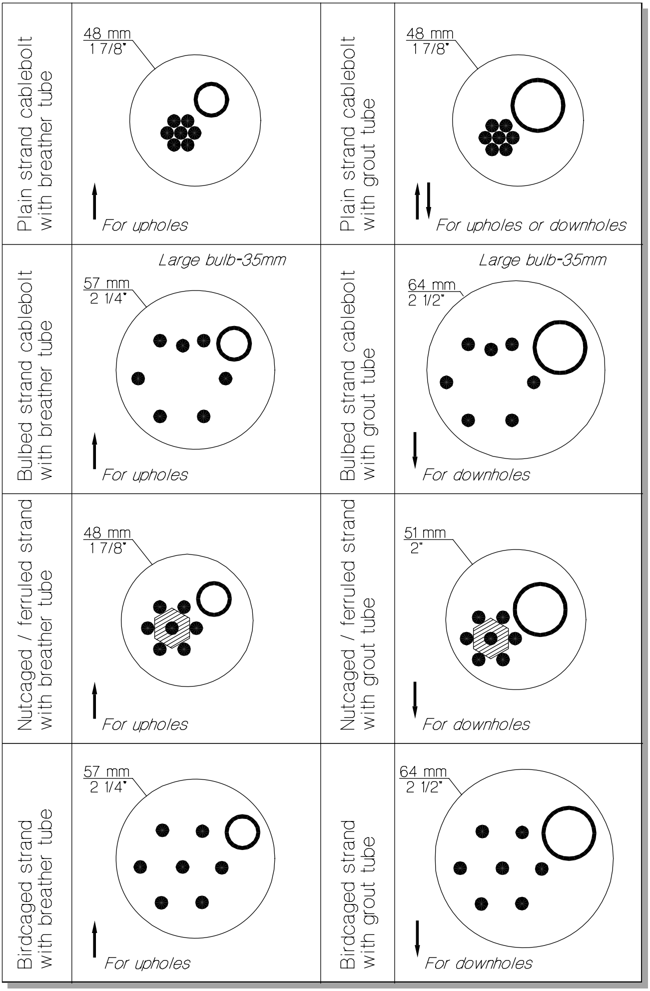


Figure 2.10.1: *Minimum* recommended borehole sizes for single strand cablebolts. The minimum recommended tube sizes are : Breather tube = 10 mm I.D.; Grout tube = 17 mm I.D. Increase the borehole size if undue resistance is encountered when placing the cablebolt element.

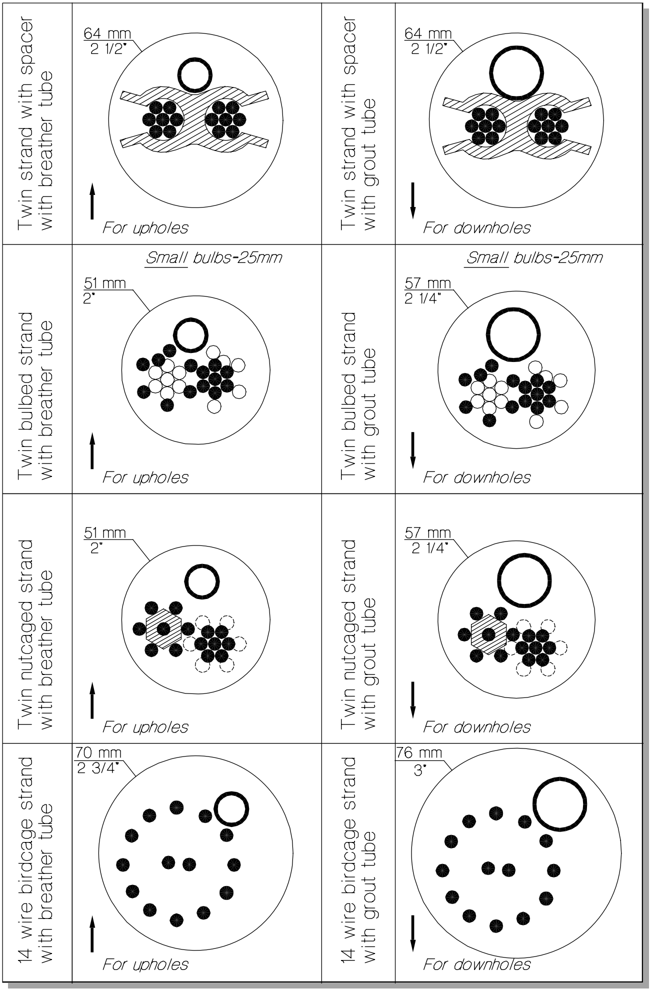


Figure 2.10.2: *Minimum* recommended borehole sizes for twin strand cablebolts. The cablebolt wires shown as open circles indicate the position of the next cage in the offset strands. Tube sizes shown are the minimum recommended: Breather tube=10 mm I.D.; Grout tube=17 mm.

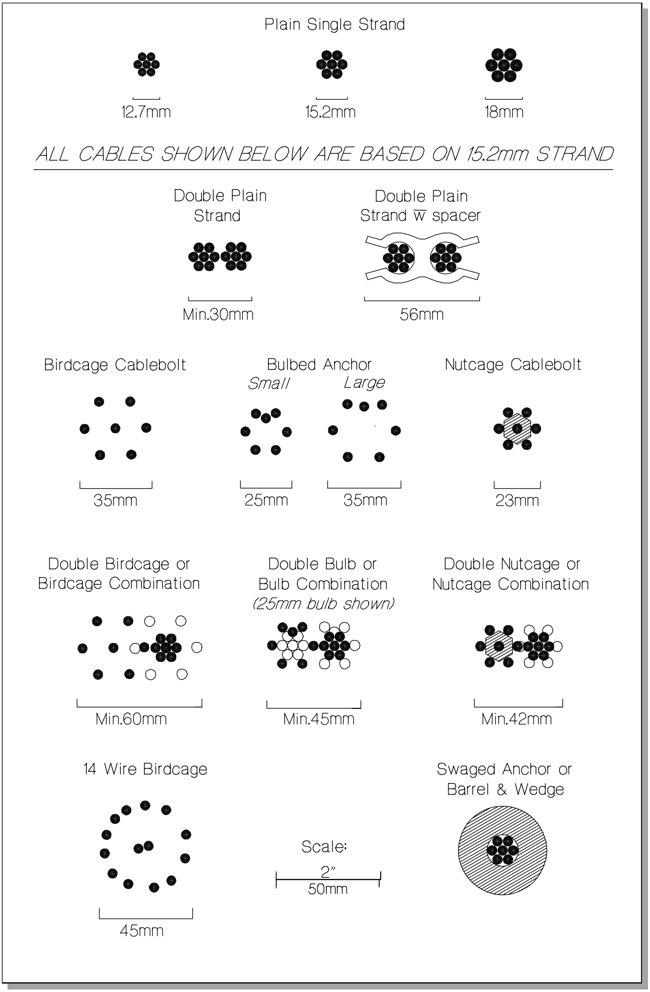


Figure 2.10.3: Cablebolt strand geometry. The cablebolt wires shown as open circles indicate the position of the strand in the next, offset cage along the modified geometry. Check that the dimensions shown are correct for the specific cablebolts on the site.

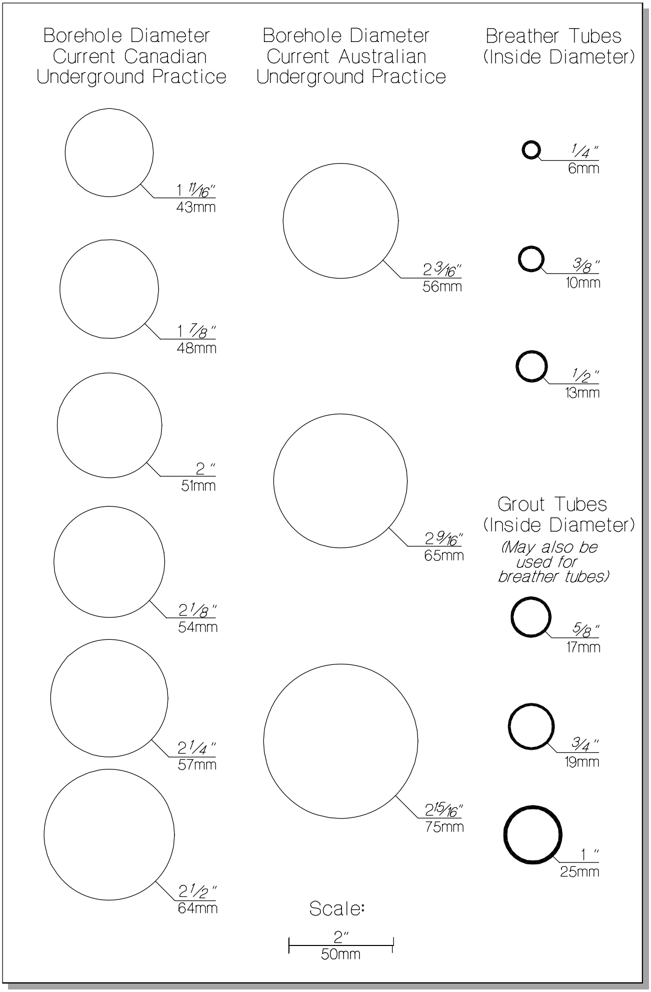


Figure 2.10.4: Borehole and tube geometry. Note that in some applications, where grout flow is unduly restricted by normal sized breather tubes, grout tubing is used for the breather tube. Add any hole or tube sizes in use on the site that are not shown here.

2.11 Selection of Installation Equipment

The equipment used in the installation of cablebolts has the following requirements:

- The drilling equipment must be able to drill boreholes of the maximum length and diameter specified in the design. The cablebolt element (cablebolt strand(s), spacers and tubes) must fit easily into the borehole.

- The grout mixer must deliver well mixed batches of grout of the specified water:cement ratio in a reasonable amount of time. The possible access constraints of the site must be considered when selecting the pump. For example a large, heavy mixer should not be chosen for a work place with limited or difficult access, such as a drift only accessible by a small raise. On the other hand, in cases where all working areas are accessible via drifts or ramps, large twin hopper mixers could be used, as long as equipment to transport the mixer is readily available.

- The grout pump must be powerful enough to completely fill the longest borehole with grout of the design water:cement ratio. As with the mixer, the equipment must be portable enough to be easily moved into the work place with the most difficult access at the mine site.

2.11.1 Drilling Equipment

The complete description of the specifications of drilling equipment for use in cablebolt installations is outside the scope of this handbook. However, the following few points indicate some of the requirements of the drilling equipment for cablebolting.

- The drilling equipment should be able to drill holes with reasonable accuracy. A rule of thumb for accuracy is that the end of the holes (20 to 25 metres long) be within 0.25 metres of the design position. The tolerance for borehole deviation may be even tighter for cut and fill operations.

- The equipment must be able to drill and clear cuttings from the maximum length and most extreme angle of borehole that will be used in the cablebolt pattern.

- The drilling mechanism and bits least likely to damage the wall rock of the boreholes should be used. Failure of the rock around the cablebolt hole will reduce the confinement provided by the borehole and thus the capacity of the cablebolt.

2.11.2 Grouting Equipment

There are numerous grout mixers and pumps available on the market with a wide variety of options for portability, batch mixing speed, pumping mechanism and power, ease of clean up and maintenance, and cost. The grouting equipment selected for each mine site must be powerful enough to mix the specified grout mix quickly and completely, and to pump the maximum length hole full of grout.

It should be possible to find the best equipment for the site by investigating the equipment available from different suppliers. The search for the best grouting equipment could include the products available from cablebolt, shotcrete, mining and civil engineering suppliers. If the capabilities of the mixer and pump are not well known for the grout consistency to be used in your application, then the supplier should give a demonstration of the operation of the equipment, perhaps by conducting a pipe pumping test that simulates the length and diameter of the boreholes at the site. Instructions for pipe pumping tests are given in Section 2.12.

The term "grout" used in civil engineering literature usually refers to $W{:}C$ ratios ranging between 1 and 7. The much thicker grout mixes used with cablebolts ($W{:}C = 0.35$ to 0.4) require increased mixing and pumping power and are more "sticky" and abrasive, leading to more frequent replacement of machine components and the requirement for more thorough clean up procedures.

Equipment Portability

The grout mixer and pump must be portable and robust enough to permit frequent moves. If access to the working areas is limited, such as in some cut and fill operations, or in captive drifts, then the size and weight of the equipment will be limited to what the crew members can readily move by hand. The equipment must also be robust enough to survive any possible rough handling during transport between working areas. In mines with good access, a larger pump may be mounted on a vehicle for easy transport and to protect the equipment from damage.

Grout Mixer Selection Considerations

The mixer selected must be able to completely mix a given volume of design consistency grout in a reasonable amount of time. The phrase "shear mixer" is often used to describe the mechanism of a particular mixer. True shear mixing creates velocity differentials within the grout mix, so that the grout is not simply spun in the bin, but undergoes radial and axial convection, creating a more thoroughly and completely mixed grout.

The grout mixers currently in use in cablebolting applications include drum mixers, colloidal mixers and paddle mixers.

Drum Mixers

Drum mixing equipment is commonly used by the construction industry to mix small volumes of concrete. The mixing mechanism of a drum mixer is similar to the action of a clothes dryer. The rotation of the bin around a 45 °axis, forces the material to tumble under the force of gravity from one side of the bin to the other. In concrete applications, the tumbling aggregate assists the mixing of the water and cement. Aggregate is not used in cablebolt grout mixes however, making this a very inefficient, ineffective mechanism for complete mixing of cement grout. As can be seen in Figure 2.11.1, a drum mixer is likely to produce weaker grout for a given water:cement ratio than will a paddle mixer.

Colloidal Mixers

In colloidal mixers, a small diameter, vortex rotor creates high speed, turbulent flow. The shape of the mixing chamber or bin is carefully designed to promote mixing as well. Complete wetting of even the finest particles of cement (colloidal mixing) is achieved in a very short time.

Colloidal mixers have not been observed by the authors in cablebolt grouting operations. These mixers are used for shotcrete in underground mines, and should be readily useable in cablebolting applications.

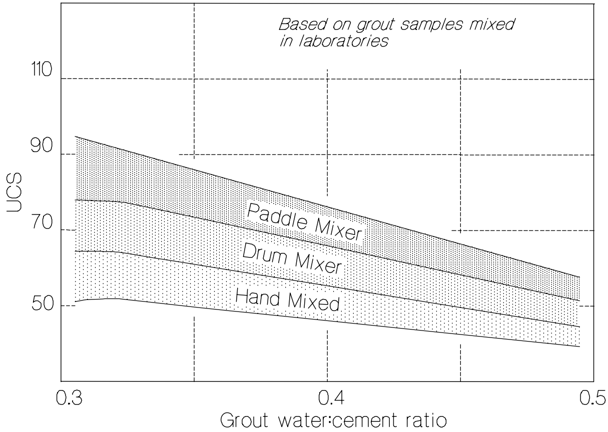


Figure 2.11.1: Effect of mixing method on grout strength from samples mixed in the laboratory (after Gendron et al, 1992).

Paddle Mixers

Paddle mixers are the most common type of mixer used in cablebolting applications. The paddles are attached to a shaft which rotates around a horizontal or vertical axis within the bin. If the paddles are rotated too slowly, or if the paddles are poorly designed, they will pass through the grout mixture, not creating any shear movement, and will produce a poorly mixed grout. Baffles may be attached to the walls of the bin to promote shear mixing within the grout, however dry cement or pockets of water may collect behind poorly designed baffles. The grout should be mixed until the consistency does not change with further mixing. The degree of mixing is especially critical when using a piston pump, since little additional grout mixing takes place once the grout leaves the bin.

Some guidelines for selecting a paddle mixer for cablebolt grout mixing are:

- A bin cover which cuts and supports the cement bags is very useful. The bags can be cut by a serrated blade or cone which is placed on top of a layer of screen. The screen should be small enough to remove any pre-hydrated lumps of cement, but not so fine that excessive dust is created. The screen should be positioned on the cover so that the dry cement is added to the region of highest mixture flow velocity within the bin (away from the centre and side walls).

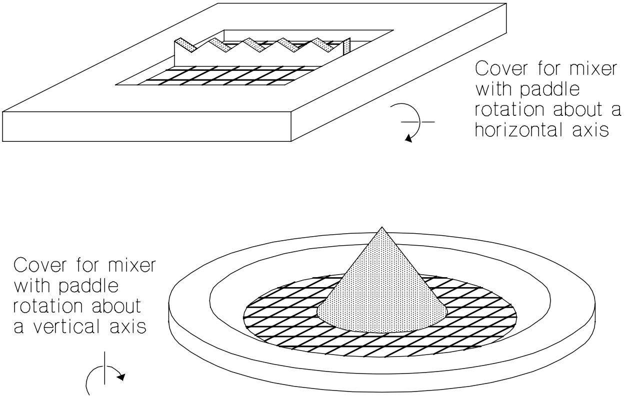


Figure 2.11.2: Possible grout mixer bin lids which will cut and support the cement bag. The screen is intended to remove any lumps from the dry powder.

- The mixer should be designed so that the grout flow moves in a turbulent manner throughout the mixing bin, and does not spin around at the same elevation. This can be achieved by well designed blades and baffles that are placed at an angle to the direction of rotation. Mixing is aided by recirculation of the grout through the pump and grout hose and back into the bin.

- Rounded blades or flat blades aligned perpendicular to the direction of rotation are unacceptable, since they simply pass through the grout without inducing any internal shearing of the cement and water. Flat blades placed at an angle to the direction of rotation should be used.

- Blades (usually replaceable rubber strips) which scrape around the walls and floor of the bin can be used to remove any pockets of dry cement.

- Dead spaces within the bin, where cement powder or water can collect without becoming part of the grout mix, should be avoided in the mixer design. Dead spaces can be created behind baffles which are located at 90° to the direction of the grout flow. Therefore baffles should only be used if they can be designed to increase the shearing of the mix without creating dead spaces. Where they are used, the baffles should be angled to the direction of the grout flow. Dead spaces can also be created around the shaft of the mixer, where the grout velocity is lowest. Therefore the paddles should be spaced out near the walls of the bin and close to the shaft as well. Dry cement should not be added along the bin walls or at the shaft.

- The shape of the mixing bin is also important. Any sharp angle changes in the surface of the bin, such as at the edge of any embayments, may collect unmixed water or dry cement. Therefore the interior surface of the bin should be as streamlined as possible to promote optimum grout flow and mixing.

- The bin should be provided with a convenient and easy mechanism for tipping or pouring the grout mixture into the pump hopper.

- The bin should be easy to clean, with easy access to the paddle blades and baffles. The interior surface of some mixer bins are plastic coated to reduce the chance of grout sticking to the walls or base.

- The size of the bin should provide reasonable batch volumes. See the discussion on the following page regarding the selection of batch size.

- The correct water:cement ratio of the mixed grout is essential for optimal performance of the cablebolt and for use with the particular installation method. Some mixers provide a water metering device, which can help to control the grout water:cement ratio. Water is always placed in the mixing bin first, and then the dry grout is added slowly.

Grout Mixing

Grout must be completely mixed in batches to ensure that all of the grout pumped into the holes is of uniform quality and strength. Mixing is complete when there is no further change in the consistency of the grout and the surface of the mixture appears smooth. Some mine sites advocate continuous mixing to speed up the grouting process, however tests conducted on grout sampled from continuous mixes at mine sites have demonstrated a wide variability in strength.

The best configuration for efficient cablebolt grouting is a double batch system with two mixing bins that keep the pump continuously supplied with grout.

The optimum size of a grout batch can be determined through trials with the cablebolting crew. The batch must be:

- mixed from a full number of cement bags (usually bags are 25 or 40 kg.)
- small enough to be easily mixed, and used up before grout set starts. Generally, each batch should be used up within 15 minutes (0.35 *W:C*) to 30 minutes (0.4 *W:C*) from the end of mixing.
- able to fill a complete number of boreholes. Figure 2.11.3 can be used to estimate the volume of grout required to fill a borehole. Partial hole grouting from a single batch should be avoided, because if the next batch is delayed, the grout will set up in the tubes and the cablebolt hole will be lost.
- large enough that the cablebolt crew is kept well supplied with grout. Long mixing times will hold up the grouting operation, resulting in reduced productivity of the cablebolting crew, and more frequent cleaning of the pump.

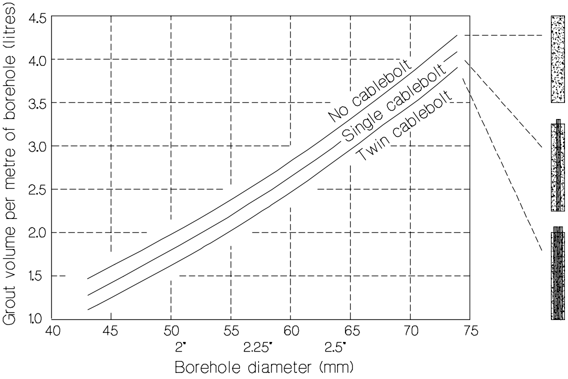


Figure 2.11.3: Grout volume required per unit length of cablebolt borehole.

Grout Pump Selection Considerations

The grout pump must be able to pump the thickest grout into the longest cablebolt borehole that will be used on the site. The grout pump manufacturers usually provide pumping specifications for their equipment that indicate the maximum distance grout of a specific water:cement ratio can be pumped through a grout hose of a given diameter. The pumping specification must exceed the most difficult pumping conditions expected on the site. If several rings of boreholes will be grouted from a central position, the extra length of grout tube from the pump to the furthest hole collar should be added to the required grouting distance.

If there is any question about the ability of the pump to do the job, request that the supplier demonstrate the performance of the pump in a pipe pumping test that simulates the worst conditions expected at the site (longest hole, smallest diameter tube, longest length of tube outside the hole and thickest grout).

A "recirculation" valve which diverts the grout flow through the grout hose and back into the hopper is very useful. In this way continuous grout flow within the moving parts of the pump is possible when the crew are between cablebolt holes, or if the installation is stalled for any reason. Continued agitation or mixing of the grout within the pump hopper will help prevent segregation or settling of the cement particles out of the mix.

Increasing the volume of water used in the grout mixture will enable a longer borehole to be grouted with a given pump. This is due to the reduced frictional resistance to flow between the borehole or tube walls and the wetter grout. On the other hand an increase in the water:cement ratio will reduce the bond strength of the cablebolt and may prevent the use of the grout tube installation method in upholes.

Some suppliers suggest that flushing the pump with clear water is adequate for cleaning. However thicker grouts ($W{:}C \leq 0.4$) may stick to the metal surfaces inside the pump even after flushing. If the pump is not thoroughly cleaned, lumps of hardened cement may block the grout pump tubes or installation tubes, and may even damage the pump itself. Therefore it should be possible to take the pump apart quickly and easily for cleaning after the grouting is complete.

Other factors to consider when selecting a grout pump are the outside dimensions, the weight, the total operating weight, the power system, the air requirements and the portability of the equipment.

Grout pumps commonly used in cablebolting can be classed into two groups on the basis of their pumping mechanism: piston pumps and progressing cavity pumps.

Piston Pumps

The pumping action of a piston pump is shown in Figure 2.11.4. In a single-acting piston pump, grout is pushed into the grout hose on the up-stroke alone. Double-acting pumps push grout on both strokes. The capabilities of piston pumps vary widely, depending upon the design of the piston, valves, and the pump chamber, the consistency of the grout, and the power of the motor. Mine site observations indicate that a piston pump (Spedel 6000) can fill a 12 metre long hole with 0.4 *W:C* grout, using the breather tube method (Nickson, 1992). Piston pumps are generally very portable, easy to clean and easy to maintain.

The grout batch should be agitated or mixed throughout the time that the grout is being pumped to prevent the solid cement particles from settling out of the mix. Some piston pumps are placed directly into the grout mixing bin, while others are supplied with a separate bin from which the grout is pumped. In both cases, a paddle mixer is usually used to keep the cement in suspension through out the grouting process, and the pump is placed into a side chamber of the bin.

The pulsing action of a piston pump may prevent the formation of a consistent, uniform flow front. If this is the case, the piston pump should not be used for the top down, grout tube installation method for upholes.

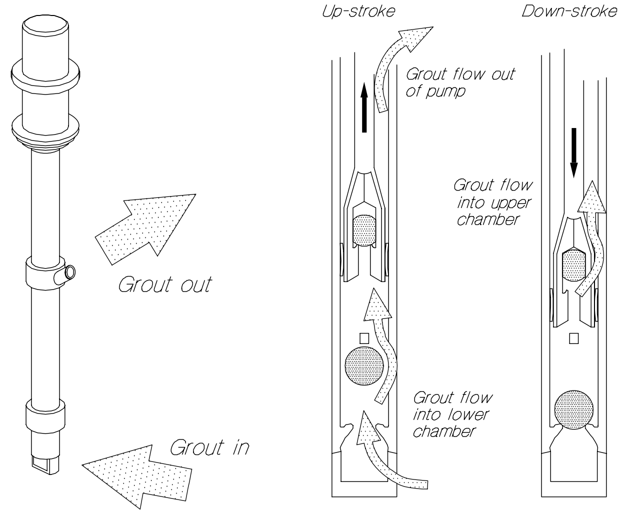


Figure 2.11.4: Configuration and pumping mechanism of a single-acting piston pump

Progressing Cavity Pumps

The rotor and stator are the two key elements of a progressing cavity or eccentric screw pump. The rotor is a single pitch steel spiral with circular cross section and a degree of eccentricity. The stator is a double pitch internal spiral with pitches at 180°. As the rotor turns, it rotates concentrically around its own axis, and moves eccentrically as well, creating both a complete seal along the length of the stator at all times and cavities which progress continuously (in a non-pulsating manner) along the length of the pump.

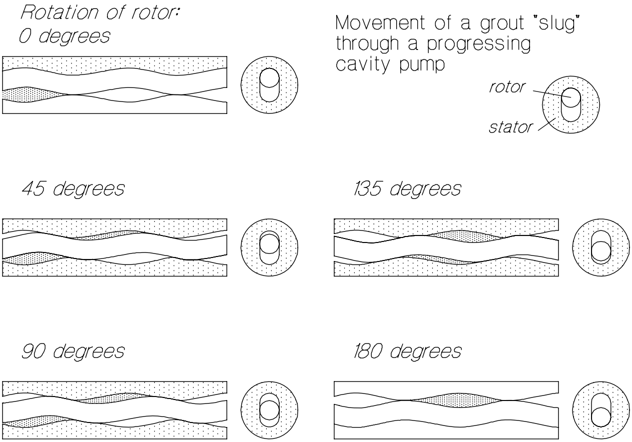
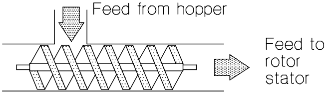


Figure 2.11.5: Progressing cavity pumping mechanism

In many of the progressing cavity pumps currently available, the grout is fed into the rotor/stator assembly by a horizontal auger located at the bottom of the grout hopper.



The pumping pressure developed in a progressing cavity pump relies upon an absolute seal between the suction and pressure side of the pump in any position of the eccentric screw. If the rotor is damaged, or the rubber stator begins to wear out and enlarge, pumping pressure will be lost. At that time, replace the stator.

In field observations of progressing cavity pumps an 18 metre long uphole was pumped full of 0.35 *W:C* grout using the grout tube method (Nickson, 1992).

2.11.3 Breather and Grout Tubes

The selection of appropriate breather and grout tubes is important. The cost of the tubing is a very small portion of the total cost of a cablebolt installation, and so the tubes are usually overlooked in the design of and material specification for a cablebolt system.

The diameter and pressure rating of the tubes must be great enough to transmit the grout easily. Cluett (1991) observed crushing of 6 mm I D. breather tubes in long holes where excessive pumping pressure was applied to try to force grout back down an undersized breather tube. Rupture of grout tubes during grout tube installations in long holes have been reported at a number of mine sites as well.

The selection of the tubing must take into consideration the sources of resistance to grout flow in the tube(s):

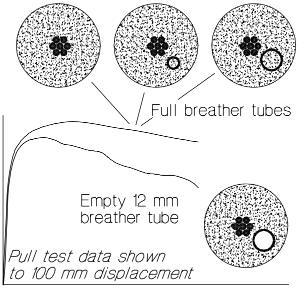
- The grout consistency. As the grout *W:C* is reduced, the frictional resistance to grout flow increases, requiring higher pressure and larger diameter tubing.
- The diameter and length of the tube(s). Increasing resistance to grout flow is created by decreasing tube diameters or increasing tube lengths.
- The number and severity of diameter changes. Frictional losses in the tube(s) can be severe due to diameter changes along the grout flow path as is discussed in Section 2.5.4.

Breather tubes of 11 mm inside diameter (I.D.) and 1.5 to 2 mm wall thickness should be adequate for most cablebolt holes pumped with grout of $W:C \geq 0.4$. In Australian mines breather tubes are often 17mm I.D. Failure of the grout to return back down the breather tube could indicate that the breather tube is too small or that the grout is flowing away into fractures in the rockmass. In the first case, the increase in pressure required to force grout into too small a tube will cause the pump to stall, the collar packing to blow out, or the breather tube to crush or collapse. In this case, increase the diameter of the breather tube. If there is any evidence of crushing of the breather tube, increase the required tube pressure rating. Loss of grout into a fractured rockmass is indicated when an excessive volume of grout has been pumped into a hole, and there has not been any return along the breather tube. In this case, use the procedures for grouting in fractured rock given in Chapter 3.

Grout tubes are usually 17 to 25 mm I D. and 2 to 3 mm wall thickness. The minimum recommended pressure rating for grout tubing is 100 psi. The grout tube may burst in situations where pumping pressure increases as thick grouts are pumped along long lengths of grout tubing or when grout is difficult to pump back down a breather tube (tube too small or grout too thick). In this case, higher pressure rated tubing should be purchased.

The occurrence of problems may prevent complete grouting of the borehole. The ungrouted sections of the cablebolt strand will have no load carrying capacity.

An open, ungrouted 11 mm I.D. breather tube within the grout column (0.45 *W:C*) has been found to reduce the capacity of a plain strand cablebolt by 30% (Goris, 1990: laboratory pull tests conducted on 25 cm long samples confined in steel pipe). On the other hand, Goris found that fully grouted breather tubes do not reduce the capacity of cablebolts.



Note: Influence of empty tube is dependent upon proximity to cable

Figure 2.11.6: Effect of fully grouted and empty breather tubes on cablebolt capacity (after Goris, 1990)

2.11.4 Installation Accessories

Installation accessories include items which are used with the cablebolt strand itself, to help insert the cablebolt into the hole, to support the cablebolt prior to grouting and to block the collar of upholes.

Cablebolt Dispensers

Coils containing a complete uncut length of cablebolt strand are often the cheapest and most convenient way to purchase the strand. If it is properly handled and stored, a coil will provide a large supply of clean strand. Coils of cablebolt strand may be shipped in individual packs which are bound with steel straps, or within refillable dispensing cassettes. Mine sites which use individual coil packs usually manufacture or purchase their own refillable racks or dispensing cassettes.

Dispensing cassettes or racks can be horizontal or vertical, and stationary or rotating. The dispenser should be easy to load. Whatever the configuration of the dispenser, the prime requirement is that the cablebolt does not become tangled and unwound. Guides through which the cablebolt is pulled may help to keep the coil together and untangled. The strand is usually pulled from the centre of the coil in stationary dispensers. The strand will acquire a twist for each loop that is removed from the coil. The twist should be in the direction of the cablebolt lay, so that the cablebolt strand will tighten up instead of unwinding. The strand is usually pulled from the outside of the reel of a rotating dispenser. A brake to stop the rotation of the dispenser should be provided.

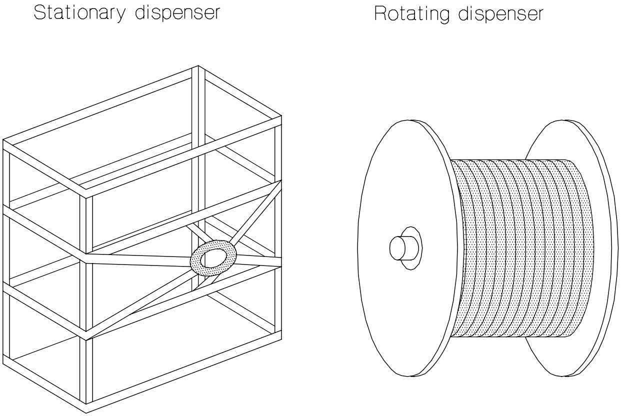


Figure 2.11.7: Possible cablebolt dispensers

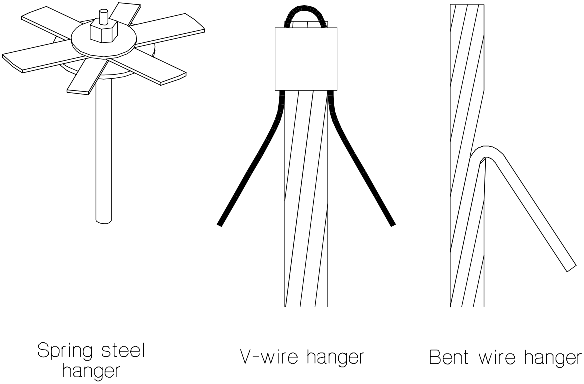


Figure 2.11.8: Cablebolt hangers

Cablebolt Hangers

The cablebolt must be well secured in uphole installations, between the time of placement of the cablebolt and the time of grout set. Some examples of hangers observed in use in underground mines by the authors are shown in Figure 2.11 8. The most important consideration when selecting the hanger or collar wedge for use at a site is that the cablebolt must remain completely secure in the uphole until the grout has set. **A cablebolt falling from an uphole can cause serious injury**. The choice of a particular hanger will depend upon the ease of use and the cost of the hanger, the borehole diameter, the method of inserting the cablebolt in the hole, the grouting method, and prevention of damage to the cablebolt strand and tubes. Longer cablebolts will require stronger hangers. For example extra steel strips should be added to spring steel hangers for long cablebolts. In situ pull tests on cablebolts with hangers can be used to ensure adequate hanger strength.

There are a number of different configurations of hangers and wedges in use at mine sites. Some hangers are formed by bending one wire of the strand, while others are separate items which are attached to the cablebolt. Hangers can be used at either the collar or toe of the hole, and wedges are used at the collar. Toe hangers will help support the cablebolt as it is being inserted into the borehole. Hangers at the collar end of the cablebolt will be easier to insert into the hole, but will not help support the cablebolt during placement.

Some hangers centralize the cablebolt in the borehole, while others push the cablebolt to one side of the hole (offset hanger). In uphole grout tube installations, it is advantageous to use a centralizing hanger to start the cablebolt in the middle of the hole to promote optimum grout encapsulation around the steel strand. When offset hangers such as the bent wire hanger are used, attach tubes that extend to the toe of the hole beneath the overhanging, protective hanger wire.

The diameter of the borehole can be critical to the strength and support capabilities of the hanger. If the borehole diameter is too large, the hanger will not lodge into the rock as well and may not be able to support the cablebolt. If the borehole diameter is too small, it will become difficult to insert the cablebolt into the borehole.

When using wooden wedges at the borehole collar, it is very important that the grout and/or breather tube are not pinched or constricted in any way as this will impede the grout flow.

Hangers made by cutting one wire of the cablebolt strand have deliberately not been included in Figure 2.11.8, because these hangers reduce the capacity of the steel strand (by at least 1/7 or 15%) and lead to eccentric loading of the cablebolt. In addition, there are very few tools available which will cut just one wire of the strand, so the capacity reduction of the cablebolt at the position of the hanger can be much greater than 15%.

Cablebolt Spacers

Cablebolt spacers are often used in Australia when more than one cablebolt is installed within a single borehole. The spacers keep the cablebolts separated and away from the borehole wall, aiding complete grout encapsulation of the strands. In some circumstances, where a single cablebolt should be centralized within the borehole (such as in corrosive environments in which the grout provides protection against deterioration of the steel), spacers may also be used.

Spacers are usually formed of plastic and so are relatively inexpensive. The spacer must snap firmly onto the cablebolt strand(s) so that it will remain in place as the cablebolt element is inserted into the borehole. The edges of the spacer should be rounded to prevent snagging on the borehole wall. The borehole diameter should be ≥ 5 mm greater than the largest cross-sectional dimension of the spacer. The cross-sectional shape of one spacer from Australia is shown in Figure 2.10.2.

Borehole Collar Sealing

Collar sealing is required for uphole grouting installations. The collar can be sealed on a first pass or at the time of grout pumping. Collar sealing materials that are commonly used on a first pass include burlap, cotton waste, expansive foam and grout plugs. Collar sealing can also be done at the time of grouting with a rubber cone or victaulic pipes. Drawings and installation procedures for each of these collar sealing methods are given in Chapter 3.

Grout Tube Connectors

Connectors are required to join the grout pump hose and the installation grout tube during grout pumping. The connector must be easy to fasten and strong enough to withstand the pressure built up in the grout hoses. The most common type of connector consists of a plastic screw cap with a hole in the centre which fits over the grout tube. This cap screws onto a threaded metal or plastic end which is attached to the grout pump hose. Some mine sites have also used vice grips with a modified circular grip to hold the two tubes together.

Cablebolt Cutters

The cablebolt strand supplied in a coil is cut into lengths prior to installation in the borehole. Cablebolt cutting is also required at the face in cut and fill stopes where lengths of the strand are exposed by successive mining lifts. Several methods can be used.

An air powered grinder will cut through a cablebolt strand fairly quickly if the grinder blade is replaced as soon as it gets dull. The grinder can be a hand tool or where possible may be a larger tool which is permanently mounted on a truck or on a working platform in a central stores area. The production of sparks and steel shards when cutting the cablebolt necessitate the use of a face shield and leather gloves by the operator.

An oxyacetylene torch will very quickly burn through the cablebolt strand, but requires a lot of bulky materials including the cutting torch, regulators, hoses and the oxygen and acetylene bottles. The use of pressurized gases and flame will be a safety concern in some mines as well. In addition, the operator must be careful to avoid touching the hot end of the cablebolt for a period of time after cutting.

Hydraulic cutters can be very efficient if they are large enough to cut through the cablebolt strand in one action. The large size and weight of a powerful hydraulic cutter make it practically useable only when it can be mounted on mobile equipment. In this case, the operator can be far enough away from the cablebolt to be removed from any shards or whipping of the cablebolt.

Explosives have been used to cut cablebolts exposed in cut and fill mining. Explosives suitable for cutting cablebolts are available from some suppliers. It must be possible to attach the explosive to the cablebolt safely and for the explosive to cut the strand easily. In addition, the explosive must be readily available and safely useable in the underground working environment.

Cablebolt Pushers

Cablebolt pushers consist of two rotating rollers between which the cablebolt passes. The surface of the rollers should be formed of strong material that will resist tearing and undue wear. The rollers must grip the cablebolt tightly enough and have enough power to push the longest cablebolt into an uphole. Cablebolts slipping within under-designed pushers have been reported at some mine sites. On some pushers, the rollers are individually powered by air motors so that if the cablebolt jams in the hole, the motors will stall and there will be no damage to the unit. For pushers used to place modified geometry cablebolts, the rollers may be mounted on spring loaded arms to accommodate the changes in the diameter of the cablebolt. A pusher mounted on an articulated arm can be easily positioned beneath the borehole collar.

Cablebolt Trucks

The cablebolting operation can be streamlined when a well equipped truck is supplied to the crew. The utility of a cablebolting truck depends on good access to the working areas. Depending upon the specific requirements of each site, the truck can be outfitted with a scissor lift, cablebolt cutter, cablebolt pusher, strand storage platform or coil dispenser, grout mixer and pump, grout storage platform, water supply and lights.

2.12 Pipe Pumping Test Procedures

Pipe pumping tests can be used to investigate the completeness of grout pumping into the cablebolt hole or the capabilities of the equipment, and for training the crew in cablebolt installation procedures. The advantage of pipe pumping tests is that the grout column can be cut apart after the grout has set to investigate the quality and completeness of the grout column.

The materials required for pipe pumping tests are:

1) Pipes. The number, length and diameter of the pipes required will depend upon the objectives of the test, but are usually of the same length and diameter as the cablebolt holes. The pipe material chosen should be robust enough to survive the conditions of the test, but also be within the test budget. Steel pipes are strong, but opaque and hard to cut; PVC pipes are strong but opaque; and clear acrylic pipes are transparent so that the grout flow front can be observed during the test, but brittle and expensive.
2) Pipe joiners for longer test lengths. The joiners should fit over the outside of the pipes and must not impede the flow of the grout.
3) End caps to seal the toe end of the pipes.
4) Grout mixing and pumping equipment.
5) Cement and any additives.
6) Cablebolt strand(s).
7) Breather and /or grout tubes.
8) Hangers, collar wedges, spacers and collar packing materials.
9) When required, cylinders for collecting grout samples for strength or sedimentation tests.

The test procedure should follow these steps:

- Suspend or support the pipe in the orientation of borehole.
- Place the cablebolt following normal procedures as laid out in Chapter 3.
- Mix and pump the grout, ensuring that the grout mix proportions are correct. Note the time at which grouting starts.
- During transparent pipe tests, monitor the flow of the grout within the pipe and note any areas where flow is unduly constricted, or where some of the column is ungrouted. A video and/or photographic record of the tests is very useful for training purposes and for analyzing the test results.
- Make a note of the time that grout first appears at the end of the breather tube or at the collar. If this grout has a watery appearance, keep pumping until grout of the mix consistency begins to flow from the tube or hole collar. Record the time that the pumping continued after the first appearance of grout.
- Kink over and tie off the tube(s).
- Make detailed notes about each test, including information about the *W:C*, mixing time, pumping rate, progress of the grout flow front, and any problems.

To examine the completeness and quality of the grout column, cut sections through the pipe and cablebolt, after the grout has set (> 72 hours). If a modified geometry cablebolt was tested, some of the slices should be made through the flared sections and some through the sections of regular strand.

Some problems which have been observed in pipe pumping tests conducted at mine sites and by the authors are shown in Figure 2.12.1. Photos of sections cut through pipe pumping test samples are given by Goris et al. (1994). Empty voids in the grout column have the potential to reduce the load transfer capability and hence the capacity of the cablebolt in their vicinity to zero. The problems shown here will not always occur at each site, but should be considered if the cablebolts at a site do not perform as expected. Other quality control problems, and the potential influence of stress change on the cablebolt bond strength should also be considered in any investigation of poor cablebolt performance.

If any empty voids are found during the tests, or if incomplete grouting of the cablebolt holes are suspected, the problems could be due to:

- poor centralization of the cablebolt in the hole,
- incomplete grout mixing,
- inappropriate grout water:cement ratio,
- inappropriate grouting method, and
- inadequate breather or grout tube size.

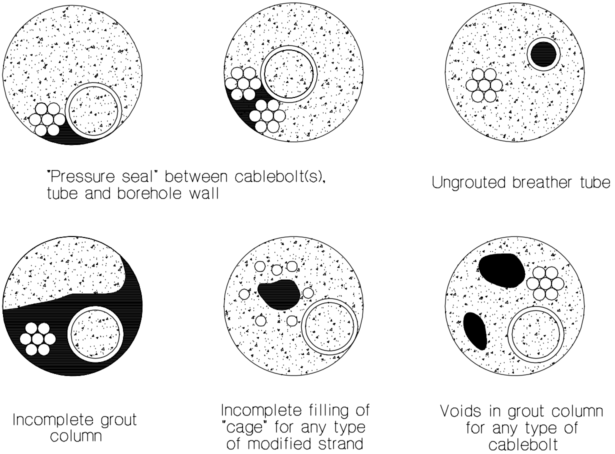


Figure 2.12.1: Grouting problems found in pipe pumping tests conducted by the authors and at a number of mine sites

2.13 Demand

While capacity determination answers the question "What *can* be done ?", demand assessment is the process of determining "What *should* be done ?". Design relates these two questions to arrive at a solution which satisfies both, within the constraints of economics, operational limitations and practical ability.

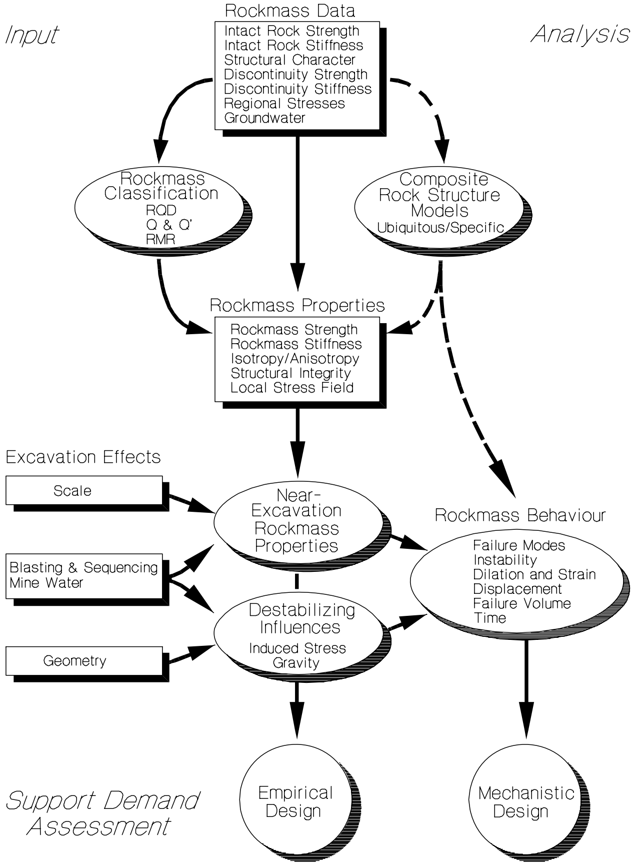


Figure 2.13.1: Demand assessment for support design in underground mines

2.13.1 Excavation Response

Figure 2.13.1 illustrates the basic flow of investigation involved in rigorous demand assessment. The first goal is the determination of rockmass properties and excavation influences. Then, either through empirical (experience based) methods or though mechanistic analysis of rockmass behaviour, stability and support demand can be evaluated. The following sections form a practical summary of this methodology. Readers interested in more detailed rock mechanics are directed to Bouchard (1991), Brady and Brown (1985; 1993), Goodman (1976; 1980), Hoek et al. (1995), Hoek and Brown (1980), Hudson (1989), Jaeger and Cook (1979). Two relationships are central in determining general excavation response characteristics (Figure 2.13.2):

- The ratio of far field or induced stresses to rock strength
- The ratio of block size to excavation dimension

Stress (crudely defined as loading over an area) results from rock loads acting at depth and from tectonic adjustments within the earth. The *stress field* (the variation of stress with orientation over three dimensions) is disturbed by the creation of underground openings. As a result induced stress changes occur which can relax the rock mass (destress) or which can increase the stresses tangent to the boundary to a point close to the strength of the rock. In the latter case, fracture, damage and ultimately disintegration can occur. Stresses are either *compressive* (tending to push in) or *tensile* (tending to pull apart) and have their respective counterparts of compressive and tensile strength. *Shear* (distortional) stress results from oblique combinations of the above *normal* stresses in three dimensions.

Strength is the ability of the rock to withstand elevated levels of stress without sustaining damage. *Yield strength* marks the onset of such damage while *ultimate strength* indicates the maximum limit of stress which can be endured before complete rupture. *Failure* is an arbitrary term which must be qualified with respect to these limits; yielded rock may carry significant load around an excavation.

Block size relates to the average dimension of competent rock blocks created by the intersection of natural breaks in the rock called discontinuities. Joints are natural extension features, while shears are discontinuities which reflect previous or on- going movement (relative slip) between two blocks of rock. All rockmasses possess some discontinuities. The influence of these features on stability of an opening is normally controlled by the relative inter-joint spacing or block size with respect to the dimensions of the excavation. Clearly, rockmasses with closely spaced discontinuities (*Heavily jointed, fractured or broken* rockmasses) are more likely to have stability problems and to require the addition of artificial support, than are relatively *intact* or *massive* rockmasses. A third scenario develops when the rockmass has a moderate concentration of highly persistent (long) discontinuities which mutually intersect in the vicinity of an opening. These joints or shear structures can form large intact blocks, slabs or wedges which can be released into the excavation and which may require support.

Rockmass Behaviour

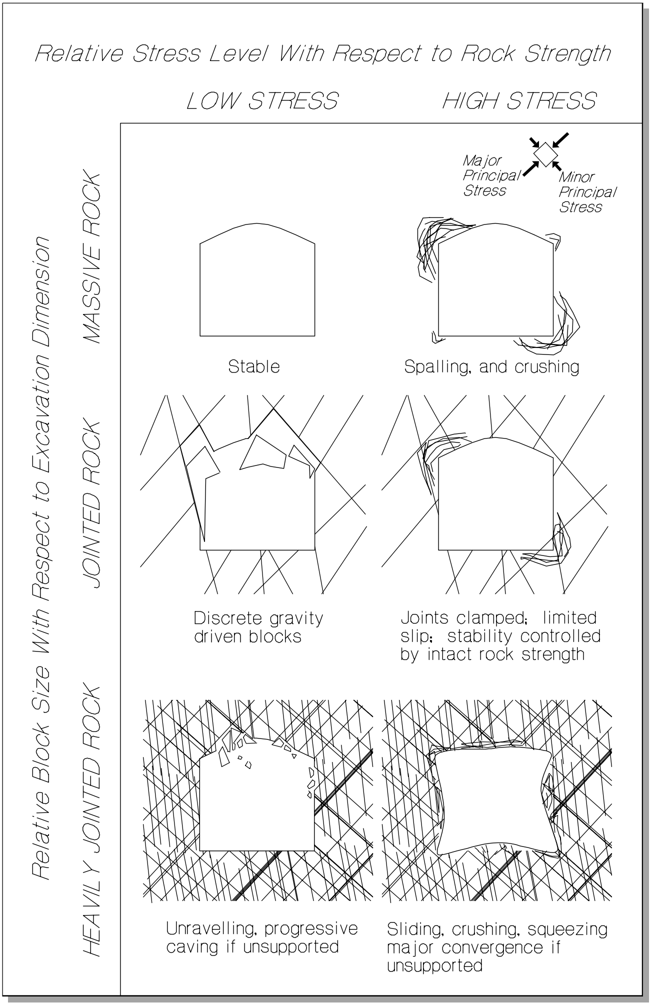


Figure 2.13.2: Stress, structural integrity and failure modes (after Hoek et al., 1995)

2.13.2 Stress - A Brief Introduction

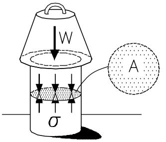


Figure 2.13.3: Stress

Stress in its simplest form can be calculated in a one-dimensional example (Figure 2.13.3), as load divided by the area over which the load acts. Stress acting on a plane can have two components. One is a normal component acting perpendicular to the surface. The other is a shear component acting parallel to the surface (Figure 2.13.4a). When acting on a separation plane between two solid masses, a normal compressive stress will tend to push the two halves together (a negative or tensile stress will pull them apart). A shear stress acting on the separation plane will tend to slide the two halves past each other in opposite directions. Normal compressive stresses acting on a solid will compress or collapse the solid (*compressive strain*) as shown in Figure 2.13.5. Shear stresses will cause an angular distortion (*shear strain*) as shown.

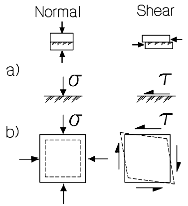


Figure 2.13.4 Stress on a a) Plane b) Volume

Stresses in 3 dimensions are more difficult to visualize. Within a rockmass at depth, stresses act in all directions upon a sample unit volume and are associated with three dimensional deformation of the unit volume (*strain*). These stresses (and the corresponding strains) vary with direction. The mathematical entity used to describe such a state is the stress *tensor* which expresses the three normal stresses and six shear stresses acting on the faces of a fictitious and infinitesimally small cube (placed within the stress field) in three orthogonal directions at specified orientations. While the stress state at a point is unique, the tensorial description depends on the orientation of these reference axes. Figure 2.13.5.a) shows a schematic representation of a stress tensor expressed with respect to the global axes shown. The three sets of coplanar stress components are illustrated in the sections in Figure 2.13.5.b). Note the equality of co-planar shear stresses.

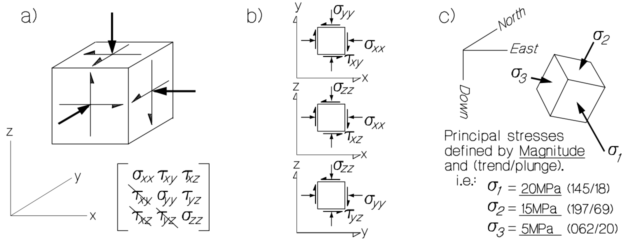


Figure 2.13.5: a) Elemental cube showing tensor convention (+ve directions shown). b) Sections through elemental cube c) Principal stresses with typical notation

For every valid stress state, there exists a unique orientation of the reference cube (Figure 2.13.5.c) at which the shear stresses reduce to zero. The remaining normal stresses are called the principal stresses and are usually quoted as major principal stress σ_1, intermediate principal stress σ_2, and minor principal stress σ_3 (compression positive sign convention). The magnitude and orientation of these principal stresses uniquely defines the state of stress at a point. The orientations are quoted as trend (angle *CW* from North) and plunge (downward angle from the horizontal).

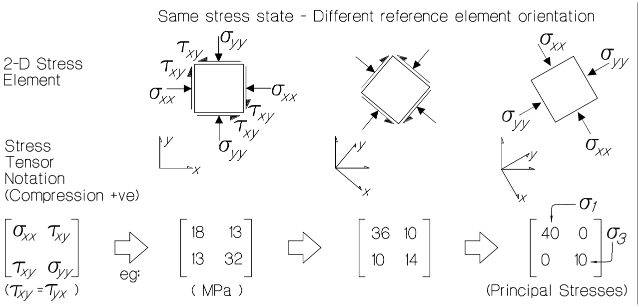


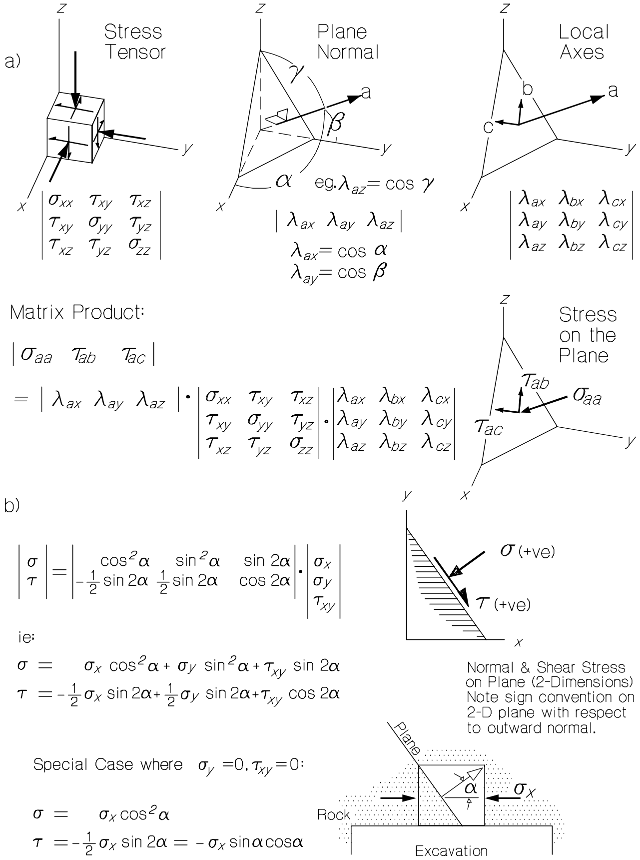
Figure 2.13.6: 2-Dimensional stresses; tensor rotation and principal stresses.

The concept of tensorial stress and principal stress directions and magnitudes is easier illustrated by considering only one of the sections in Figure 2.13.5.b) and ignoring all out of plane stresses (Figure 2.13.6). The two-dimensional stress state shown does not change as the square reference element is rotated. Only the tensor description changes - hence both the tensor magnitudes and reference orientation must be specified when describing a stress state in this way. For a certain orientation (at the right of Figure 2.13.6) the shear stresses vanish. The resultant normal stresses are the *principal stresses* (in 2-D denoted by σ_1 and σ_3). The orientation of the principal axes must be specified when quoting principal stresses.

Note that this kind of two-dimensional analysis is not normally valid in a three dimensional world. The individual components or directions of a three-dimensional tensor cannot be considered separately and independently in this fashion. This is the nature of a tensor. Two-dimensional analysis is only valid in circumstances where the excavation geometry is long in one direction (e.g. a tunnel) and one of the principal in situ stress components is aligned with the tunnel axis or out-of plane direction. In this case, there are no out-of-plane shear stresses and the analysis need only consider the in-plane components. The out-of-plane direction is confined (no induced strain) in this *plane-strain* case. The out-of-plane normal stress will change but can be ignored in most elastic analyses (no rock yield) that meet the geometric conditions.

Stresses on a plane

There are applications where it is necessary to evaluate the stresses on a plane such as a joint or fault within a stress field, in order to assess the potential for slip or dilation. Figure 2.13.7 a shows the tensor arithmetic necessary to do so in three dimensions. Figure 2.13.7.b gives the procedure for two-dimensional calculations.



$$\begin{vmatrix}\sigma_{xx} & \tau_{xy} & \tau_{xz}\\ \tau_{xy} & \sigma_{yy} & \tau_{yz}\\ \tau_{xz} & \tau_{yz} & \sigma_{zz}\end{vmatrix}$$

eg. $\lambda_{az} = \cos\gamma$

$$\begin{vmatrix}\lambda_{ax} & \lambda_{ay} & \lambda_{az}\end{vmatrix}$$

$$\lambda_{ax} = \cos\alpha$$
$$\lambda_{ay} = \cos\beta$$

$$\begin{vmatrix}\lambda_{ax} & \lambda_{bx} & \lambda_{cx}\\ \lambda_{ay} & \lambda_{by} & \lambda_{cy}\\ \lambda_{az} & \lambda_{bz} & \lambda_{cz}\end{vmatrix}$$

Matrix Product:

$$\begin{vmatrix}\sigma_{aa} & \tau_{ab} & \tau_{ac}\end{vmatrix} = \begin{vmatrix}\lambda_{ax} & \lambda_{ay} & \lambda_{az}\end{vmatrix} \cdot \begin{vmatrix}\sigma_{xx} & \tau_{xy} & \tau_{xz}\\ \tau_{xy} & \sigma_{yy} & \tau_{yz}\\ \tau_{xz} & \tau_{yz} & \sigma_{zz}\end{vmatrix} \cdot \begin{vmatrix}\lambda_{ax} & \lambda_{bx} & \lambda_{cx}\\ \lambda_{ay} & \lambda_{by} & \lambda_{cy}\\ \lambda_{az} & \lambda_{bz} & \lambda_{cz}\end{vmatrix}$$

$$\begin{vmatrix}\sigma\\ \tau\end{vmatrix} = \begin{vmatrix}\cos^2\alpha & \sin^2\alpha & \sin 2\alpha\\ -\frac{1}{2}\sin 2\alpha & \frac{1}{2}\sin 2\alpha & \cos 2\alpha\end{vmatrix} \cdot \begin{vmatrix}\sigma_x\\ \sigma_y\\ \tau_{xy}\end{vmatrix}$$

ie:

$$\sigma = \sigma_x \cos^2\alpha + \sigma_y \sin^2\alpha + \tau_{xy} \sin 2\alpha$$
$$\tau = -\frac{1}{2}\sigma_x \sin 2\alpha + \frac{1}{2}\sigma_y \sin 2\alpha + \tau_{xy} \cos 2\alpha$$

Normal & Shear Stress on Plane (2-Dimensions) Note sign convention on 2-D plane with respect to outward normal.

Special Case where $\sigma_y = 0, \tau_{xy} = 0$:

$$\sigma = \sigma_x \cos^2\alpha$$
$$\tau = -\frac{1}{2}\sigma_x \sin 2\alpha = -\sigma_x \sin\alpha\cos\alpha$$

Figure 2.13.7: Calculation of stresses on a plane a) 3-D; b) 2-D

In Situ and Induced Stresses - Modelling

Before modelling, it is essential to establish the existing state of stress. The initial stresses can be due to sedimentation or volcanism as well as tectonic movements and pressures. The stress state could have been further modified through folding, faulting, metamorphism, or erosion. Typical vertical stresses in MPa range from 0.025 to 0.03 times the depth in metres. In a tectonically inert, non-eroded basin (e g. South African gold fields) the horizontal stresses at depth may be one-half of the vertical. Near surface, where horizontal stresses are locked in while vertical stresses are relieved by erosion of overburden, or in areas of high lateral pressure (e.g. Canadian Shield) the horizontal stresses can vary from 1.5 to more than 3 times the vertical stress. All three principal stresses may be significantly different and may vary from location to location. Regional structure (faults and dykes) will often cause significant stress field disturbance. While several general guidelines for stress estimation are available (Hoek and Brown, 1980; Herget, 1988, Hoek et al., 1995; Zoback, 1992), it is still prudent to obtain a local measurement of in situ stress (Herget, 1988). This initial stress tensor will completely determine the induced stresses obtained through excavation analysis.

Excavations disturb the in situ stress field. An analogue to induced stress flow around an excavation is the flow of river water around a bridge post. The stresses build up (concentrate) on the sides of the excavation parallel to direction of σ_1 (Figure 2.13.8.a&b) just like the water around the post. Where the difference between major and minor principal stress is high, relaxation zones or even tension may develop on the faces which lie parallel to $\sigma_{3.}$ Typical contours of maximum induced principal stress are shown in Figure 2.13.8c. Note the convention for stress trajectories. The small ticks on the crosses represent the direction of σ_3 while the long ticks represent σ_1. The stresses at right angles to and adjacent to the walls of the excavation are zero in the absence of support or an internal pressure.

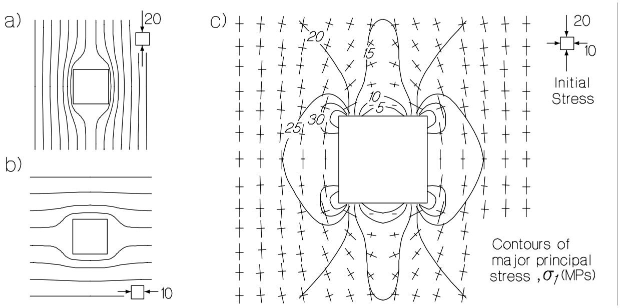


Figure 2.13.8: Induced stress flow around an excavation; stress flow for an isolated a) vertical stress; b) horizontal stress; c) Excavation disturbance of 2-D stress field

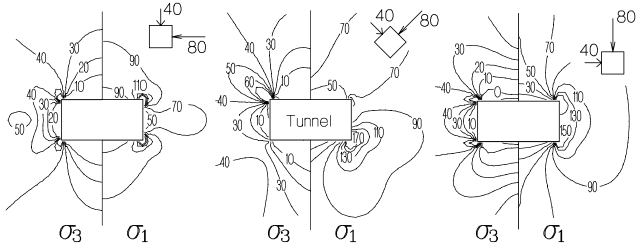


Figure 2.13.9: Influence of far-field stress orientation on induced stresses (example)

Hoek and Brown (1980) present a set of charts for stress analysis of simply shaped excavations. Analysis of induced stresses around more complex openings requires the use of numerical models. The example in Figure 2.13.9 illustrates the effect, on the induced major principal stresses, of different in situ (far-field or pre-excavation) stress orientations. Figure 2.13.10 shows both major and minor principal stresses due to a more complex stope geometry. Excavations often possess zones of *overstress* and of *relaxation*. High stress may cause rupture while low stress allows joints to dilate and blocks to unravel under gravity loading.

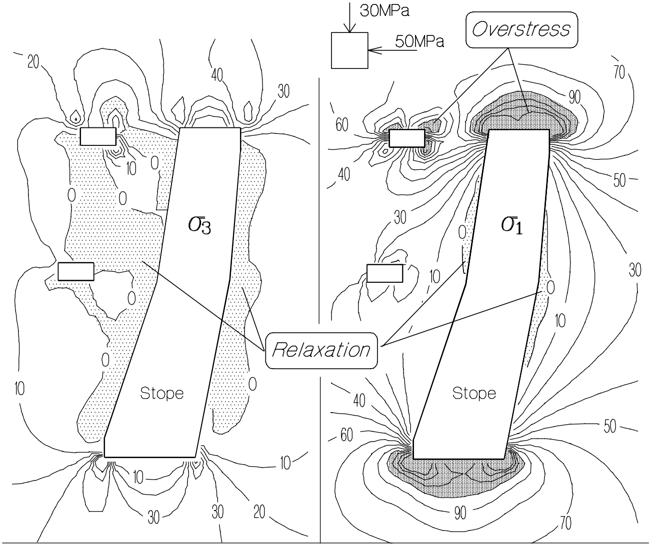


Figure 2.13.10: Zones of overstress and relaxation. Both may lead to rockmass failure.

2.13.3 Strength

Stresses tangent to and adjacent to the face of an excavation can be compared directly to the uniaxial compressive strength of the rock or rockmass (Section 2.14 2). The laboratory values, however, usually overestimate the strength in the field. Typically, field strengths of visibly competent (strong and brittle) rock adjacent to an excavation have a uniaxial or unconfined compressive strength (*UCS*) of 1/3 to 1/2 of the laboratory *UCS* (Martin et al , 1993; 1994; 1995; Brace et al., 1966; Bieniawski, 1967). When the tangential stresses at the boundary exceed this value, induced fracturing of the rock may be predicted. The strength of massive unfractured rock in tension (σ_T) is typically 5 to 10% of the *UCS*.

The compressive yield strength of the rockmass within one excavation radius of the boundary can be estimated by calculating the principal stress difference (σ_1-σ_3) and comparing this value to 1/3 to 1/2 *UCS*. Rock zones where the induced principal stress difference exceeds this value may sustain damage. The rock may only fail (fall down), however, if the confining stresses are relaxed during subsequent mining steps.

While the initial yield or damage threshold of the rock may be given by either of the above simple relationships, the ultimate rupture of this yielded rock may be more dependent on confinement (Figure 2.13.11.a). Hoek and Brown (1980; 1988) developed the Hoek-Brown criteria which relates strength in terms of maximum allowable σ_1 to confinement in terms of σ_3 through the relationship:

$$\sigma_{1S} = \sigma_3 + \sqrt{m\sigma_C\sigma_3 + s\sigma_C^2} \quad \text{where} \quad \sigma_C = UCS \text{ and } m, s \text{ are material constants.}$$

For intact rock, *s*, is given as 1.0. Note that the *UCS* shown here refers to the intact rock strength. Typical values for *UCS* and m_i of intact rock specimens are given in Table 2.14.1. For more fractured, damaged or jointed rockmasses, Hoek et al. (1995) present values of m/m_i and s for varying deviations from ideal intact conditions. These values are summarized in Table 2.15.1. The value of *s* tends to zero as the disintegration or disturbance of the rockmass becomes complete.

Elastic (non-yielding) modelling programs will typically give *factors of safety* against failure (normally the ratio of strength to stress) given the appropriate strength parameters. Areas which show low factors of safety (especially below 1.0) are areas where support may be required, both to reinforce the failing rockmass and to hold up the failed material against gravity. For modelling programs which allow yielding in the analysis (inelastic or *plastic* analysis), peak (initial failure, A) and residual (post peak strength, B) values may be specified as in Figure 2.13.11.b). The residual values determine the ultimate strength of the failed material. While ductile material (A-A') carries stress after yield, very brittle material with little strength after yield will clearly not be able to support itself. As well, if the rock is very strong but brittle and if the stresses are high, the system is capable of releasing a great deal of energy upon rupture resulting in rockburst conditions (Hoek and Brown, 1980).

Joints (continuous weakness planes) can dilate and separate under low stresses or relaxation and can also slip under gravity loading or under excavation induced shear stresses. The tensile strength of a persistent joint surface is zero. The Mohr-Coulomb criterion relates shear strength, τ_s, to a confinement (normal stress, σ_n) independent strength component or *cohesion*, c, and a confinement dependent component defined by *friction angle,* ϕ. An incremental frictional component (*dilation angle*, i) can be added to account for joint roughness (Patton, 1966):

$$\tau_S = c + \sigma_n \tan(\phi + i) \qquad \text{\{Figure 2.13.11.c\}}$$

Normally c is conservatively assumed to be zero. ϕ can vary between 15 degrees (clay gouge) to 35 degrees for coarse grained rocks. Dilation angle i can be as high as 20 degrees, for rough surfaces at low confinement, to zero at high confinement (high σ_n). Alternative strength envelopes for joints are shown in Figure 2.13.10.c). An observed non-linearity in shear strength with increasing confinement is analogous to decreasing the dilation angle ,i, and increasing cohesion, c, with increasing normal stress. This simulates the tendency for shearing through rough asperities as normal confinement increases. Barton et al. (1973; 1976; 1977; 1990) have developed a more rigorous non-linear relationship which is summarized in Stacey and Page (1986) and in Hoek et al. (1995).

The Mohr-Coulomb criterion can also be applied to the rockmass as a whole. The strength envelope defines the yield point of the critical fictitious shear plane within the rock. If half circles are plotted on the σ_n axis between σ_1 and σ_3, the same equation as above can be used to find the strength envelope which is tangent to the suite of failure circles obtained from testing data (Figure 2.13.11d). Expressed in terms of σ_1 and σ_3 the limiting Mohr-Coulomb yield envelope (Desai & Siriwardane, 1984) becomes:

$$\frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_1 + \sigma_3}{2} \sin\phi + c\cos\phi$$

where $\frac{\sigma_1 - \sigma_3}{2}$ is the *deviatoric stress* and $\frac{\sigma_1 + \sigma_3}{2}$ is the *confining pressure.*

2.13.4 Block Size and the Influence of Scale

The stability of excavations in any rockmass decreases with increasing scale (excavation span). This is due in part to the involvement of larger potential zones of rupture or of larger agglomerations of blocks. The same support density in a larger excavation may not be as effective as in a smaller excavation. In addition, larger volumes of rock have a higher probability of including discontinuities, fractures and other flaws. Laboratory samples, for example, are by necessity taken from intact pieces of drill core, avoiding cracks and flaws where possible. The strengths obtained, then, will inevitably be higher than those achieved in situ. Similarly, boreholes are smaller than caverns, and thus will have less chance of intersecting critical flaws, thereby exhibiting higher strengths (Figure 2.13.12).

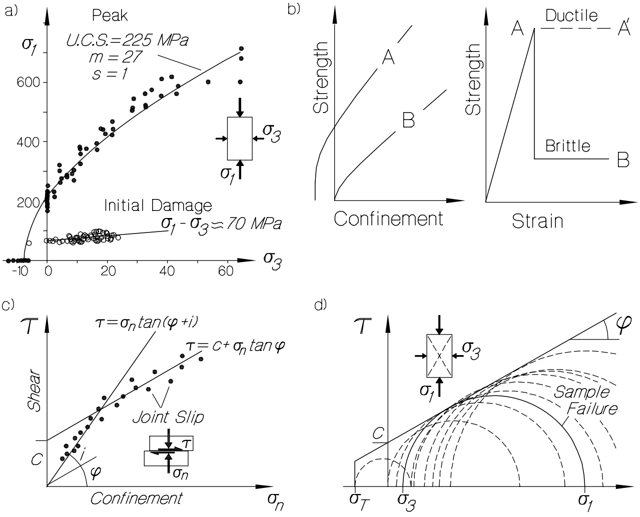


Figure 2.13.11: a) Intact granite example: Initial damage and peak Hoek-Brown strength criteria (after Martin, 1995); b) Ductile vs brittle post-peak behaviour; c) Mohr-Coulomb and Patton shear strength - joint slip; d) Mohr-Coulomb strength for rock and rockmasses

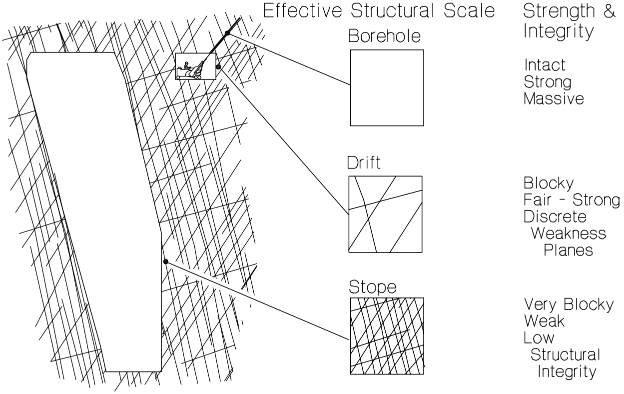


Figure 2.13.12: Scale dependent rockmass strength and structural integrity

2.14 Rockmass Classification

One of the most potentially complex tasks assigned to a rock mechanics engineer is the determination of representative mechanical properties of a rockmass. While tests have been devised to quantify strength, stiffness and other properties of laboratory rock specimens, it is a much more daunting task to evaluate the quality and expected behaviour of a rockmass in the field. Fortunately, numerous researchers have developed *empirical* methods (based on numerous case histories) to quantify the relative integrity of a rockmass and thereafter to estimate mechanical properties for excavation and support design. These methods are referred to as *rockmass classification systems*.

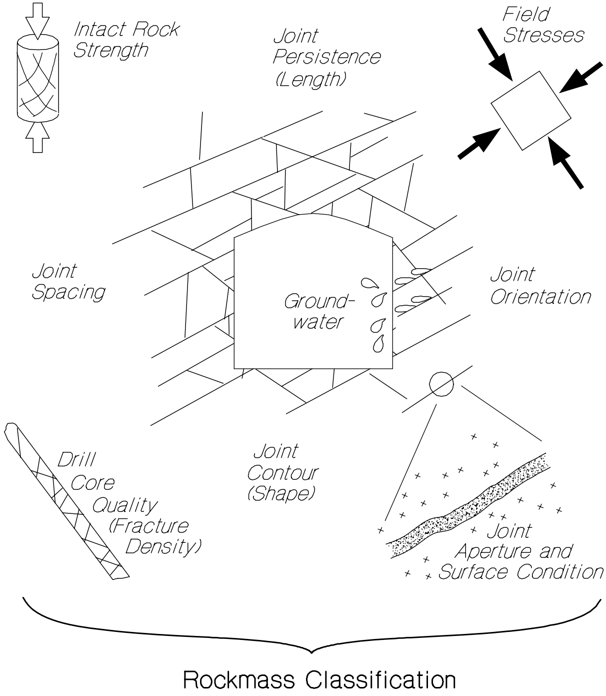


Figure 2.14.1: Basic components of a rockmass classification scheme

2.14.1 Rockmass Classification Components

Rockmass behaviour is controlled by the following components (Fig. 2.14.1):

Intact Rock Strength

Stronger rocks are more likely to be stable in general conditions than weaker rocks. Stiffness usually correlates directly with strength (Deere, 1968).

Field Stresses

At moderate depth the rockmass is likely to be confined and held together (clamped). Near surface, in late stage mining areas which have become relaxed, joints can open up, decreasing stiffness, strength and stability. At greater depth, stresses induced by the creation of the excavation may exceed the strength of the rock, resulting in induced fracturing and instability.

Fracture Density or Drill Core Quality

Diamond drill core from geotechnical or exploration drilling provides a convenient means of assessing the structural integrity of the rockmass prior to excavation. Numerous breaks in the core indicate a highly fractured or jointed rockmass which is more likely to be unstable when excavated.

Joint Persistence

Joints which are highly persistent (long) are more likely to combine with other structures to form large free blocks of rock, than are short joints. These blocks may require support to ensure stability.

Joint Spacing

Closely spaced joints result in a smaller block size, increasing the potential for internal shifting and rotation as the rockmass deforms, and reducing stability.

Joint Contour, Aperture and Surface Condition

Planar joints are able to slip more readily than wavy or undulating surfaces. Similarly, smooth or polished surfaces have lower frictional slip resistance than rough or stepped surfaces. Open joints or infilled joints are less stable than tight or healed fractures.

Groundwater

Groundwater can destabilize an excavation by eroding or weakening joint surfaces and infillings. In addition, water pressure reduces the frictional resistance to slip along fractures and further destabilizes the rockmass.

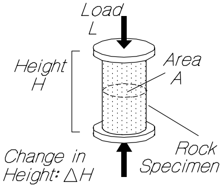
Joint Orientation

Joints can intersect an excavation at unfavourable orientations, creating the potential for slabbing, sliding blocks, stress induced slip or wall separation.

2.14.2 Data Collection

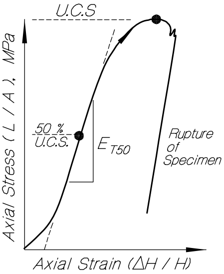
Certain basic input is required for rockmass classification and for other forms of stability assessment. Only the most common and necessary are described here. Refer to Hoek et al. (1995), Hoek and Brown (1980), Brady and Brown (1993), Bieniawski (1989) and other rock mechanics or rock engineering texts for additional investigations and analyses.

Intact Rock



Uniaxial Compressive Strength, *UCS*

UCS is defined as the maximum uniaxial (one dimensional) compressive stress sustained by a cylindrical sample, in a laboratory test (I.S.R.M , 1981), before disintegration (as shown at right). The stress is calculated as maximum load divided by the cross-sectional area of the sample.



Elastic Stiffness - Young's Modulus, *E*

Young's modulus, *E*, the modulus of elasticity defines the slope of a linear approximation of the response curve (stress vs strain) at stress levels around one-half of the uniaxial compressive strength. In this region, the sample is assumed to be elastic and the strains (deformation / length) are assumed to be fully recoverable upon unloading. Poisson's Ratio, ν, is a third important parameter for numerical analysis. In the sample test described above, ν relates the radial strain to the axial strain for a given stress increment.

Table 2.14.1: Typical *Intact* Properties (after Stacey and Page, 1986; Hoek et al., 1995)

Rock Type	UCS MPa	m_i s=1	E GPa	ν	Rock Type	UCS MPa	m_i s=1	E GPa	ν
Andesite	240	19	60	0.2	Gneiss	220	33	60	0.2
Basalt	230	17	60	0.2	Granite	220	33	60	0.2
Diabase	240	19	90	0.2	Limestone	180	8	70	0.3
Dolerite	240	19	90	0.2	Sandstone	40-80	19	20	0.2
Dolomite	100	10	70	0.2	Shale	120 *	4-9	15	0.1
Gabbro	280	27	90	0.2	Quartzite	240	24	80	0.2

** with marked anisotropy (strength varies with loading direction across laminations)*

In Situ (Far Field) Stresses

The determination of the local in situ stresses in the area of a proposed excavation is beyond the scope of this book. Consult local databases and refer to Hoek et al. (1995), Herget (1988), Brady and Brown (1993) and other texts for general stress-depth relationships. It is preferable however, to measure the local stress field using special instruments and procedures described in the above texts.

Structural Data

Again, Hoek et al. (1995) and Hoek and Brown (1980) and Hoek and Bray (1981) and others describe field mapping techniques used to evaluate the structural integrity of the rockmass. It is first necessary to identify the presence of major through-going structures such as shears, faults or major weakness zones in the vicinity of the proposed excavation. These discrete structures must be assessed separately as they will dominate local behaviour.

Next, the ubiquitous (present everywhere) structure must be assessed. Systems of extension joints and minor shear structures will have formed under historical stress fields, which were relatively consistent over a local region. As a result, there are usually several distinct groups of similarly oriented structures within a rockmass. These are called *joint sets*. Ungrouped joints are referred to as *random.*

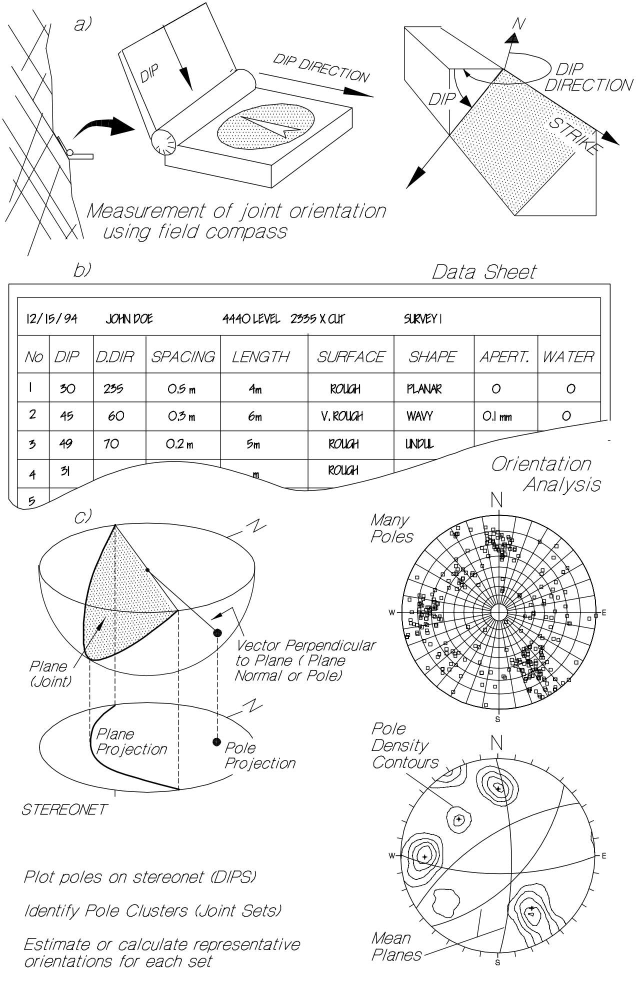
A compass (Figure 2.14.2a) is typically used in the field to record the orientation of joints in the wall, floor or back. In addition (Figure 2.14.2b) it is necessary to record qualitative information about the joint surfaces for rockmass classification and later analysis. A minimum of 100 local measurements are normally required to define the structure in a zone of rock. More measurements improve the data reduction accuracy. It is necessary, however, to restrict the data to distinct, local groups in areas of changing rockmass quality and nature.

The stereonet (Figure 2.14.2c) is used to visually and statistically resolve the data into clusters or sets. Computer software such as DIPS (Diederichs and Hoek, 1989; Hoek et al., 1995) can be used for this purpose. Representative (mean) orientations for each cluster are used in analysis.

Discontinuity Strength

Refer to Barton and Choubey (1977) or Hoek et al. (1995) for detailed joint strength determination and application. For simplified Mohr-Coulomb analysis ($c=0$, ϕ, and i in Section 2.13.3) near excavation boundaries, friction, ϕ, varies from 20 degrees for schistose joint walls, to 30 degrees for competent granular or crystalline rocks. Add 5 degrees if the joint is completely dry. The dilation angle, i, for low confinements, varies from 2 degrees for smooth joints, to 6-10 degrees for rough joints and to more than 14 degrees for very rough or stepped surfaces.

Structural Data



12/15/94 JOHN DOE 4440 LEVEL 2335 X CUT SURVEY 1

No	DIP	D.DIR	SPACING	LENGTH	SURFACE	SHAPE	APERT.	WATER
1	30	235	0.5 m	4m	ROUGH	PLANAR	0	0
2	45	60	0.3 m	6m	V. ROUGH	WAVY	0.1 mm	0
3	49	70	0.2 m	5m	ROUGH	UNDUL		
4	31			m	ROUGH			
5								

Figure 2.14.2: Joint mapping and orientation analysis (after Diederichs, 1990)

2.14.3 Rock Quality Designation, *RQD*

Deere et al (1967) developed the Rock Quality Designation, *RQD*, in response to the need for a quick and objective technique for estimating rockmass quality from diamond drill core logs during the initial exploratory phase of construction. The method is simple and efficient to implement in mining environments and can be assigned to the drillers themselves or to the geologists analyzing core for grade assessment. Information gained at this early stage of exploration is extremely valuable for the geomechanics engineer involved in the mine planning process.

RQD is calculated as the ratio of the sum of the lengths of all pieces of core greater than 10 cm to the total length of the core run. This total length of core must include all lost core sections. Breaks created by the driller during removal from the core barrel are to be ignored. Core discing due to high stress should not be considered in the calculation of *RQD* but should be noted in the drill log (give an estimate of discing frequency). Discing does not contribute to *RQD* but does indicate potential risk of brittle overstress problems during excavation.

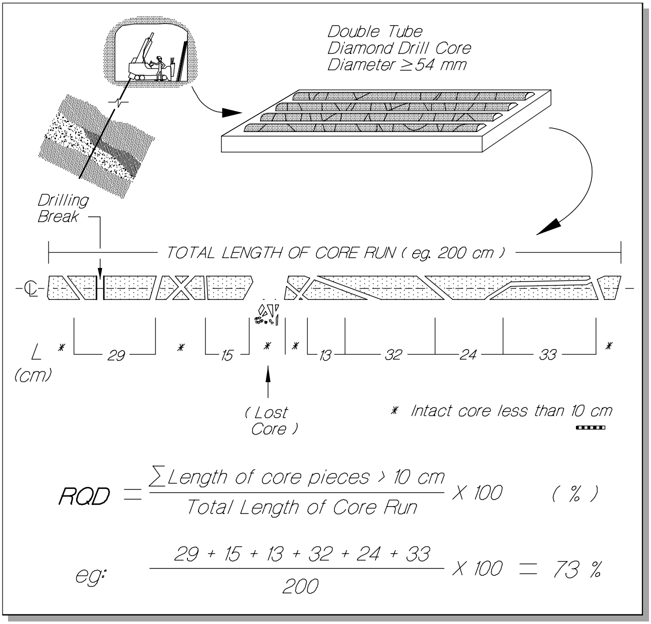


Figure 2.14.3: Conventional method for evaluating *RQD* from drill core

A great deal of work has been done to correlate *RQD* with joint frequency, rockmass stiffness, and other properties (Deere and Miller, 1966; Deere and Deere, 1988; Cording and Deere, 1972; Coon and Merritt, 1970; Bieniawski, 1979). Philosophically, *RQD* provides a crude estimate of the percentage of the rockmass which can be expected to behave in a fashion similar to a laboratory sample (typically 10 cm long). A rockmass with a low *RQD* (< 50) has few intact blocks larger than 10 cm. In such a rockmass, joints and fractures dominate the rock's response to stress and gravity. The strength and stiffness of the rock, as determined in the laboratory, has little relevance here. On the other hand, rockmasses with *RQD* > 95 % possess strength and stiffness much closer to the values obtained in the lab. Joints may still dominate behaviour in low stress environments but may have little or no influence at depth (provided joints are clean and tight). Deere proposed the following categories of rockmass quality:

Table 2.14.2:

Rock Quality Designation (Description)	*RQD Value*
Very Poor	**0 - 25**
Poor	**25 - 50**
Fair	**50 - 75**
Good	**75 - 90**
Excellent	**90 - 100**

RQD does not accurately reflect conditions in rockmasses with joint spacings greater than 0.3m. Rockmasses with block sizes in this range can be problematic and therefore require additional parameters for adequate classification. While *RQD* forms the starting point for most assessment procedures, more comprehensive classification schemes are discussed in the following sections.

Also consider the directional nature of *RQD*. For example, a drill hole parallel to a set of major laminations in highly anisotropic rock will yield a relatively high *RQD*. Core taken from a hole perpendicular to the lamination set will give a much lower value. When practical, drill core from two or more boreholes at different angles should be considered for complete assessment of *RQD*.

RQD is intended to give a measure of in situ and undisturbed rockmass conditions. Therefore all core breaks due to drilling, handling and discing must be ignored in the calculation of *RQD*. In addition, minor cracks in the core which are not related to established jointing should also be ignored. Failing to do so may result in an overconservative or unrepresentatively low measure of *RQD*. In hard rock mining applications, RQD typically measures between 50 and 100. Values lower than this represent special conditions or an unusually poor rockmass. Exceptions include *RQD* measured perpendicular to schistosity or foliation. Such a measurement may be much lower than the *RQD* of the surrounding rock. Blast damage to a rockmass can also be reflected in a reduced RQD (Løset, 1992).

Ideally the *RQD* should be measured as soon as possible after the core has been removed from the core barrel. *RQD* should be a part of the preliminary logging procedure. If a clearly marked rule is laid out along side of the core box, a geologist or technician can become very efficient at estimating *RQD* with a minimum of additional time expenditure. The window size for *RQD* calculation and recording can vary between 2 m and 10 m (e g. record a separate measure for each successive 2 m of core recovered) depending on the resolution required for the project. The window should be reduced in zones of geological transition or where the measured *RQD* is observed to change significantly over short distances. In addition, *RQD* can be remeasured some time after recovery to determine if the rock is susceptible to rapid disintegration upon exposure.

Alternate methods of estimating RQD

Unfortunately, the engineer is often required to estimate *RQD* without timely access to drill core or without historical logs of *RQD*. Palmström (1982) proposed a fallback method of estimating *RQD* from exposed joint traces on excavation walls or outcrops. The Volumetric Joint Count, J_V, is the sum of the number of joints per metre for each of the major joint sets present. Alternatively, the inverse of the representative true spacings for each set can be used, as shown in Figure 2.14.4. Note that true spacings must be used and not the apparent spacings produced by oblique intersection with the rock wall. This measure is valid for rockmasses with 3 or more well developed joint sets. This estimated *RQD* will represent a maximum value. That is, no random joints or fractures are considered and damage due to blasting and stress are also ignored. Palmström (1995) gives alternate relationships for one and two dominant joint sets, while Priest (1993) and Priest and Hudson (1976, 1981) present RQD relationships using scanlines.

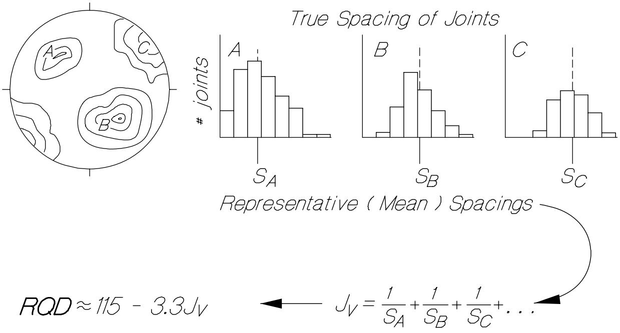


Figure 2.14.4: Obtaining RQD from volumetric joint count, *Jv*

Another simple method for estimating *RQD* is illustrated in Figure 2.14.5. A two metre graded rule can be placed on an exposed rock face. Calculate *RQD* as described for the drill core, considering any well developed joint which intersects the ruler as a core break. When estimating *RQD* for an undisturbed rockmass, care must be taken to consider only in situ joints and not induced tensile cracks and blast related fractures. Disregard any fractures which are less than 30 cm in length and consider disregarding larger fractures which are clearly induced (have a "sugary" surface). This will give a "best case" or upper-bound value of *RQD*.

This technique can also be used to determine the degree of degradation due to blasting and excavation. By considering all joints and fractures (induced) in the measurement of RQD_W (*wall*), an estimate of post-excavation rockmass quality can be obtained (ignore fractures less than 30 cm in length). This may be a more relevant value for local support design. Note the additional subscript *W* attached to this measurement. Maintain this notation to differentiate the value from true *RQD* (joints only). Palmström's *RQD* (1982) and this crude RQD_W measurement serve to provide an upper and lower bound respectively for local rock quality.

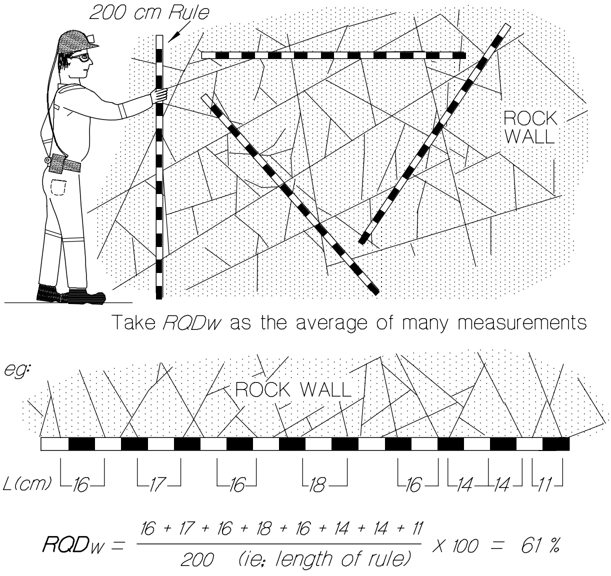


Figure 2.14.5: Estimating equivalent *RQD* (RQD_W) from exposed wall jointing

2.14.4 Rock Mass Rating, *RMR*

This rating system is also known as the Geomechanics Classification and was developed by Bieniawski (1976) for use in design of tunnels in hard and soft rock. As more data was collected, Bieniawski continued to refine his classification scheme making changes and adjustments as necessary. This discussion is based on the latest revision detailed by Bieniawski (1989; 1993).

The scheme considers six factors:	Factor	Range
• Uniaxial strength of the intact rock material:	***A1***	0 - 15
• Rock Quality Designation, *RQD*:	***A2***	3 - 20
• Spacing of discontinuities:	***A3***	5 - 20
• Condition of discontinuities:	***A4***	0 - 30
• Groundwater conditions:	***A5***	0 - 15
• Orientation of discontinuities (adjustment for tunnels & mines):	***B***	(-12) - 0

A numerical factor is assigned to each category above and the sum of these factors yields the Rock Mass Rating, *RMR:*

$$RMR = A1 + A2 + A3 + A4 + A5 + B$$

Based on this relationship and the parameters which are described in detail in the following pages, Bieniawski proposed the following rockmass classifications:

Table 2.14.3:

Rock Mass Class	*Description*	*RMR*
I	Very Good Rock	**81 - 100**
II	Good Rock	**61 - 80**
III	Fair Rock	**41 - 60**
IV	Poor Rock	**21 - 40**
V	Very Poor Rock	**0 - 21**

NOTE: *Bieniawski(1989) suggests that poor blasting can reduce RMR by up to* 20%

The rockmass to be considered should initially be divided into geologically or geomechanically distinct zones (e.g.; hangingwall granite, hangingwall schist, ore zone, footwall gabbro, main fault zone, etc.). Each zone should be classified separately. If during the classification process, significant changes in structural character or in proposed excavation profile are noted, then additional subdivisions and classifications should be made until all unique geomechanical zones are identified and assigned a rating. These zones permit adaptation of design to local conditions and provide an immediate reference for future planning.

Factor A1 - Strength of Intact Rock Material

Strength of the intact rock can be obtained from uniaxial compressive strength tests (I.S.R.M., 1981) or from Point Load Index tests (Hoek and Brown, 1980):

Table 2.14.4:

Uniaxial Compressive Strength (MPa)	*Point Load Strength Index (MPa)*	*Factor A1*
> 250	>10	**15**
100 - 250	4 - 10	**12**
50 -100	2 - 4	**7**
25 - 50	1 - 2	**4**
5 - 25	n/a	**2**
1 - 5	n/a	**1**

Factor A2 - Rock Quality Designation, RQD

RQD is used in the RMR classification as a measure of structural integrity:

Table 2.14.5:

Rock Quality Description	*RQD %*	*Factor A2*
Very Good Rock	90 - 100	**20**
Good Rock	75 - 90	**17**
Fair Rock	50 - 75	**13**
Poor Rock	25 - 50	**8**
Very Poor Rock	0 - 25	**3**

Factor A3 - Spacing of Discontinuities

Calculate the average true spacing for each joint set. Use the smallest of these average values to determine Factor *A3:*

Table 2.14.6:

Minimum Average Discontinuity Spacing (cm)	*Factor A3*
> 200	**20**
60 - 200	**15**
20 - 60	**10**
6 - 20	**8**
< 6	**5**

Factor A4 - Condition of Discontinuities

Based on limited information about the character of the discontinuity surfaces and using the broad categories listed below, Factor *A4* can be estimated:

Table 2.14.7:

Description of Discontinuity Surfaces (Roughness, Persistence, Separation, Wall Condition)	*Factor A4*
Very Rough Surfaces, Not Continuous (Non-persistent), No Separation (Full Wall Contact), Unweathered Joint Walls	**30**
Slightly Rough Surfaces, Moderately Persistent, Separation < 1mm, Slightly Weathered Joint Wall Rock	**25**
Slightly Rough Surfaces, Moderately Persistent, Separation < 1mm, Highly Weathered Joint Walls	**20**
Slickensided OR Gouge < 5mm OR Separation 1 - 5 mm, Continuous (Highly Persistent)	**10**
Soft Gouge (or Clay) > 5mm Thick OR Separation > 5mm Continuous (Highly Persistent)	**0**

When more information is available and a higher degree of accuracy is warranted (due in part by the dominant nature of this parameter with respect to *RMR*), use the chart below to calculate Factor *A4*. Consider each subfactor separately and then add up the results to obtain *A4*:

$$A4 = E1 + E2 + E3 + E4 + E5$$

Table 2.14.8:

Persistence or Length (E1)	*Separation or Aperture (E2)*	*Surface Roughness (E 3)*	*Infilling or Gouge (E4)*	*Weathering of Joint Wall (E5)*
< 1 m (**6**)	None (**6**)	Very Rough (**6**)	None (**6**)	Unweathered (**6**)
1 - 3 m (**4**)	< 0.1 mm (**5**)	Rough (**5**)	Hard Infilling < 5 mm (**4**)	Slightly Weathered (**5**)
3 - 10 m (**2**)	0.1 - 1.0 mm (**4**)	Slightly Rough (**3**)	Hard Infilling > 5 mm (**2**)	Moderately Weathered (**3**)
10 - 20 m (**1**)	1 - 5 mm (**1**)	Smooth (**1**)	Soft Infilling < 5 mm (**2**)	Highly Weathered (**1**)
> 20 m (**0**)	> 5 mm (**0**)	Slickensided (**0**)	Soft Infilling > 5 mm (**0**)	Decomposed (**0**)

Factor A5 - Ground Water

Groundwater (or persistent mine water at depth) can play a significant role in rockmass behaviour by altering joint surface conditions with time and by creating an "effective stress" condition in which normal rock pressure is relieved across joint surfaces, thereby reducing the frictional shear strength. *RMR* accounts for this effect through the inclusion of Factor *A5*:

Table 2.14.9:

Inflow in litres per 10m tunnel length	$\frac{\text{Jnt. Water Pressure}}{\text{Major Princ. Stress}}$	*General Conditions*	*Factor A5*
None	0	Dry	**15**
< 10	< 0.1	Damp	**10**
10 - 25	0.1 - 0.2	Wet	**7**
25 - 125	0.2 - 0.5	Dripping	**4**
> 125	> 0.5	Flowing	**0**

Factor B - Joint Orientation Adjustment

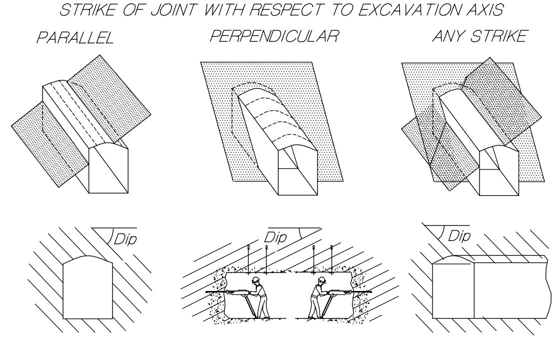
The basic value of *RMR*, which is used for estimation of stiffness and strength properties, and for classification of the rockmass independent of the proposed excavation, includes the summation of the five *A* factors only. When *RMR* is to be used for determination of support requirements and general stability assessment, the relative orientation of dominant discontinuities with respect to the excavation must be taken into account. In addition to adjustments for foundations and slopes, Bieniawski (1989) provided the following classes for Factor *B* for tunnelling and mining (note the negative adjustments to *RMR*):

Table 2.14.10:

Orientation of critical (most detrimental) joint set with respect to tunnel or mine excavation	*Factor B*
Very Favourable	**0**
Favourable	**-2**
Fair	**-5**
Unfavourable	**-10**
Very Unfavourable	**-12**

These descriptions (i.e. favourable, unfavourable, etc.) are based on the consideration of relative strike and dip of the joint and of the excavation and on the relative direction of tunnelling (development) with respect to joint dip as summarized in Figure 2.14.6.

Factor B - Joint Orientation Adjustment (continued)



DIP (degrees)		DRIVE WITH DIP	DRIVE AGAINST DIP	
45 - 90	Very Favourable	Very Favourable	Fair	n/a
20 - 45	Fair	Favourable	Unfavourable	n/a
0 - 20	Fair	Fair	Fair	Fair

Figure 2.14.6: Effect of discontinuity orientation on stability - use with factor *B*

The orientation adjustments for joints striking perpendicular to the tunnel are based on the assumption that patterned bolting is being installed with each round. The difference between driving with dip and driving against dip arises from the ability to safely bolt potential blocks, before they become liberated, when driving with dip. When the discontinuity strike is neither parallel nor perpendicular to the tunnel, use an adjustment based on dip which lies between these two extremes.

Additional Notes

Bieniawski (1989) summarizes a number of modifications which could lead to improved applicability of the Geomechanics Classification, *RMR*, to mining. In addition, Laubscher (1977, 1984), Laubscher and Taylor (1976) and Page and Laubscher (1990) and Stacey and Page (1986) describe a classification system based on RMR called the Modified Rock Mass Rating which accounts for blasting, stress change, mining influences and reduced design stand-up times encountered in mining.

2.14.5 Rock Tunnelling Quality Index, *Q*

Barton et al. (1974) studied a large number of underground excavation case histories and developed the Tunnelling Quality Index, *Q*. It is a means of classifying rockmasses with respect to in situ parameters including rock quality, joint condition and stress state. *Q* is defined by:

$$Q = \frac{RQD}{Jn} x \frac{Jr}{Ja} x \frac{Jw}{SRF}$$

where:

RQD is the *Rock Quality Designation* (Section 2.14 3).
This parameter indicates the percentage of rock which can be expected to possess strength and stiffness properties comparable to a 10cm laboratory sample of intact rock. *RQD* ranges from 10 to 100 when being used in the calculation of *Q*.

Jn is the *Joint Set Number*
This factor accounts for the number of repetitive joint sets and the relative dominance of random fracturing and jointing. *Jn* ranges from a value of 0.5 (no joints) to 20 (completely crushed rock).

RQD/Jn is a crude representation of the *average block size.*
The extreme values of *RQD/Jn* thus calculated are 0.5 to 200. Clearly this is an extremely crude index of block size. It does, however, provide a means of comparison and can be used to empirically estimate support spacing and surface retention requirements.

Jr is the *Joint Roughness Number*
Jr describes the large and small scale surface texture of the critical joint set. *Jr* ranges from 0.5 (unfavourable) to 4.0 (favourable).

Ja is the *Joint Alteration Number*
Ja describes the surface alteration and frictional resistance of the *critical* joint set and ranges from 0.75 (favourable) to 20 (unfavourable).

Jr/Ja represents *joint surface integrity and strength*. It favours rough, unaltered, discontinuous joints. When rocks are smooth or slickensided (polished by shear) and/or if they contain low friction coatings or filling, they are considered detrimental to stability. *Jr/Ja for the critical joint set should be used in the calculation of Q*. This is the joint set most likely to cause problems based on the values of Jr and Ja, and also based on the geometry of the joint. Joints which make a shallow angle (<35°) with respect to a surface are the most critical, followed by joints parallel to a surface. Joints which are perpendicular to an excavation surface are usually the least critical. In the case where gravity sliding is the dominant failure mode, inclined joints (> 35° with respect to horizontal) are likely to be critical. Clearly, some subjective judgement is required here in order to determine the critical set.

Jw	is the *Joint Water Reduction Number* *Jw* accounts for the destabilizing effect of high water pressures and of joint washout by water influx. *Jw* ranges in value from 1.0 for dry excavations to 0.05 for excavations with excessive inflow and pressure.
SRF	is the *Stress Reduction Factor* *SRF* modifies *Q* to account for high in situ stresses which may cause compressive failure of the rock. The destabilizing effect of very low confining stresses in structured ground is also considered. Either condition results in a higher value of *SRF* and therefore in a lower value for *Q*. Optimum *SRF* (0.5) is achieved under moderate confining stress which locks up the joint structure while posing no danger of overstress of intact rock. *SRF* is also affected by large scale weakness zones which may cause unfavourable conditions for excavation
Jw/SRF	is a complex factor representing the *active stress (and strength) state* in a rockmass as it occurs in situ and as it may be altered by the presence of water and cross-cutting structural weaknesses.

Barton et al. (1974) proposed the following classifications of rockmass quality based on the evaluation of *Q*:

Table 2.14.11:

Tunnelling Quality Index $Q = \frac{RQD}{Jn} x \frac{Jr}{Ja} x \frac{Jw}{SRF}$	*Rock Mass Description*
0.001 - 0.01	Exceptionally Poor
0.01 - 0.1	Extremely Poor
0.1 - 1	Very Poor
1 - 4	Poor
4 - 10	Fair
10 - 40	Good
40 - 100	Very Good
100 - 400	Extremely Good
400 - 1000	Exceptionally Good

These classifications can be used to make relative comparisons between different rockmasses. *Q* is used later in this chapter to determine stability of excavations, to estimate strength and stiffness parameters and to make crude support recommendations. The factors which make up *Q* are determined as shown in the following tables. In hard rock mines, Q typically ranges from 0.1 to 100.

RQD *Rock Quality Designation*

RQD is calculated using drill core from the area of interest. Other methods of calculating *RQD* in the absence of core samples are given in Section 2.14.3. The fundamental interpretation of *RQD* is the same regardless of the method:

$$RQD = \frac{\text{Sum of lengths of core sticks greater than 10cm long}}{\text{Total length of core run}} \times 100$$

Table 2.14.12:

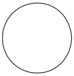
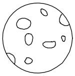
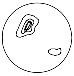
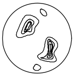
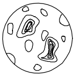
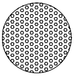
Rock Quality Designation (Description)	*RQD Value*
Very Poor	**0 - 25**
Poor	**25 - 50**
Fair	**50 - 75**
Good	**75 - 90**
Excellent	**90 - 100**

NOTE: *When RQD is less than 10, use 10 for the purposes of calculating Q and Q'.*

Jn *Joint Set Number*

Jn is determined from the results of joint mapping, stereonet plotting and cluster contouring as shown below. Note that a joint set must be relatively well developed as a cluster. Otherwise it should be considered as a random joint.

Table 2.14.13:

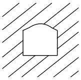
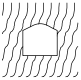
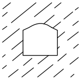
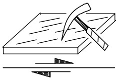
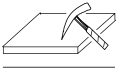
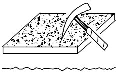
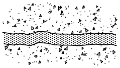
# of Joint Sets		*Jn*			*# of Joint Sets*
Intact Rock No Joints		**0.5**	**1**		Few Random Joints Only
1 Set		**2**	**3**		1 Set + Random
2 Sets		**4**	**6**		2 Sets + Random
3 Sets		**9**	**12**		3 Sets + Random
> 4 Sets Heavily Jointed		**15**	**20**		Earthlike, Crushed Rock

NOTE: *Stereonets should show local joints only (from current design zone) for Jn*

Jr *Joint Roughness Number*

Jr relates both large and small scale surface texture for discontinuities:

Table 2.14.14:

	Large Scale:	Planar	Undulating	Discontinuous
Small Scale:	*Jr (Critical Set)*			
Slickensided		**0.5**	**1.5**	**2.0**
Smooth		**1.0**	**2.0**	**3.0**
Rough		**1.5**	**3.0**	**4.0**
Gouge-Filled No Wall Contact		**1.0**	**1.0**	**1.5**

NOTE: *Add 1.0 to Jr if mean spacing of critical joint set exceeds 3m*

Ja *Joint Alteration Number*

Barton et al. (1974) offer a comprehensive listing of alteration classifications and *Ja* factors. The following chart is abbreviated for hard rock mining:

Table 2.14.15:

Typical Description (Critical Joint Set)	*Ja*
Tightly Healed	**0.75**
Surface Staining Only	**1.0**
Slightly Altered Joint Walls. Sparse Mineral Coating.	**2.0-3.0**
Low Friction Coating (Chlorite,Mica,Talc,Clay) < 1 mm thick	**3.0-6.0**
Thin Gouge, Low Friction or Swelling Clay 1 - 5 mm thick	**6.0-10.0**
Thick Gouge, Low Friction or Swelling Clay > 5 mm thick	**10.0-20.0**

Jw *Joint Water Reduction*

Jw accounts for the weakening effect of groundwater and for the effective normal stress reduction due to water pressure. Consider mine water only if it is persistent. Do not consider water inflow from temporary drilling, for example.

Table 2.14.16:

Joint Water Description	*Pressure (kPa)*	*Jw*
Dry Excavation (Less than 5 litres/min locally)	< 100	**1.0**
Medium Inflow or Pressure	100-250	**0.66**
Large Inflow or High Pressure No Joint Filling	250-1000	**0.5**
Large Inflow or High Pressure Outwash of Joint Filling	250-1000	**0.33**
Exceptionally Large Inflow or Pressure Decaying After Excavation	> 1000	**0.2-0.1**
Exceptionally Large Inflow or Pressure No Reduction After Excavation	> 1000	**0.1-0.05**

SRF *Stress Reduction Factor (a; rock stress)*

SRF is used to account for fracturing of the rock due to overstressing during excavation and to account for reduced confinement of structurally dominant rockmasses near surface (or in late-stage, destressed mining environments).

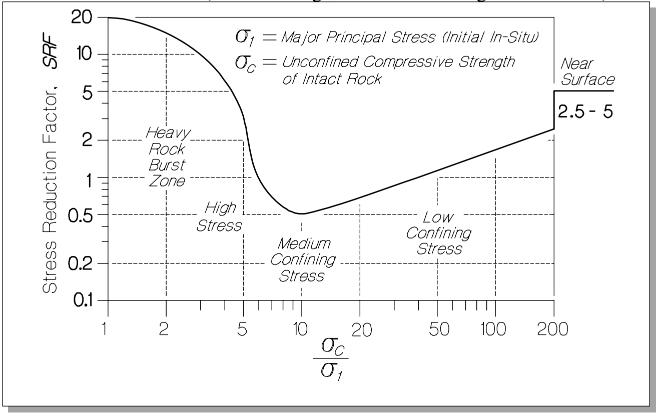


Figure 2.14.7: SRF with respect to in situ stress. Note: for highly anisotropic stress: when $5<\sigma_1/\sigma_3<10$, use $\sigma_c=0.8\sigma_c$; when $\sigma_1/\sigma_3>10$, $\sigma_c=0.6\sigma_c$

SRF *Stress Reduction Factor (b; weakness zones)*

SRF can also be used to account for major weakness zones in areas where their presence dominates the behaviour of the rockmass and causes loosening when excavated. Use these factors (*SRF;b*) instead of those on the previous page (*SRF;a*) when these weakness zones influence or intersect the excavation:

Table 2.14.17:

Weakness Zone (If Present)		*SRF*
Single shear zone in competent rock *or*	Excavation Depth	
Single weakness zone containing clay,	> 50 m	**2.5**
and/or chemically disintegrated rock	< 50 m	**5.0**
Loose open joints, heavily jointed or 'sugar cube' (any depth)		**5.0**
Multiple shear zones in competent rock *or* Loose surrounding rock (any depth)		**7.5**
Multiple occurrences of weakness zones containing clay, and/or chemically disintegrated rock *or* Very loose surrounding rock (any depth)		**10.0**
Swelling rock - chemical swelling due to presence of water		**5-15**
Squeezing rock - plastic flow of weak rock under stress		**5-20**

NOTE: *Reduce these values by 25-50% when weakness zones influence but do not intersect the excavation.*

Additional Notes

- For critical intersections and access portals use 2.0 to 3.0 x *Jn* to evaluate *Q*.
- Where drill core is not available, see Section 2.14.3 to evaluate *RQD*.
- Only well developed joint orientation clusters should be considered as sets. Foliation sets should be considered only if the potential for significant parting exists. Otherwise, in either case, consider the set as random.
- When determining *Jn*, filter joint data set to include only those joints within a reasonable spatial distance from the proposed excavation segment. All joint sets may not be present in every location. Delineate design zones based on convenient excavation steps (e.g. for each stope) or based on structural change and obtain local values of *Q*.
- The joint set with minimum *Jr/Ja* should be used to evaluate *Q* unless this joint is favourably oriented for stability (non-sliding or perpendicular to clamping). In this case, define a different critical joint. *Jr/Ja* relates to surfaces most likely to initiate failure.
- Use *SRF;a* if intact rock dominates stress response. Use *SRF;b* if the rock mass contains clay or if large scale weakness zones are present. In this case, the intact rock will play little role in stress response.

2.14.6 Modified Rock Quality Index Q'

The Rock Tunnelling Quality Index, *Q* has been used with a great deal of success (Barton et al., 1992) in the design of tunnels in rock. It contains six parameters which were deemed to influence the inherent stability of the rockmass and which therefore dictate the degree of support required for tunnelling.

The parameter *SRF*, however, becomes redundant when the classification system is used for the estimation of rockmass properties for the purpose of analytical or numerical modelling for design. The influence of stress is taken into account within the model.

The Modified Rock Quality Index *Q′* is given as;

$$Q' = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times Jw$$

SRF is set to 1.0 which is equivalent to a moderately clamped but not overstressed rockmass. In addition, in most underground hard rock mining environments, the excavations are relatively dry (not considering transient mine water flow from drilling or backfilling). *Jw*, therefore can also be set to 1.0 in this case. In environments with high water pressures, stress-based analyses should include the effects of water pressures and flow if *Jw* is to be dropped from *Q′*. When specifying the Modified Rock Quality Index, always use the apostrophe (') to distinguish it from Barton's *Q*.

This parameter should better reflect the inherent character of the rock mass, independent of the excavation size and shape which are considered separately in subsequent analyses. Use *Q′* therefore to estimate rockmass modulus and strength (Section 2.15 and Hoek et al., 1995) while using the original *Q* directly when applying Barton's stability and support recommendations (Section 2.16).

Factor *Q′* with *Jw* also set to 1.0;

$$Q' = \frac{RQD}{Jn} \times \frac{Jr}{Ja}$$

is used along with several other factors (accounting for jointing, stope geometry and overstress) to determine the Modified Stability Number, *N′*, which is used in the Modified Stability Graph method (Mathews et al., 1981, Potvin, 1988; Bawden, 1993 and Hoek et al., 1995) for dimensioning of open stopes in mining and for the design of cablebolt support in these environments (Section 2.17).

2.14.7 Comparison of Rockmass Classifications

The previous discussions of RQD, RMR, Q and Q' illustrate only a portion of the development history of classification schemes. Classification methods for tunnelling and rock excavation begin with an early reference by Terzaghi (1946), followed by work by Lauffer (1958), Wickham et al. (1972), Laubscher (1977, 1984, 1993), Palmström (1995) and many others. In addition, other researchers have modified RMR for application to mining problems (Laubscher and Taylor, 1976; Page and Laubscher, 1990; Cummings et al., 1982; Kendorski et al., 1983). The treatment given in this book is merely intended as an initial and practical introduction to the most popular methods. Interested readers are directed to the preceding references. Comprehensive treatment of classification techniques can be found with bibliographies in Hoek and Brown (1980), Hoek et al. (1995), Franklin (1993) and Bieniawski (1989).

It is interesting to note the differences between the four systems presented here. Figure 2.14.8 illustrates the relative emphasis in each system of specific characteristics of the rockmass and of the environment. It is important to keep these differences in mind when comparing and utilizing classification results.

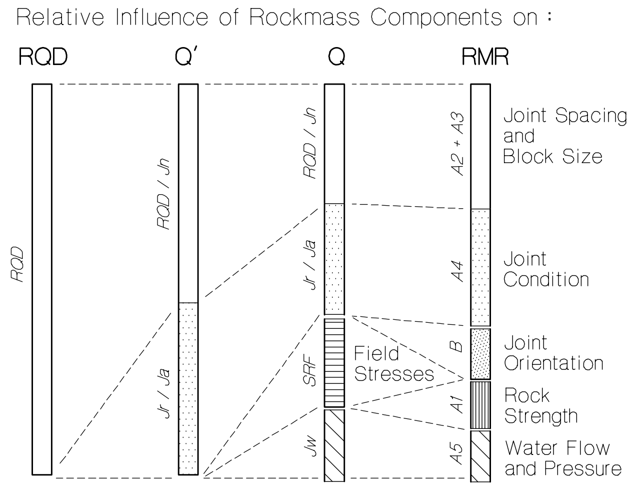


Figure 2.14.8: Rockmass component contributions to RQD, Q, Q', and RMR

In the initial stages of a project, it is always advisable to employ at least two classification schemes and compare results and recommendations before continuing (Kaiser et al., 1986). With increased experience at a particular site, it may be adequate in time to maintain the use of only the most appropriate method (i.e. that which seems to best predict behaviour as monitored).

Many relationships have been proposed which attempt to relate the outputs of different rockmass classification schemes. In particular, the relationship (Bieniawski 1979, 1993):

$$RMR = 9\ ln_e Q + 44 \quad or \quad RMR = 21\ log_{10} Q + 44 \quad or \quad Q = 10^{(RMR - 44)/21}$$

has been used throughout the literature. While this relationship can be useful for comparison purposes or where correlations (e.g. rockmass modulus) are only available for one classification system, it should not be used to apply the results from one classification method to obtain recommendations based on another. This caution is due to the differences summarized in Figure 2.14.8. Figure 2.14.9 below illustrates the scatter in the relationship and the hazards inherent in its use.

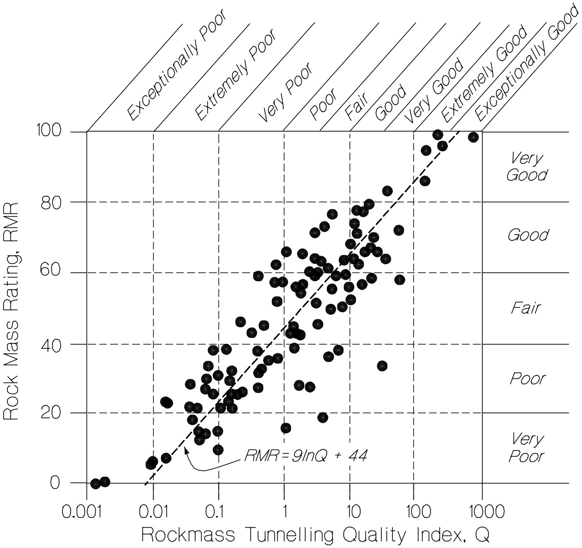


Figure 2.14.9: Comparison between RMR and Q results (after Bieniawski, 1993)

2.15 Rockmass Properties from Classification Systems

It is relatively simple to measure intact rock properties such as uniaxial compressive strength and rock modulus (I.S R M., 1981) from laboratory specimens. In situ fracturing (at all levels of scale), jointing, relaxation and weathering all serve to degrade the properties of the rockmass. Measurement of the real values of rockmass strength and stiffness is difficult and is beyond the practical and economic scope of most mining operations.

One alternative is to use rockmass classification to adjust for in situ effects and to estimate these parameters.

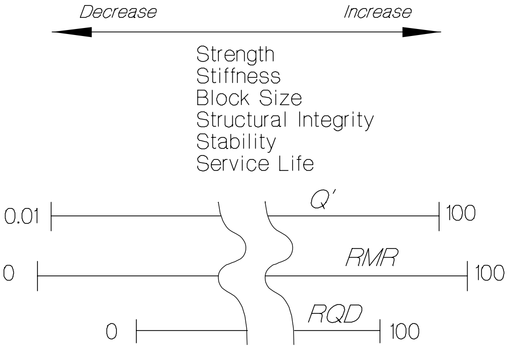


Figure 2.15.1: Influence of rockmass quality (as estimated by rockmass classification) on in situ rockmass properties

2.15.1 Rockmass Strength

A great deal of research (Deere, 1968; Hoek and Brown, 1980; Hoek, 1983; Hoek and Brown, 1988; Hoek et al., 1995; and others) has focussed on the estimation of triaxial strength characteristics of rockmasses. A detailed treatment of these developments is beyond the scope of this chapter. In short, increased fracture density has a dominant influence on the cohesive (confinement independent) shear strength of rockmasses. Frictional (confinement dependent) shear strength reduces as joint interlock is reduced, and internal (block) mobility is increased. As a conservative estimate, the tensile strength of a heavily jointed rockmass reduces to zero.

Unconfined compressive strength, UCS_R, of a non-intact rockmass can be estimated from the laboratory strength UCS_L and using either RMR (Section 2.14.4) or Q' (Section 2.14.6) using the following relationships:

$$UCS = \sigma_{cR} = \sqrt{s}\left(UCS_L\right) \quad \text{where } s = e^{\left(\frac{GSI-100}{9}\right)} \quad \{\text{but for } GSI < 25; \quad s = 0\}$$

$$GSI = \text{Geological Strength Index (Hoek et al., 1995)}$$

$$GSI = RMR - 5 \quad \{\text{for } RMR > 23\} \qquad \text{or} \qquad GSI = 9\ln_e Q' + 44$$

Note, however, that even in ideal apparently intact rock, the uniaxial yield strength (onset of damage) rarely exceeds 1/2 of the laboratory test value (Bieniawski, 1976; Brace et al., 1966; Martin et al., 1993; 1994; 1995).

Hoek et al. (1995) present the summary in Table 2.15.1 for triaxial or confinement dependent strength based on rockmass condition. Refer to this reference for a more detailed discussion of rockmass strength.

Table 2.15.1: In situ rockmass strength and stiffness (after Hoek et al., 1995)

HOEK-BROWN CRITERION $\sigma_1^F = \sigma_3^F + \sqrt{m_b \sigma_C \sigma_3^F + s\sigma_C^2}$ σ_1^F = major principal stress at failure σ_3^F = minor principal stress at failure σ_C = unconfined compressive strength of *intact* rock m_b, s = rockmass strength parameters m_i = strength parameter for intact rock ROCKMASS STRUCTURE	SURFACE CONDITION	VERY GOOD (Jr/Ja = 3 to 5) Very rough, unweathered surfaces	GOOD (Jr/Ja = 1 to 3) Rough, slightly weathered, iron stained	FAIR (Jr/Ja = 0.3 to 1) Smooth, moderately weathered or altered	POOR (Jr/Ja = 0.05 to 0 3) Slickensided, highly weathered with compact coatings/ fillings containing angular fragments	VERY POOR (Jr/Ja < 0.05) Slickensided highly weathered with soft clay coatings or fillings
BLOCKY (*RQD/Jn* > 7.5) Very well interlocked, undisturbed rockmass consisting of cubical blocks: 3 orthogonal joint sets	m_b/m_i	0.60	0.40	0 26	0.16	0.08
	s	0.19	0.62	0.015	0.003	0.0004
	E(GPa)	75	40	20	9	3
	v	0.20	0.2	0 25	0.25	0.25
	GSI	85	75	62	48	34
VERY BLOCKY (*RQD/Jn* = 0.25 - 7.5) Interlocked, partially disturbed with polyhedral angular blocks formed from 4 or more joint sets	m_b/m_i	0.40	0.29	0.16	0.11	0.07
	s	0.62	0.021	0.003	0.001	0
	E(GPa)	40	24	9	5	2.50
	v	0.2	0.25	0 25	0.25	0.3
	GSI	75	65	48	38	25
EXT. BLOCKY/SEAMY (RQD/Jn < 0 25) Folded and faulted with many intersecting discontinuities forming angular blocks	m_b/m_i	0.24	0.17	0.12	0.08	0.06
	s	0.012	0.004	0.001	0	0
	E(GPa)	18	10	6	3	2
	v	0.25	0.25	0 25	0.3	0.3
	GSI	60	50	40	30	20

2.15.2 Stiffness: Rockmass Modulus

The stiffness of a rockmass controls the response to loading and unloading at levels of stress below the strength of the material. Elastic strains and therefore displacements can be directly related to stress changes using the rockmass stiffness or modulus. Rockmass modulus is required as input into most numerical and analytical models of rockmass behaviour. Unfortunately, it is very difficult to accurately estimate and, in mining, impractical to measure with any confidence. The rockmass modulus, in simple terms, directly relates a uniaxial stress change with an induced strain or a displacement increment in a unit length of rock. It is a combination of the intact rock response and the combined responses of all of the cross-cutting joints, as shown below.

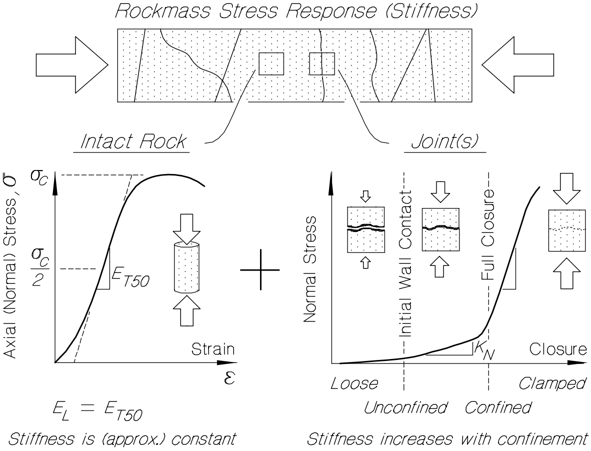
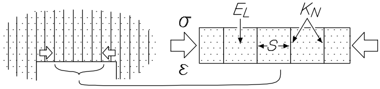


Figure 2.15.2: Influence of joint closure on rockmass modulus

As illustrated above, rockmass stiffness or modulus depends on the modulus of the intact rock, on the joint density and on the joint surface character, wall stiffness and infilling. The modulus of a fractured or jointed rockmass, however, depends primarily on the level of confinement. Due to the lower unconfined modulus, a 1 MPa stress increase (or decrease) at low confinement (e.g. near surface) will result in much larger strains than will a similar stress increment at higher confinements and depths. At lower confinements, higher strains result from the requirements for initial closure to compress the softer joint surfaces or infilling. The modulus significantly increases once full closure is achieved.

If K_N represents the closure stiffness of a joint (GPa/m) at a given range of confinement, if E_L is the lab rock modulus (E_{T50} in Figure 2.15.2) and if S represents the spacing of a single joint set perpendicular to the applied stress, then the rockmass modulus, E_{RM} is given by:



$$\frac{1}{E_{RM}} = \frac{1}{E_L} + \frac{1}{K_N S}$$

For anisotropic rockmasses, such as illustrated above, the stiffness obtained in the above equation applies to the direction perpendicular to the joint surfaces. The modulus parallel to the joints (e.g. parallel to an undamaged foliated hangingwall with minimal cross-jointing) will be better represented by the intact rock modulus.

Models have been presented (Amadei and Goodman, 1981; Gerrard, 1982) to calculate rockmass moduli (isotropic and directional) which incorporate multiple joint sets. These models normally require measurement and input data which is beyond the scope of mining operations and most construction projects.

Many large scale techniques have been developed to measure elastic modulus and/or deformation modulus (ratio of applied stress to total of inelastic and elastic strain) in the field (Bieniawski, 1978; Barton et al., 1980; Goodman, 1980; Grimstad and Barton, 1993; Rocha and da Silva, 1970; Rocha, 1970; Serafim and Pereira, 1983). These techniques include:

- Radial Jacking Test (I.S.R.M., 1979)
- Flat Jack Tests (I S.R.M., 1986)
- Goodman Jack (Goodman, 1972; Hustrulid, 1976)
- Plate Bearing Tests (I.S R M., 1989)
- Borehole Dilatometer (I S.R.M., 1987; Hyett et al., 1992; Yow, 1993)
- Large Scale Triaxial Tests (Müller, 1974; van Heerden, 1975)
- Petite Seismique Technique (Heuzé, 1980; Bieniawski, 1979)

Back analysis techniques using modelling and instrumentation have also proven useful for estimation of rockmass modulus:

- Tunnel Relaxation (Waddell et al., 1970)
- Pillar Monitoring (Wagner, 1974)

Rockmass classification schemes provide a practical alternative and facilitate preliminary estimates of rockmass modulus (± 50 %). Correlations between *Q*, *RQD* and *RMR* have been made using many of the techniques listed above. A summary of this body of work is given on the following pages.

Rockmass modulus from rockmass classification : Case histories

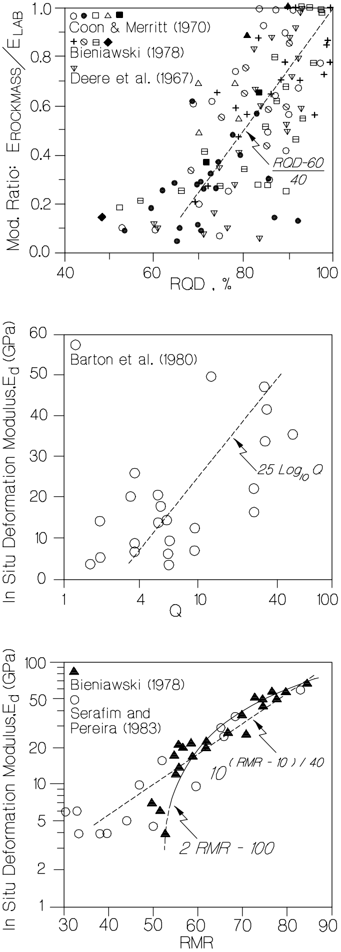


Figure 2.15.3: Rockmass modulus from classification

Many attempts have been made by researchers and engineers to relate rockmass classification results to rockmass modulus as measured by a wide variety of field testing techniques.

RQD provides a measure of the percentage of a rockmass volume which can be expected to behave in manner similar to a lab sample. There is therefore a relationship between *RQD* and the modulus ratio; the ratio between the modulus of the rockmass and that of a standard lab sample. Note the scatter, however, in this graph.

Barton et al. (1980) sought a relationship between *Q* and modulus. As the data is limited, the scatter is great. Also note that the evaluation of *Q* does not involve the intact rock properties even though the intact rock modulus must govern at higher values of *Q*. *Q* does incorporate a measure of the clamping stress which has a direct influence on modulus of fractured rock.

RMR incorporates the compressive strength of rock which is related to modulus (Deere, 1968). Two alternative curve fits from different authors are shown and seem valid for *RMR* > 50.

In all cases on this page the applicability limits of the fitted curves must be respected.

Rockmass modulus from rockmass classification: Recommendations

The figure at right gives crude limits for modulus-ratio estimation using *RQD*. Note that higher stresses tend to close fractures which in turn increases the overall modulus. In moderate to high stress environments and in virgin ground, use the upper design zone. In loose, destressed or disturbed ground, use the lower zone. The centre line represents an expected relationship for a tight but not overstressed rockmass. In anisotropic rock *RQD* must be taken in the direction of interest.

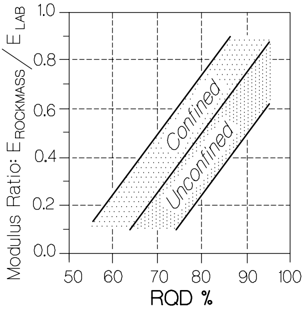


Figure 2.15.4: Modulus vs RQD

Below is a suggested range of absolute rockmass modulus with respect to *Q'* > 1 and *RMR* > 40. Note that *Q'* (*RQD/Jn x Jr/Ja*) is used here. *RMR* should not include the joint orientation correction. In anisotropic rock, measure *RQD* and spacing in the direction of interest. Use the design zones as shown to account for the degree of stress and clamping in situ. The rockmass modulus is limited to a maximum defined by the Young's Modulus (E_{T50}) of an undisturbed laboratory sample. Use the E_{T50} directly for *RMR* > 85 or *Q'* >100.

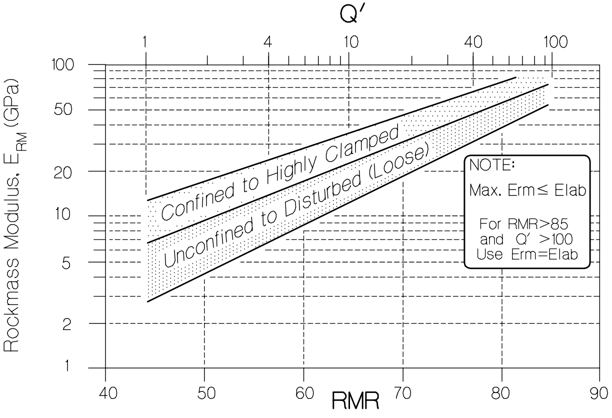


Figure 2.15.5: Rockmass Modulus vs Q' and RMR

2.16 Empirical Design

Rockmasses can be extremely complex media for construction. It is often difficult to apply mechanistic (based on physical mechanisms) analysis tools to the design of excavations in rock. Rockmass classification methods have been calibrated to provide alternative tools for this purpose. Classification and its application to underground support design is primarily founded in civil engineering tunnel construction. This is particularly true for *RQD* (Deere et al., 1967), *RMR* (Bieniawski, 1976, 1989, 1993) and *Q* (Barton et al.,1974; Barton, 1988; Grimstad and Barton, 1993). Laubscher and Taylor (1976) and Laubscher (1993) modified *RMR* for use in the design of block caving mines. In addition, Mathews et al. (1981) and Potvin (1988) developed an extension of the *Q* system and applied it to open stope design. Potvin's method is described separately in Section 2.17 as it is specifically applicable to cablebolt design.

This section outlines some applications of *RQD*, *RMR* and *Q* to the determination of unsupported excavation stability, stand-up time, general support recommendations and specifically, to cablebolt design. It is important to understand the origin of these empirical support methods. That is, they are primarily based on tunnels at low to moderate depth (0 to 500 m). Civil tunnels must be completely stable (no local block fallout) and must endure for many years or decades. Recommendations for stability and support may therefore not be directly applicable to mining. Wherever possible the authors of this handbook have attempted to adapt the recommendations for mining openings and for cablebolt support.

Cablebolt densities (number of cablebolts per unit face area) and bolt spacings are primarily based on overall support pressure requirements (support load or support capacity per unit face area) which have been derived for rockbolts. The bolt densities prescribed for cablebolts are reduced by a factor corresponding to the increased unit capacity of a cablebolt strand as compared to that of a rockbolt. This increased spacing may allow local block fallout to occur between cables. It is recommended that the reader refer to Stillborg (1986), Choquet (1991), Hoek and Brown (1980), Hoek et al. (1995), or other rock support guides for recommendations for rockbolting. Rockbolts and screen or straps should be used in combination with the cablebolts to arrest this local unravelling where necessary.

In addition, the bolt lengths recommended by most empirical guidelines refer to mechanically anchored rockbolts or resin grouted bolts. These devices have fixed attachment points (anchor and head) or have highly adhesive bonds which generate full load capacity over short anchor lengths (< 1 m). Cablebolts, however, transfer their load to the rockmass over larger bond lengths. In addition, the top of a cablebolt hole may contain a void of 0.5 to 1 m depending on the installation method and quality control. For this reason a *minimum* anchor length of 2 m has been added to all length guidelines to adjust them for cablebolting applications.

2.16.1 Rock Quality Designation, *RQD*

Deere et al. (1967, 1969) have developed tunnel support guidelines for different excavation methods based on *RQD*. These recommendations have limitations due to the limited scope of *RQD* as a rockmass quality indicator. Clearly, the influence of joint condition, rock strength and confinement (field stress) are ignored in the calculation of *RQD*. As a crude measure of structural integrity, *RQD* can be a convenient tool for preliminary design. For tunnel spans of 6 to 12 m, Deere et al. (1969) proposed the approximate relationship:

$$\textit{Bolt Spacing (m) for a Square Pattern} = 0.02 \times RQD\ (\%)$$

This relationship implies that cablebolts as primary support of tunnels > 5 m wide should be economically feasible for *RQD* values greater than 70% (*Fair* to *Excellent* Rock) if 1.4 m is taken as the practical minimum cable spacing. A tight pattern of mechanically anchored bolts and mesh are recommended below this value. Shotcrete becomes a competitive support option below *RQD* = 60%.

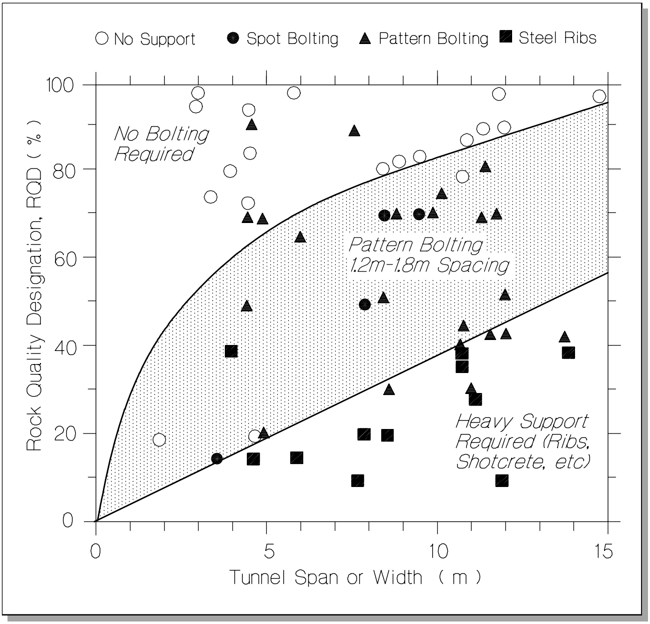


Figure 2.16.1: *RQD*-based stability and support guidelines (after Merritt, 1972)

2.16.2 Rock Mass Rating, *RMR*

No-Support Limits and Stand-up Time

The Rock Mass Rating, *RMR* was originally developed by Bieniawski (1973) and updated in 1979. Other authors have modified *RMR* for specific applications:

Mining:
  Laubscher (1977, 1993); Kendorski et al. (1983)
Coal Mining:
  Ghose and Raju (1981); Newman (1981); Sheorey (1993); Unal (1983); Venkateswarlu (1986)
Slope Stability:
  Romana (1985, 1993)

In Figure 2.16.2, Bieniawski (1993) presents the revised chart relating Span and Stand-up time with his 1989 Rock Mass Rating System. The points in this graph represent groundfalls in tunnels and in mining excavations. The concept of stand-up time was originally conceived by Lauffer (1958, 1960), to represent the duration of time within which an excavation will remain serviceable and after which significant instability and caving occurs.

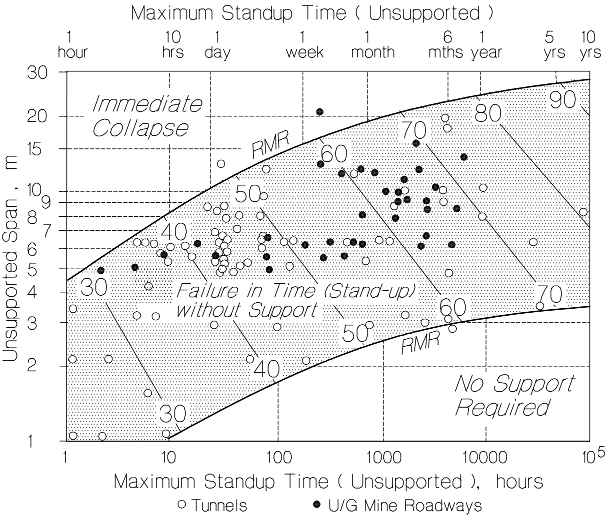


Figure 2.16.2: Unsupported Tunnel Limits (after Bieniawski, 1993, 1989).

In order to use Figure 2.16.2, first determine the *RMR* for the rockmass in question. The intersection of a specified *RMR* contour with the bottom of the shaded zone gives the maximum span which can remain stable indefinitely without support. Within the shaded zone, the *RMR* contour line gives the anticipated stand-up time without support. Above the shaded zone (e.g. a 20 m span with *RMR*=60) unsupported excavations will disintegrate shortly after development. Note the range of data for which this relationship was derived.

In order to present these guidelines in a manner consistent with other systems, Figure 2.16 2 has been replotted with *RMR* on the horizontal axis, as shown in Figure 2.16.3. For a temporary mining opening such as a 10 m topsill (e.g. with a required stand-up time of 1-2 months) it can be seen that a rockmass with a Rock Mass Rating of greater than 65 may not need support (apply an appropriate safety factor - multiplier of 2) with the exception of pinned screen for personal safety.

Note that poor blasting can reduce *RMR* by up to 20% (Bieniawski, 1989). Following logic developed by Barton et al. (1974) and Barton (1994) for the *Q* system, *RMR* can be increased by up to 10% (*RMR* > 30%) for near vertical stope walls. Note that the full *RMR* including joint orientation adjustment is used here.

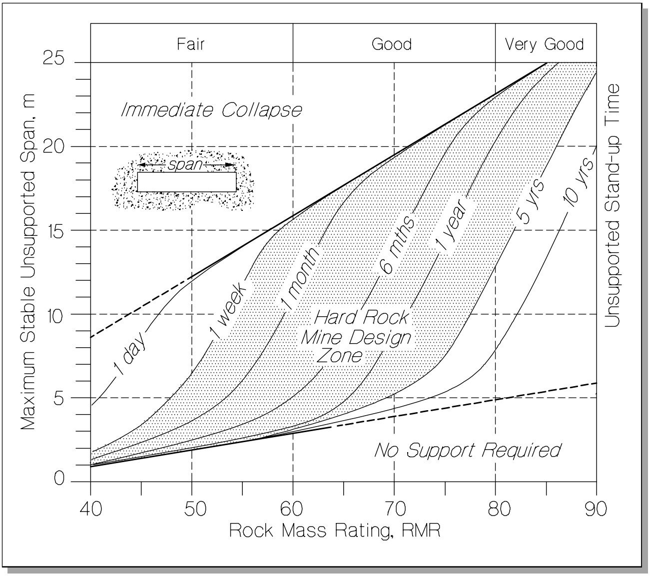


Figure 2.16.3: Alternative representation of Figure 2.16.2 stand-up time guidelines

RMR - Support Guidelines

Bieniawski (1979, 1993) presents support guidelines for a 10 m wide horseshoe shaped, drill and blast tunnel under 25 MPa of vertical stress:

Table 2.16.1: Support recommendations from *RMR* (after Bieniawski 1993)

RMR	*Excavation (Horseshoe, 10m Span)*	*Combined Permanent Support*		
		20mm Rockbolts Fully Grouted	*Shotcrete*	*Steel Sets*
81-100	Full face, 3m advance.	None (Spot Bolting if req'd).	None	None
61-80	Full face, 1.5m advance. Install support 20m from face.	Bolts in crown 3m long, 2 5m spacing, Some mesh.	50mm as required in crown.	None
41-60	Top heading & bench, 1.5-3m advance in top heading with rapid support. Full support 10m from face.	Systematic bolts 4m long, 1 5-2m spacing in walls and crown. Mesh in crown.	50-100mm in crown and 100mm in sides.	None
21-40	Top heading & bench, 1.0-1.5m advance in top heading with immediate support. Full support within 10m of face.	Systematic bolts 4-5m long, spaced 1-1.5m in crown and walls with wire mesh.	100-150mm in crown and 100mm in sides.	Light to medium ribs spaced 1.5m where required.
< 20	Multiple drifts 0.5-1.5m advance in headings. Install full support immediately.	Systematic bolts 5-6m long, 1-1.5m spacing in crown and walls with wire mesh. Bolt invert.	150-200mm in crown. 150mm in sides and 50mm on face.	Medium to heavy ribs spaced 0.75m with lagging and forepoling. Close Invert.

This excavation could be loosely equated to a typical mining haulageway at moderate depth up to 1000m, although the support recommendations will be very conservative for such an application.

Note that the bolt spacings and shotcrete thicknesses, etc. in this table are specified for a combination support system as listed. Do not extract, for example, bolt spacings from this table for use as a single component system as this will result in under-designed support.

RMR - Semi-Empirical Support Guidelines

RMR has been adapted by various authors for support design. Figure 2.16.4 illustrates one such development by Unal (1983). The concept is simple and yet it produces reasonable results for cablebolt length and moderately conservative (for mining applications) recommendations for cablebolt density. For extremely poor rocks (*RMR* < 10), the height of the zone requiring support is assumed to be equal to the span. This height is modified by *RMR* as shown until the rock is completely self-supporting at *RMR*=100%. Support pressure is the amount of distributed load applied to the surface of the excavation (roof in this case) to resist further displacement of the rockmass. It is assumed that the cablebolts used here are pre-installed or installed at the excavation heading immediately after blasting and that stiff systems are in use (modified strand or well grouted and plated plain strand cablebolts).

The cablebolt density as plotted here refers to the quantity of complete cablebolts (single or double strand cablebolts) per square meter of excavation face area. This is a convenient measure of cable distribution since it gives support pressure directly when multiplied by the capacity or tension in the steel cable. Cablebolt length is simply calculated as the height of the supported rock zone with an extra 2 m added to provide a minimum reliable anchor for the fully grouted cablebolt. Cablebolts typically have at least 0.5 m of poorly placed grout at the top of an uphole. This inactive length increases when quality control is poor or when a quantity of grout flows away into fractures after placement, reducing the grouted column height. If either condition is suspected, increase this anchor length accordingly. 2 m represents the minimum prudent anchor design.

The cablebolt densities in Figure 2.16.4 are calculated for a rock specific weight of 26 kN/m^3 and for steel capacities of 20 tonnes (200 kN) for single strand and 40 tonnes for double strand cablebolts. These values correspond to the onset of inelastic yield and should be used for permanent installations. For temporary and non-critical openings, 25 and 50 tonnes can be used respectively, corresponding to ultimate breaking strength of cables. This results in a 20% decrease in cablebolt density as noted in Figure 2.16.4.

Stimpson (1989) further developed this concept of a supported height, incorporating the influence of in situ stress ratio and excavation height:span ratio. The shape of the loosening zone becomes an ellipse with its long axis oriented in the direction of major principle stress. this is due to the confining effect of higher stresses. Note that the opposite trend will be observed if the stresses are high enough to cause rockmass failure. In this case the ellipse will be oriented with the long axis perpendicular to the major principle stress. The modified loading height is then modified as a function of *RMR*. Detournay and St. John (1988) present a method for calculating the depth of failure around deep circular openings in anisotropic stress fields. *RMR* can be used to obtain rock strengths for this model.

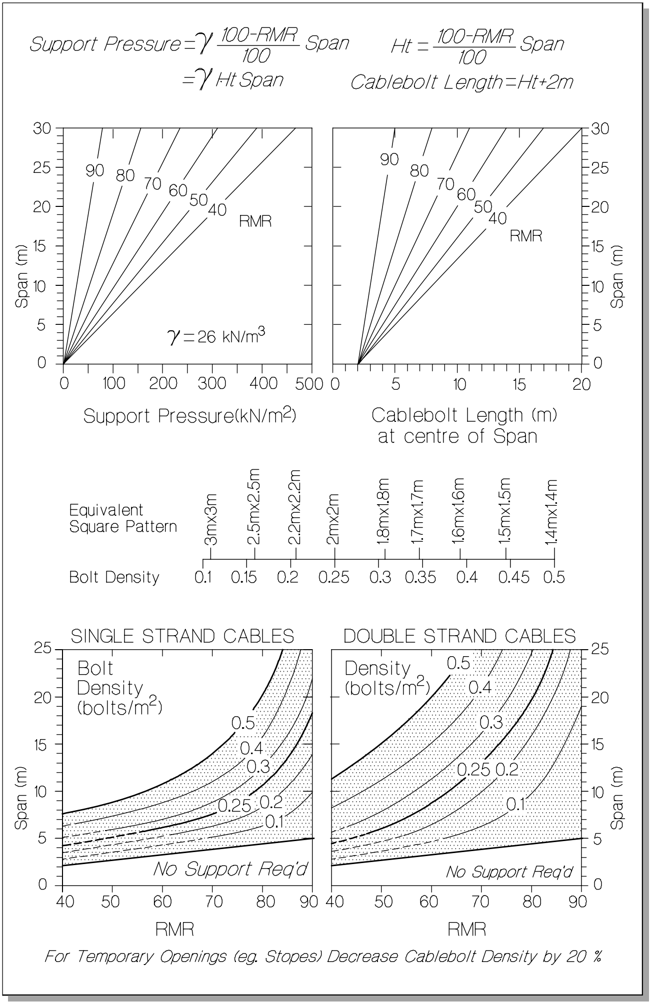


Figure 2.16.4: Tunnel support pressure, cablebolt length and density guidelines with respect to span and *RMR* (based on Unal, 1983)

2.16.3 Rock Tunnelling Quality Index - *Q*

No Support Limits

Barton et al. (1974), Barton (1988, 1994) describe the application of the *Q*-system for rockmass classification to the determination of no-support limits for various types of excavations. Approximately 200 case examples were originally classified to originally calibrate this system. Since then over two thousand new empirical tunnel and large cavern designs have been successfully carried out (Barton et al , 1992). Figure 2.16.5 shows the original database of supported and unsupported excavations. The shaded zone represents the limits of practical support application. The lower boundary of this zone is the limit of stability for unsupported excavations of a given Equivalent Span, *ES* = *Span/ESR*, where:

Table 2.16.2:

Type of Excavation *(after Barton, 1988)*	*Number of Cases*	*ESR*
Temporary mine openings.	2	**approx. 3-5 ?**
Permanent mine openings; Low pressure water tunnels; Pilot tunnels; Drifts and headings for large openings.	83	**1.6**
Storage caverns; Water treatment plants; Minor road and railway tunnels; Surge chambers; Access tunnels, etc.	25	**1.3**
Power stations; Major road and railway tunnels; Civil defense chambers; Portals; Intersections.	79	**1**
Underground nuclear power stations; Railway stations; Sports and public facilities; Factories.	2	**approx. 0.8 ?**

Excavation Support Ratio, *ESR* is a factor used by Barton to account for different degrees of allowable instability based on excavation service life and usage. Divide the span of the excavation by the appropriate *ESR* value to obtain the equivalent span for use in Figures 2.16.5 and 2.16.7.

Note that the number of mining case histories leading to the recommendation of *ESR* = 3 to 5 for temporary mine openings is limited. Based on the authors' experience, **a maximum of 3 is recommended** for mine openings unless local experience justifies an increase.

Certain mining excavations are more critical than others from both an operational and a safety point of view. Figure 2.16.6 provides no-support limits in order of decreasing reliability, relating them to Barton's original *ESR* values. Figure 2.16.6 is plotted against actual excavation span.

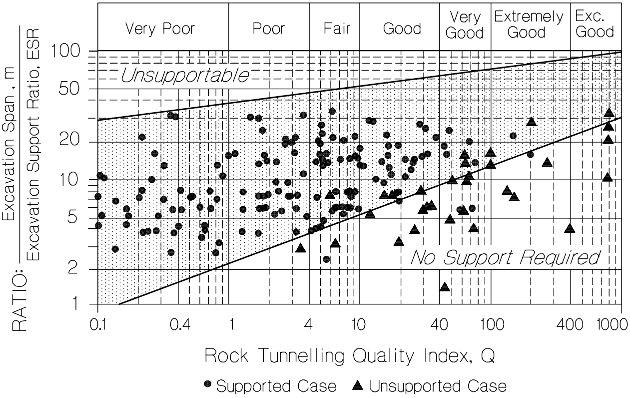


Figure 2.16.5: Case history database for *Q*-System (after Barton, 1988)

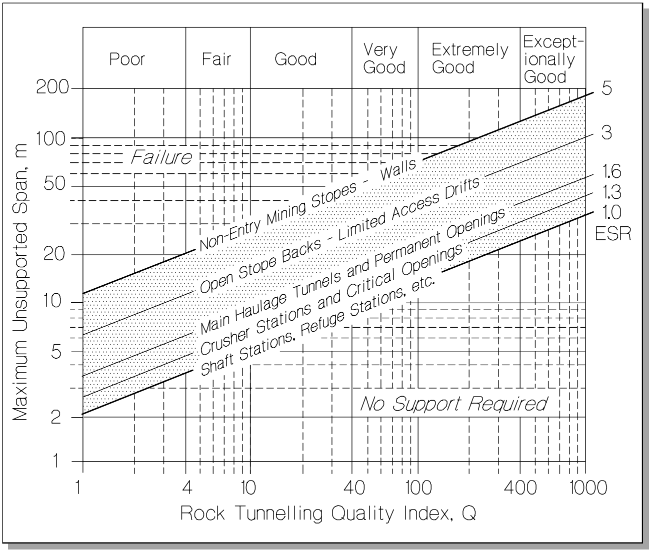


Figure 2.16.6: *Q*-system; No-support span limits for underground mine openings

Q - Support Guidelines

Support recommendations based on the *Q*-system have evolved over the years as more and more case histories have been added to the database. Barton (1988) presented a tabulated series of detailed support recommendations based on different combinations of rock quality, *Q*, and on Equivalent Span (*Span/ESR*). Grimstad et al. (1993) proposed a summary graph based on these recommendations which is designed to accommodate advances in shotcrete technology. A version of this graph is shown in Figure 2.16.7. Again, this graph was developed for permanent support in civil tunnels, shafts and caverns. These recommendations are likely to be too conservative for mining. Cable lengths shown on the right side are valid for *ESR* = 1. For greater values of *ESR*, these lengths should be increased in accordance with actual span. A reasonable rule-of-thumb for mining would be to multiply the lengths shown by $(ESR)^{0.5}$.

Barton et al. (1974) recommend the following adjustments to *Q* for vertical walls (*Qw*) to account for the reduced demand for support on the wall:

$\boldsymbol{Q_w = 5\times Q}$ for $Q>10$, $\boldsymbol{Q_w = 2.5\times Q}$ for $0.1<Q<10$ and $\boldsymbol{Q_w = Q}$ for $Q<0.1$

Caution should be used when combining the above adjustments with large values of ESR (> 2). It is possible that unconservative designs may result.

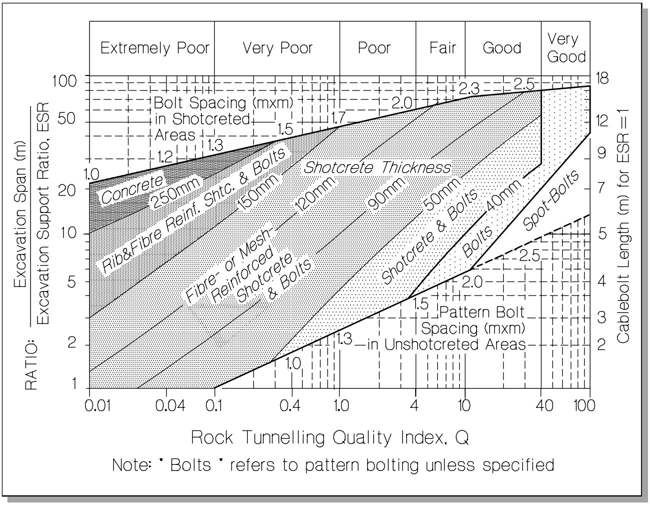


Figure 2.16.7: Tunnelling Support Guidelines (after Grimstad et al., 1993). Bolt lengths have been modified for cablebolting.

Support Pressure and Bolt Spacing

Barton et al. (1974) proposed relationships for support pressure, *p*, in MPa;

For more than 2 joint sets ($Jn > 6$);

$$p = \frac{.2\,Q^{-\frac{1}{3}}}{Jr}$$

For 0 - 2 joint sets ($Jn < 6$);

$$p = \frac{.2(Q^{-\frac{1}{3}})\sqrt{Jn}}{3Jr}$$

If grouted cablebolts are installed prior to or immediately upon excavation of the face in question, this support pressure can be crudely related to the installed cablebolt capacity per unit area of excavated rock face or to an equivalent bolt spacing. This relationship is plotted in Figure 2.16.8, for *Jn*>6, using the initial yield-strength of steel cable (200 kN/strand). A spacing increase of 10% as noted implies the use of the short term breaking-strength (250 kN) and can be used in temporary and non-critical applications. Note that this relationship does not consider excavation span. This implies that a surface reinforcement action is involved creating a self-supporting rock span. The cablebolt must limit internal displacements and therefore must be stiff (modified geometries). In addition, in the case of double-strand bolts and wider spacings, plates and intermediate rock bolting may be required to maintain surface integrity between cables.

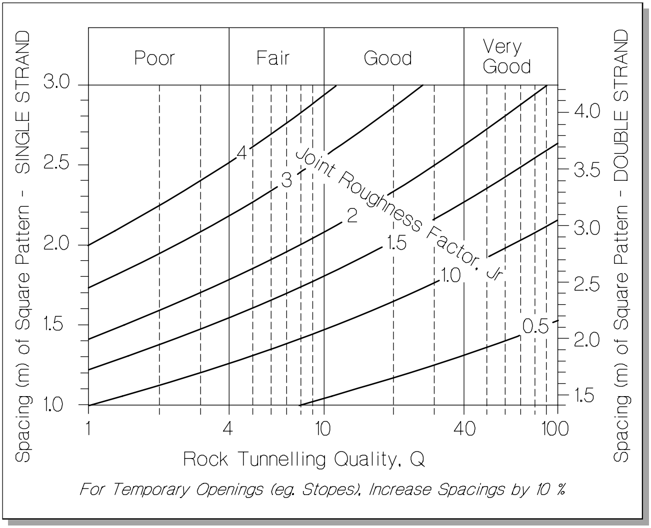


Figure 2.16.8: Cablebolt spacings for mining excavations (*Jn* > 6)

2.16.4 Empirical Cablebolt Design - General Limits

Cablebolt support is ideally suited for moderate to large openings in blocky ground under low to moderate stress. This design zone is shown in Figure 2.16.9 with respect to *RMR*, *Q*, induced stress and rock strength. Cablebolt effectiveness is limited to the following broad conditions:

- Highly fractured and soft rockmasses at low stress levels will tend to unravel between cablebolts spaced within economic limits. More continuous forms of restraint (e.g. mesh) and reinforcement (e.g. shotcrete, rebar) are required as surface retention to make the cablebolt support more effective.
- Highly fractured rocks at elevated stress levels may exhibit squeezing and disintegration which cannot be arrested effectively by cablebolts. Plating is necessary along with intermediate bolting, mesh and/or yielding linings.
- Blocky or Massive rocks at high stress levels are subject to unpreventable spalling and ultimately to violent rupture which cannot be prevented by cables alone. Cables, in combination with other restraint and reinforcement elements can preserve the integrity of the broken rockmass after such brittle failure depending on the severity of the overstress.

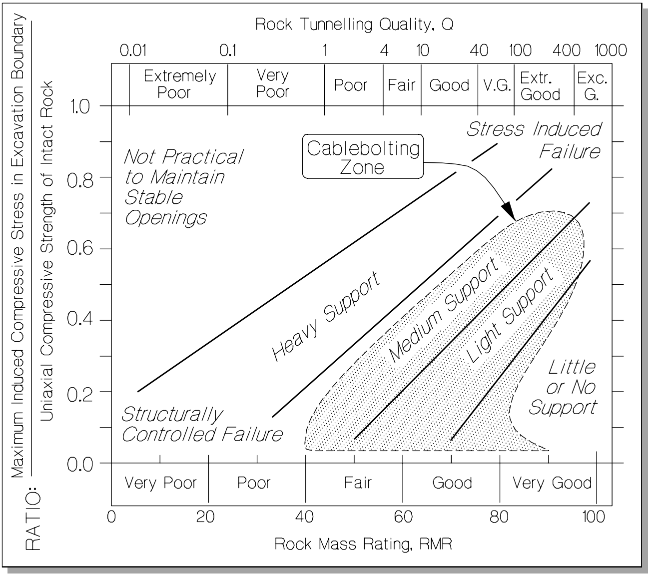


Figure 2.16.9: Limits of cablebolt application (base graph after Hoek, 1981)

2.16.5 Empirical Design - Rules of Thumb

Classification systems serve to differentiate between different rockmasses and to adjust design accordingly. Rules of thumb for support design have been developed for blocky to fractured ground (U S.C.E. 1980; Lang, 1961; Farmer and Shelton, 1980; Coates and Cochrane, 1970; Laubscher, 1984). These are based on tunnels, caverns and mine openings and summarize current practice.

Most of these guidelines are designed for rockbolting (mechanical or resin grouted) and as such can be used to select spacings for face support to supplement cablebolting in fractured ground. In many cases the recommended spacings will not be economically practical for use directly with cablebolts.

The lengths quoted in these rules of thumb should be adjusted for cables by adding a minimum of two (2) extra metres of embedded length (unless it is indicated that this adjustment has already been made by the authors as is the case in Figure 2.16.10). Extrapolating to obtain cable lengths for spans greater than those shown in these figures is not recommended. The figure boundaries represent the applicability limits based on the source data. Figure 2.16.10 illustrates a data set of rockbolt lengths in existing tunnels and caverns.

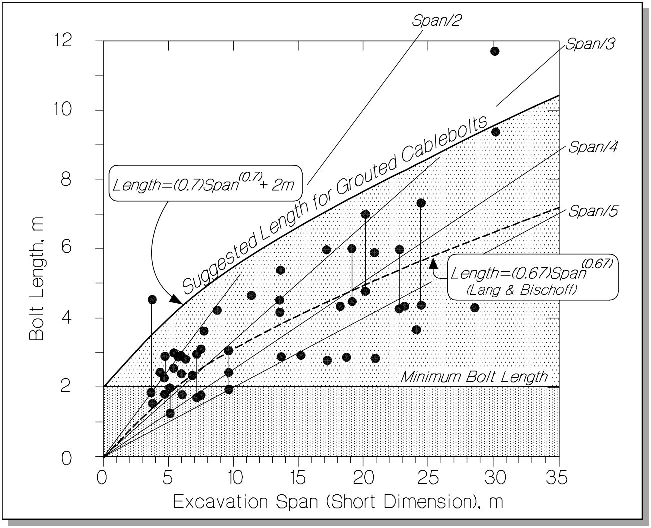


Figure 2.16.10: Bolt lengths in current practice (after Lang and Bischoff, 1984) with adjustment for cablebolt application (relationships are for S.I. units)

The U.S. Army Corps of Engineers (1980) developed a suite of simplified recommendations for rockbolt spacing, length and support pressure, summarized in Table 2.16.3. Rock loads are based on support pressure from actively tensioned mechanical bolts and may be inappropriate for cablebolting. These guidelines, like all others in Section 2.16 should be used in conjunction with other design tools.

Table 2.16.3: Bolting Guidelines (after U.S.C.E., 1980)

Maximum Spacing Recommendations (U S.C E., 1980):

Least of:

- 0.5 times the bolt length
- 1.5 times the width of critical and potentially unstable rock blocks
- 1.8 m (applies only to rockbolts where screen is to be attached)

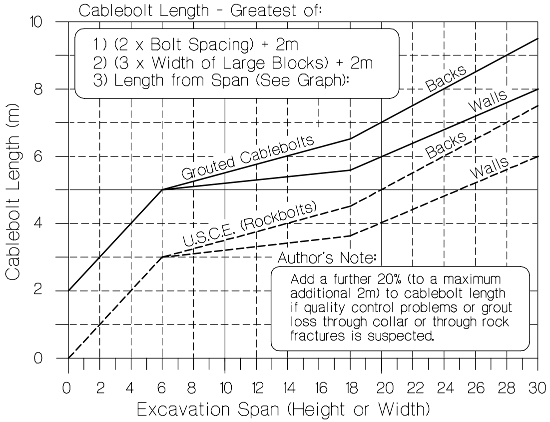
Minimum Spacing (U.S.C.E., 1980)

0 9 to 1.2 m (for cablebolts, 1 4 m is normally the economic limit)

Minimum Average Expected Bolt Loads (Applies Only to Stiff Cable Systems)

Roofs / Backs: Equivalent to weight of slab with thickness 0.2 times the span
Sidewalls: Equivalent to weight of slab with thickness 0.1 times the height

Minimum Bolt Length

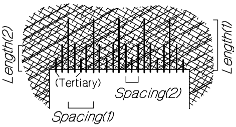


Cablebolt Length - Greatest of:
1) (2 x Bolt Spacing) + 2m
2) (3 x Width of Large Blocks) + 2m
3) Length from Span (See Graph):

Author's Note:
Add a further 20% (to a maximum additional 2m) to cablebolt length if quality control problems or grout loss through collar or through rock fractures is suspected.

Farmer and Shelton (1980) collected case histories from numerous authors and formulated the recommendations for rock bolting in Table 2.16.4. Of particular interest are the comments regarding support function and design philosophy.

Table 2.16.4: (after Farmer and Shelton, 1980)

Span (m)	*Number & (Dip °) of Joint Sets*	*Bolt Recommendations*	*Comments (after Farmer and Shelton & by authors of this handbook)*
<15	1 to 2 (0 to 45)	*Length = 0.3 x Span* *Spacing < 0.5 x Length* Install bolts perpendicular to lamination with mesh to prevent flaking. Decrease spacing in weak strata.	Bolting creates load carrying beam over span. Grouted bolts or modified cable strand create higher joint shear stiffness. Tension bolts (plate cables) in weak rock. Angle bolts where joints are vertical.
<15	1 to 2 (45 to 90)	For wall bolts: Installed at 90 to lamination *Length > Height x cos (Dip)* Installed Horizontally *Length > Height / tan (Dip)* (*Dip* = dip of joints)	Roof bolting as above. Side bolts designed to prevent sliding along planar joints. Spacing should be such that bolt capacity is greater than sliding or toppling weight. Tension bolts (plate cables).
<15	>2 with tight & clean surfaces	*Length > 2 x Spacing* *Spacing < 3 to 4 x Block Size* Install bolts perpendicular to lamination with mesh to prevent flaking. Decrease spacing in weak strata.	Bolts should be installed quickly after excavation to prevent loosening and retain tangential stresses. Tension and plate to improve radial confinement. Sidewall bolting where wedge toes daylight into excavation.
>15	< 2	Alternate Primary (1) and Secondary (2) Bolting: *Length(1) > 0.3 x Span* *Spacing(1) < 0.5 x Length(1)* *Length(2) > 0 3 x Spacing(1)* *Spacing(2) < 0.5 x Length(2)* Mesh to prevent spalling	Primary bolting supports span and major blocks. Secondary bolting retains surface blocks. Limit spacings (and provide load capacity) accordingly. Bolt or cable lengths should penetrate beyond extent of known discrete wedges.
>15	>2 with tight & clean surfaces	Alternate Primary (1) and Secondary (2) Bolting: *Length (1) > 0.3 x Span* *Spacing(1) < 0.5 x Length(1)* *Spacing(2) < 3 to 4 x Blk. Size* *Length(2) > 2 x Spacing(2)* Mesh as required for surface block retention	

2.17 Empirical Design of Open Stopes and Support: Mathews/Potvin Stability Graph Method

Classical empirical tools such as *RMR* and *Q* were developed from a database composed primarily of civil engineering tunnels at low to moderate depth. These tools have proven invaluable to the tunnelling engineer. The recommendations derived from these systems for dimensioning and support, however, often result in conservative designs for large temporary or non-entry mining excavations.

While these systems are appropriate for high traffic mining roadways, lunchrooms and equipment rooms where stability must be paramount, they are difficult to apply to the problem of dimensioning and support design for large open stopes. These limited access areas can be designed as temporary structures and in the case of non-entry stopes, can tolerate limited local fallout of small rock blocks provided that dilution is minimized and overall stability is maintained. These criteria permit a more economical design suitable to mining.

RMR (Bieniawski, 1989; 1993) allows for design modification based on reduced stand-up times for mining while *Q* (Barton et al., 1974) attempts to include mining applications through the use of Equivalent Support Ratio. Laubscher and Taylor (1796) modified *RMR* and introduced a classification system for caving operations and for stability of mining excavations. Readers are referred to Hoek et al. (1995) for additional discussion of these methods.

Large scale open stoping methods such as Vertical Crater Retreat, AVOCA, Longhole and Blasthole Stoping rely on the selection of a limiting stope dimension. Ideally these stopes can be designed to be self supporting. When ground conditions or the need for larger stopes mandates the use of support, cablebolting is the most logical choice and has been successfully applied. Mathews et al. (1981) proposed an empirical method for the dimensioning of open stopes based on Q' and on three factors accounting for stress, structural orientation and for gravity effects. The method is used to dimension each face of the stope separately based on a combination of these three factors and on the hydraulic radius (calculated as *surface area / perimeter*) of the face. The hydraulic radius accounts for shape as well as size of the face.

Potvin (1988) modified this original method and calibrated it using 175 case histories. Nickson (1992) added case histories and further investigated Potvin's support design guidelines. These case histories include hangingwalls, footwalls, ends and backs from a wide variety of mining environments. Other case histories can be found throughout recent literature (Bawden, 1993; Bawden et al. 1989; Greer, 1989). The method has been expanded by the authors in this handbook to provide improved support guidelines.

2.17.1 Modified Stability Number, N'

The classification of the rockmass and of the excavation problem itself is accomplished in the Modified Stability Graph Method through the use of the Modified Stability Number, *N′*, as specified by Potvin (1988), Potvin and Milne (1992) and Bawden (1993). This parameter is similar to the value *N* proposed by Mathews et al. (1981) but has different factor weightings. Canadian mines use Potvin's *N′* while at present mines in Australia, for example, use Mathews' analysis and *N*. Only *N′* (Potvin) will be considered here. This method has been referred to as the *Potvin* method, the *Mathews/Potvin* method, the *Modified Stability Graph* method and the *Stability Graph* method. The latter label will be used for the rest of this discussion for clarity and brevity.

N′ is based initially on *Q′*, where;

$$Q' = \frac{RQD}{Jn} x \frac{Jr}{Ja}$$

and where;
RQD/Jn is a measure of block size for a jointed rock mass
Jr/Jn is a measure of joint surface strength and stiffness

Modified Stability Number *N′*;

$$N' = Q' \, x \, A \, x \, B \, x \, C$$

where;

A is a measure of the ratio of intact rock strength to induced stress. As the maximum compressive stress acting parallel to a free stope face approaches the uniaxial strength of the rock, factor *A* degrades to reflect the related instability due to rock yield.

B is a measure of the relative orientation of dominant jointing with respect to the excavation surface. Joints which form a shallow oblique angle (10-30°) with the free face are most likely to become unstable (i.e. to slip or separate). Joints which are perpendicular to the face are assumed to have the least influence on stability.

C is a measure of the influence of gravity on the stability of the face being considered. Overhanging stope faces (backs) or structural weaknesses which are oriented unfavourably with respect to gravity sliding have a maximum detrimental influence on stability.

Table 2.17.1: Range of values (*for hard rock mining):

Range	*RQD/Jn*	*Jr/Ja*	*A*	*B*	*C*	*N′*
Maximum	0.5 - 200	0.025 - 5	0.1 - 1	0.2 - 1	2 - 8	**0.0005 - 8000**
*Typical**	2.5 - 25	0.1 - 5	0.1 - 1	0.2 - 1	2 - 8	**0.1-1000**

2.17.2 Stability Graph Method - Input Parameters

Compute the value of hydraulic radius, *HR*:

$$\bullet \quad HR = \frac{Area\ (m^2)}{Perimeter\ (m)} = \frac{w \times h}{2(w+h)} \qquad (units\ of\ m)$$

where *A* and *B* are the two dimensions defining the stope face to be analyzed.

Compute the modified stability number, *N'*:

- Measure or calculate the value of *RQD, Jn, Jr* and *Ja* as described in Section 2.14.5
- Compute *Q'=RQD/Jn x Jr/Ja.*

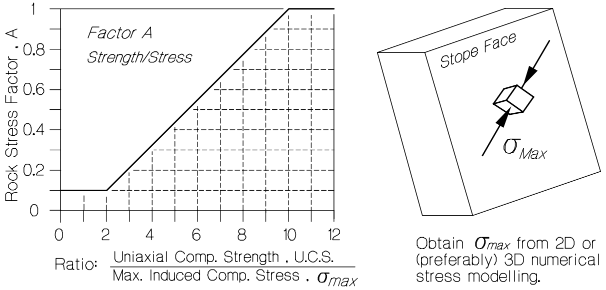
From the charts that follow:

- Evaluate Rock Stress Factor *A*.
- Evaluate Joint Orientation Factor *B*
- Evaluate Gravity Adjustment Factor *C*
- Obtain *N'=Q' x A x B x C*

Plot point (*HR,N'*) on stability graph and determine stability and design zone.

Rock Stress Factor A

Determine maximum induced tangential stress (compressive) acting at the centre centre of the stope face being considered. Obtain uniaxial compressive strength strength for the intact rock. Evaluate Stress Factor, *A*, using the graph below:



Obtain σ_{max} from 2D or (preferably) 3D numerical stress modelling.

Figure 2.17.1: Rock Stress Factor *A* (Potvin, 1988) for Stability Graph analysis

Joint Orientation Factor, B

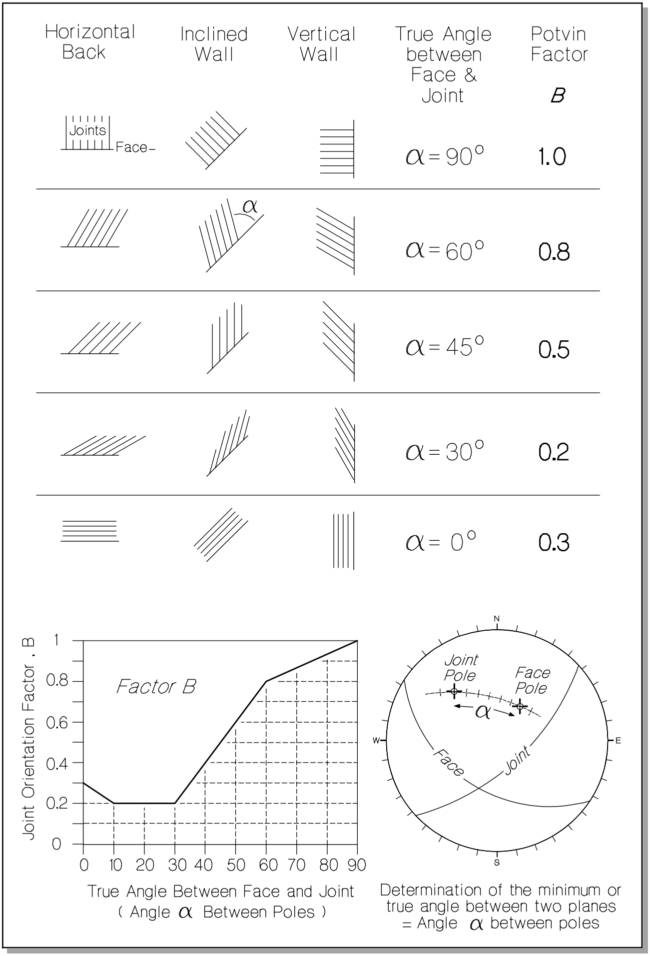


Figure 2.17.2: Determination of Joint Orientation Factor, B, for Stability Graph analysis

Joint Orientation Factor, B: Example Determination

The true angle between two planes is not immediately given by the relative dips and strikes of the planes. It must be calculated as shown on the following page or estimated from a stereonet as in this example.

Consider the hangingwall face and associated joint sets (Figure 2.17.3a). Determination of *B* involves only the pole to the face and the mean poles for each joint set 1, 2 and 3.

Using a series of small circles (cones) centred on the face pole, the angle (cone angle) from this pole to each of the joint set poles can be estimated as in Figure 2.17.3b). These small circles (cones) can be generated by hand (Goodman, 1980; Priest, 1985) or they may be automatically generated by a computer program such as DIPS (Hoek et al., 1995) as shown here. Cones drawn at 10, 30, 45, 60, and 90 degrees provide sufficient resolution to determine factor *B*.

The true angle between planes is given by the smallest angle between poles to the planes. Figure 2.17 3.b) illustrates how to determine that the angle from the face to set 1 = 20°,to set 2 = 53°, and to set 3 = 71°.

In Figure 2.17.3c), the angle contours have been replaced by corresponding Joint Orientation Factors (B). This shows clearly that joint set A is critical and that the factor, *B*, should be set to 0.2 for the Stability Graph analysis.

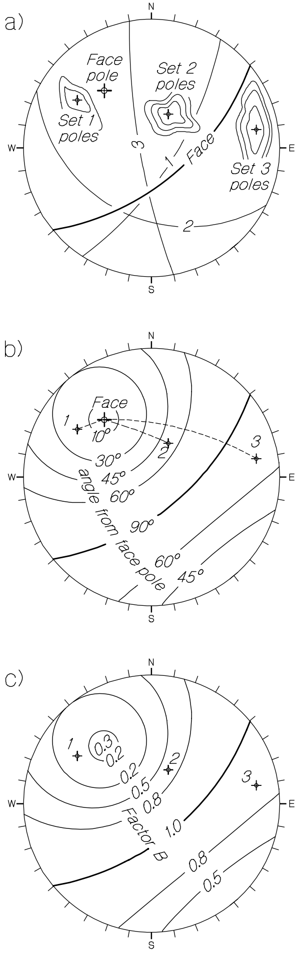


Figure 2.17.3: Estimation of true interplane angle and Joint Factor B

Joint Orientation Factor, B: Direct Calculation of Interplane Angle

It is possible to determine directly the true interplane angle between the stope face (wall plane) and the joint plane using the following simple procedure.

Given the *Dip* and the *Dip Direction* for a plane, the *Trend* and *Plunge* of the corresponding pole (normal vector) can be calculated:

$$T = Trend = Dip\ Direction + 180°$$
$$P = Plunge = 90° - Dip$$

For a stope wall plane, *w*, and a joint plane, *j*, the direction cosines with respect to the global coordinate grid (North, East, Down) are denoted by *N, E* and *D* respectively and are calculated as follows:

For the stope wall:

$$N_w = cos(T_w) \cdot cos(P_w)$$
$$E_w = sin(T_w) \cdot cos(P_w)$$
$$D_w = sin(P_w)$$

For the joint plane:

$$N_j = cos(T_j) \cdot cos(P_j)$$
$$E_j = sin(T_j) \cdot cos(P_j)$$
$$D_j = sin(P_j)$$

Next calculate the dot product, $\mathbf{w}\cdot\mathbf{j}$, between the wall face and the joint plane:

$$\mathbf{w}\cdot\mathbf{j} = N_w\ N_j + E_w\ E_j + D_w\ D_j$$

Finally, the true interplane angle, α, is given by:

$$\alpha = cos^{-1}(\mathbf{w}\cdot\mathbf{j}) = acos(\mathbf{w}\cdot\mathbf{j})$$

This calculation can easily be solved using a calculator or can be implemented in a spreadsheet or computer program.

Once this true interplane angle is calculated, it is possible to assign a Joint Orientation Factor, *B*.

Joint Orientation Factor, B
Simplified approach (special cases)

It is important to remember that measurements such as Dip and Dip Direction or Strike are made relative to a global coordinate system. They cannot be used directly to calculate the true angle between two planes since the applicable coordinate system must be changed to be relative to one of the faces. Therefore the procedures discussed on the previous pages must be implemented.

The calculation of interplane angle is simplified, however, when one of the planes is approximately horizontal or near vertical (Dip ≈ 0 or Dip ≈ 90). In the case of true angle calculation for determination of Factor, *B*, this condition must apply to either the stope face or the joint plane (or both).

Horizontal Joint or Horizontal Stope Face (Back):

Consider only the difference in Dip between the stope face and the joint plane using the graph at right to determine *B*. When one plane is approximately horizontal, then the difference in Dip approximates the true interplane angle.

Near Vertical Joint or Near Vertical Stope Face:

The difference in Strike (or in Dip Direction) must also be considered in the case of vertical features. Note that this relationship as presented by Potvin (1988) should only be used when one of the planes is near vertical.

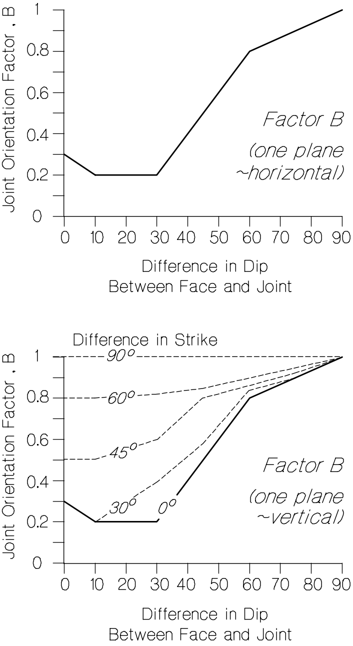
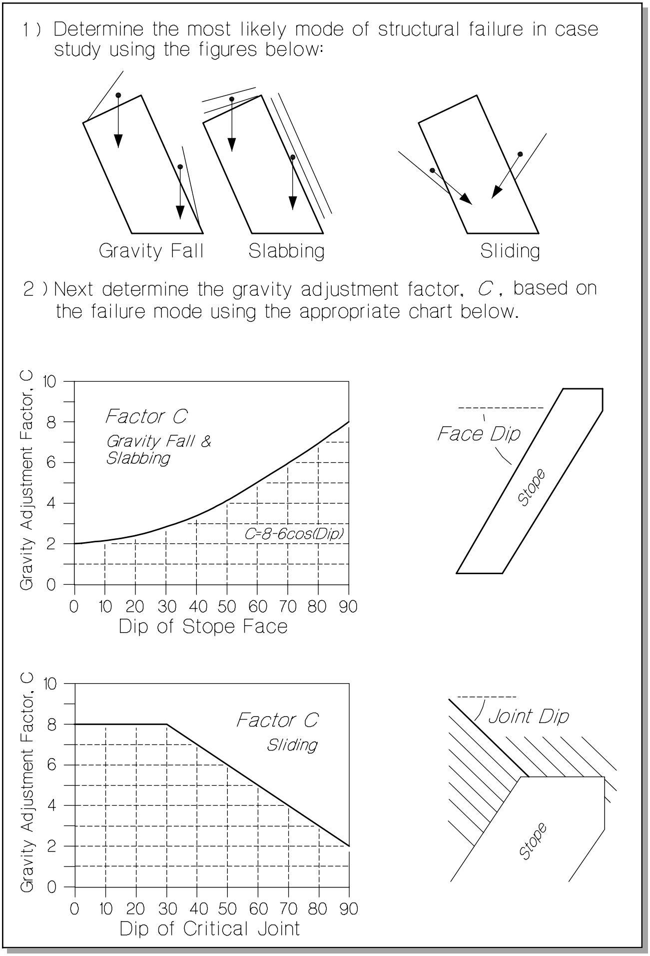


Figure 2.17.4: Simplified special cases for determining factor B

Gravity Adjustment Factor, C



1) Determine the most likely mode of structural failure in case study using the figures below:

2) Next determine the gravity adjustment factor, *C* , based on the failure mode using the appropriate chart below.

Figure 2.17.5: Determination of Gravity Adjustment Factor, C, for Stability Graph analysis

Hydraulic Radius

Before proceeding with the application of the Stability Graph, it is necessary to understand the nature of the hydraulic radius, *HR*. In short, *HR* is calculated by dividing the area of a stope face by the perimeter of that face as shown at right.

Most classification systems (e.g. *RMR* and *Q*) define stability and support zones with respect to a single value of span. This is because these methods are derived from tunnelling databases in which the long span can be assumed to be infinite and in which the short span is therefore the critical dimension. If this short span is kept constant and if the long span is reduced (to square dimensions, for example), the stability increases as a result of the increased confinement and rigidity provided by the extra two abutments. A face with a dimension ratio greater than 10:1 can be treated as a (tunnel) span equivalent to the shorter dimension.

Hydraulic radius more accurately accounts for the combined influence of size and shape on excavation stability. It is useful to become familiar with the range of "spans" for a given hydraulic radius. This will provide a means of comparison with other design methods which do not use hydraulic radius. Figure 2.17.6 illustrates these limits for a fixed hydraulic radius of 5 m. Note that although it is possible to apply this method to mining tunnels, the method has been calibrated for open stopes with finite dimensions and with lower priority for safety.

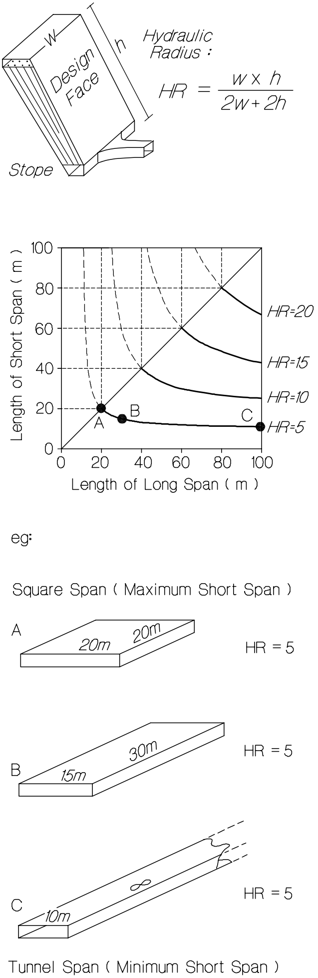


Figure 2.17.6: Hydraulic Radius, *HR*

2.17.3 Open Stope Case History Database

No-Support Limit

176 case histories by Potvin (1988) and 13 by Nickson (1992) of unsupported open stopes are plotted on the Stability Graph shown below. The modified stability number, *N′*, and the hydraulic radius, *HR*, were calculated for each case study as outlined in the previous sections. ***Stable*** stopes exhibited little or no deterioration during their service life. ***Unstable*** stopes exhibited limited wall failure and/or block fallout involving less than 30% of the face area. ***Caved*** stopes suffered unacceptable failure. Potvin plotted a *Transition Zone* defined by these cases to separate the *Stable* zone from the *Caving* zone. The upper boundary of this zone represents a recommended no-support limit for design. For a calculated value of *N′*, determine the maximum hydraulic radius for a stable stope face.

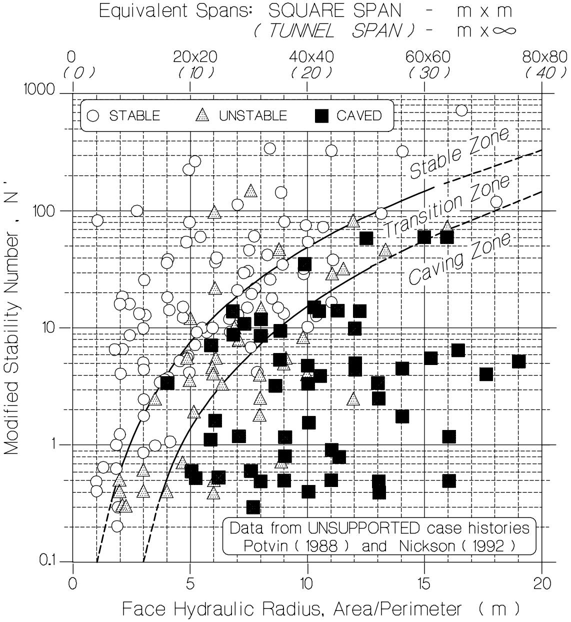


Figure 2.17.7: Database (Potvin, 1988; Nickson, 1992) of unsupported stopes

Limits of Cablebolt Effectiveness

Potvin (1988) and Potvin and Milne (1992) also collected 66 case histories of open stopes in which cablebolt support had been used. Nickson (1992) added an additional 46 case studies to this database which is illustrated below. Cablebolted stopes exhibit improved stability leading to larger stable spans (greater hydraulic radii). While this database does not take into account issues such as quality control, it does provide a reasonable demonstration of cablebolt effectiveness.

Potvin plotted a limit for cablebolt effectiveness which Nickson modified using statistical methods and additional data. The upper curve plotted below represents the limit of *reliable* cablebolt performance. Nickson proposed a zone as shown below to indicate the maximum stable hydraulic radius for cablebolted stopes (upper bounding curve) and the reduction in confidence until cables can no longer be assumed to be providing any degree of useful stope support (lower bound). Below this zone caving is inevitable.

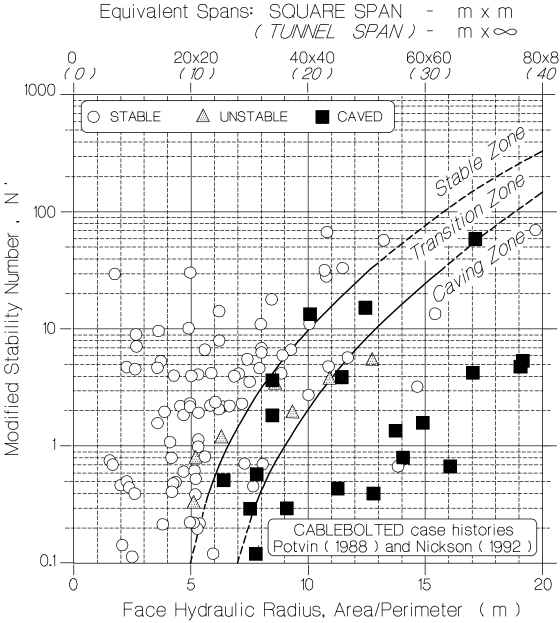


Figure 2.17.8: Database of cablebolt-supported stopes

Stability Graph - Stope and Support Design Zones

The recommended limits for unsupported and supported stopes are combined, along with the respective transition zones to obtain the design chart presented below. This graph allows the engineer to determine, from a calculated value or range of *N′*, the maximum recommended stope size and shape for an unsupported or supported case. A stope which plots well above or to the left of the uppermost design curve is capable of remaining stable without support for a reasonable service time. (Note that non-entry conditions are assumed here and that light patterned rockbolt support and mesh may be required for personnel safety in other areas). A stope which plots well into the lower-right quadrant is likely to suffer major instability with or without support. The cablebolt design zone gives the range in which cablebolts should be needed and effective. Clearly, the actual effectiveness is reduced further right and down within this design zone. As *HR* is increased or if *N′* deteriorates within this zone, the risk of failure is increased, and standup-times are reduced requiring tighter cablebolt patterns and longer bolts.

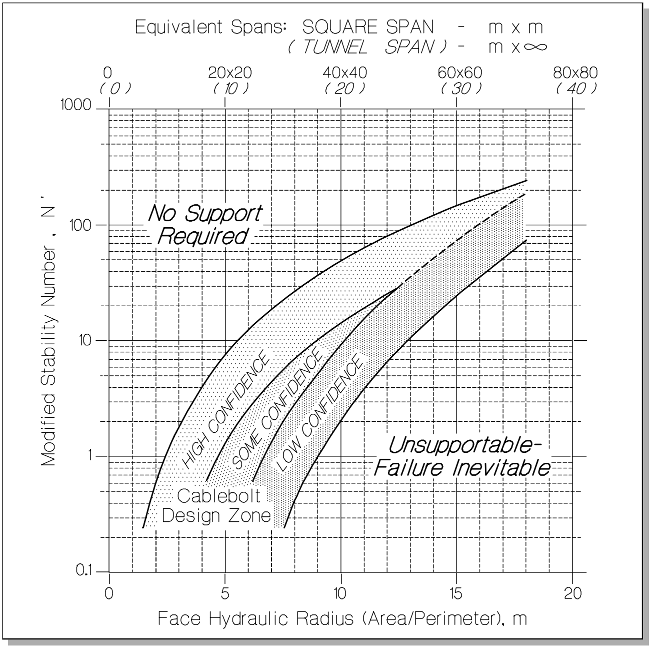


Figure 2.17.9: Design Zones for Open Stopes using Stability Graph Method

Cable Support Recommendations - Potvin (1988)

Based on his original database, Potvin (1988) determined crude guidelines for the design of patterned cablebolting. Specifically he proposed design charts for cablebolt length and cablebolt density. Cablebolt length is the length of the individual cablebolt (minimum length) and cablebolt density represents the number of cablebolts per unit area of stope face.

For the design of ***cablebolt density***, Potvin selected as the key empirical parameter, *(RQD/Jn)/HR*. This represents a measure of relative block size with respect to the excavation size. When this number was small it was expected that an increased cablebolt density would be necessary to ensure stability. The resultant design chart is shown at right. Note the different zones shown here.

Based on this data set, Potvin proposed that cablebolts were ineffective when *(RQD/Jn)/HR* was less than 0.6. In addition, note that the practical minimum cable density is 0.1 corresponding to a square pattern of approximately 3x3m. Three cablebolt density design lines are given which correspond to different degrees of conservatism. Non-entry stopes may require a lower cablebolting density than a main haulage drift for example.

The ***cablebolt length*** used in each case study was plotted against the hydraulic radius. This follows logic based on classical rules of thumb relating bolt length and span. A representative line based on current practice is shown and corresponds approximately to:

$$Length = 1.5 \text{ x } HR$$

up to a practical maximum of 15m at a hydraulic radius, *HR*=10m.

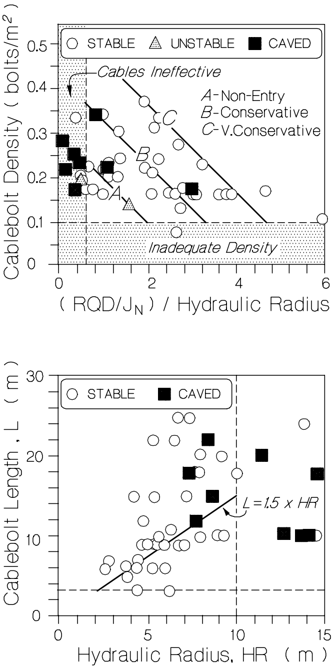


Figure 2.17.10: Guidelines for cablebolt density and length for regular patterns (after Potvin, 1988)

Cablebolt Density (bolts/m^2 of face) - Local Unravelling

Potvin (1988) plotted cablebolt densities used in case histories against *(RQD/Jn)/HR* based on the assumption that relative block size was in principle the governing empirical parameter for stope face stability and support effectiveness. Nickson (1992), however, applied statistical techniques in an investigation of many possible parametric combinations. For the combined cablebolted stope database of Potvin and Nickson, *(RQD/Jn)/HR* actually gave a very poor correlation to cablebolt density based on current practice. This is illustrated by the scatter in Figure 2.17.10. It is proposed here that the absolute block size represented by *RQD/Jn* should control local block fallout from the face and therefore should strongly influence ultimate stability of the stope. If cablebolts are spaced too far apart, unravelling will occur between bolts, progressively leading to more serious instability. The corresponding graph based on the Potvin/Nickson database is shown in Figure 2.17.11. The design zone plotted provides a crude recommended design range for cablebolt density in open stope applications. This design zone should not be applied to permanent openings or in high traffic areas, where safety is a critical issue, unless accompanied by primary support such as rockbolts and screen.

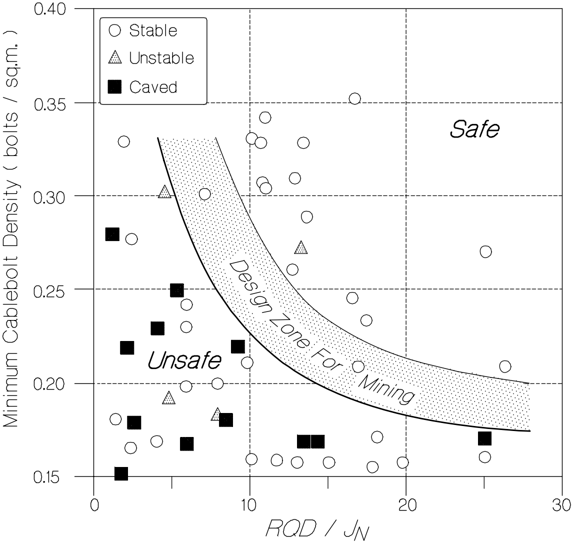


Figure 2.17.11: Guidelines for cablebolt density to control local unravelling

Cablebolt Density or Spacing - Stope Face Support

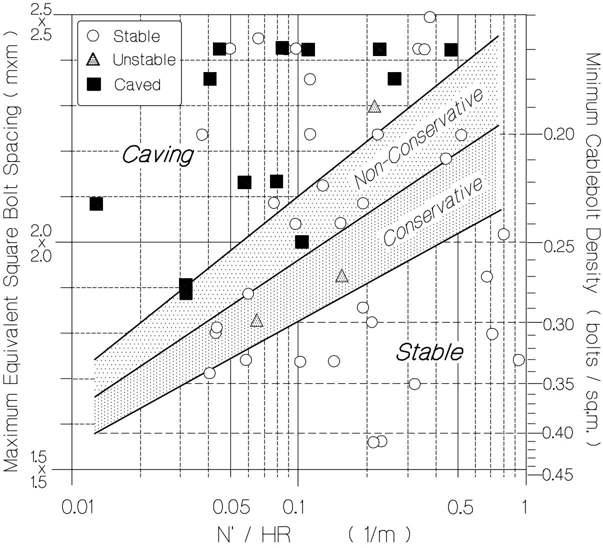


Figure 2.17.12: Guidelines for cable spacing and density - overall stope face stability

Nickson (1992) showed that the best empirical correlation with respect to cablebolt density was obtained by plotting density with respect to the parameter *N'/HR*. The logic here is similar to Potvin's usage of *(RQD/Jn)/HR*, except that *N'* contains additional information about stope inclination, stress related fracturing (parameter *A*) and favourable or unfavourable joint orientations. Nickson derived a relationship based on current practice without considering the degree of support effectiveness. The design zones proposed above in Figure 2.17.12 do relate to this degree of success. While the data scatter is great due to the trial-and-error nature of present design practice, there appears to be a reasonable limit to cablebolt effectiveness as delineated by the cluster of caved cases in the upper portion of this plot. The ***non-conservative*** zone can be used as a guide for non-entry conditions or where dilution is not critical. The ***conservative*** zone is applicable to stope backs above drilling horizons and other areas where entry is permitted. Note the two vertical scales used here. These illustrate the relationship between cablebolt density and the cable spacing of an equivalent square pattern. Use Figures 2.17.11 and 2.17.12 together to determine the critical (maximum recommended) spacing.

2.17.4 Semi-Empirical Cablebolt Design Approach

It is possible to combine the information gained from empirical methods with mechanistic assumptions and logic to develop a more sound semi-empirical design methodology. Figure 2.17.13 illustrates this approach. The ***No-Support Zone*** and the ***Unsupportable Zone*** are derived as previously discussed from examination of over 350 case histories. Within the cablebolt design zone (shaded area), however, it is possible to assume some basic support functions and modify the design accordingly. The ***Reinforce***ment zone implies that the rockmass is still partially stable, requiring cables to merely hold together the constituent blocks to form a self-supporting arch or beam. Cable spacings and lengths along the upper boundary of this zone are derived directly from the analysis in Section 2.18.12 using a back calculated rockmass stiffness. In the ***Support*** zone, however, the cables must bear the full load of the failed or loosened rockmass. Spacings and lengths along the lower boundary of this zone are therefore derived from conservative civil engineering experience (Section 2.16.5). The transition between these two extremes is continuous across the shaded zone. ***Retention*** recommendations based on ravelling failure (Figure 2.17.12) are superimposed on the above results. The maximum spacing and minimum length required to effectively carry out all of the support functions considered are then plotted in the following charts. The zone marked ***Retain*** in Figure 2.17.13 is the zone in which this function is critical with respect to spacing of cablebolt support. In the other zones, reinforcement and support dictate the maximum allowable spacing.

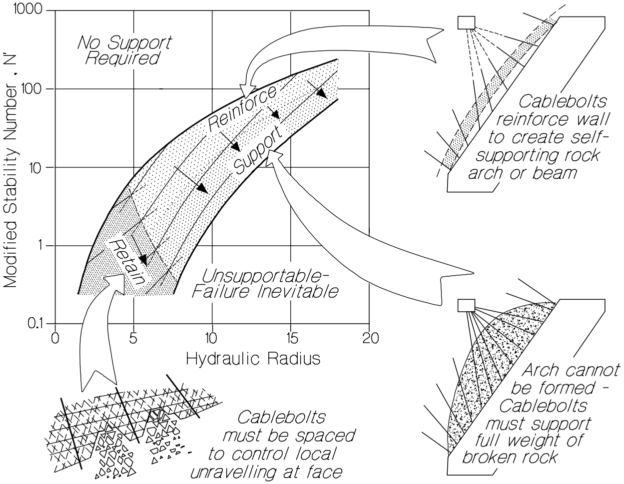


Figure 2.17.13: Five design zones for cable support of open stopes

Maximum Design Spacing for Single Strand Cablebolts

Based on the assumptions illustrated in Figure 2.17.13, recommended cablebolt spacings (for an equivalent square pattern) have been calculated for the range of *reinforcement-support* across the shaded cablebolt support zone. Where the maximum spacing so determined exceeds the recommended spacings obtained from Figure 2.17.12, *unravelling* between and around cables is assumed to dominate stability and Figure 2.17.12 therefore controls the design. The composite result is the cablebolt spacing design chart shown below in Figure 2.17.14.

For a given value of *N'* and *HR* plotting within the shaded cablebolt design zone, it is possible to determine the maximum (critical) spacing of single cables in a square pattern to ensure stability. Note that minimum cablebolt density, *Dc*, is related to maximum equivalent square spacing, *Sc*, as follows:

Cable Density, *Dc* (bolts/m^2): $\mathbf{Dc = Sc^{-2}}$ *Sc* = Cable Spacing (m)

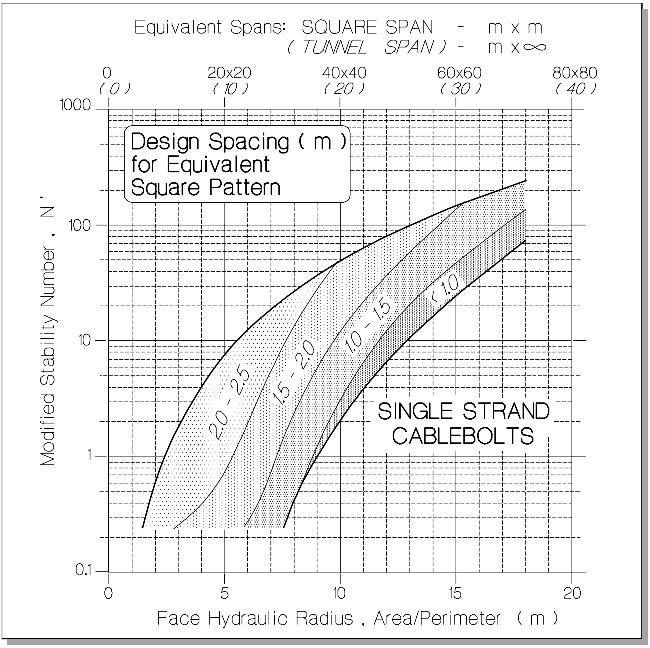


Figure 2.17.14: Recommended spacing for single strand cables (regular pattern)

Maximum Design Spacing for Double Strand Cables

Single cablebolts (15 8mm strand) can be assumed to have 20 - 25 tonnes (200-250 kN) of long term capacity provided that the bond strength and embedment length are adequate. Double strand cables normally possess approximately twice this capacity. Figure 2.17.15 below gives design ranges for double strand cablebolt spacing. Again it is important to emphasize that full load transfer from the rock to the cable is assumed. This implies good quality control and/or the use of modified geometry cables (birdcage, bulbed strand, etc) and/or the use of plates when practical. Note the expanded patterns as compared with single strand cables. Also note that double cables make little difference in the lower-left *retention* zone. Instability in this region is not related to steel capacity but only to interbolt distance. Spacings can be increased as shown (dashed lines) when cables are used in combination with a tight pattern of rebar or rockbolts or shotcrete. These primary support elements serve to retain blocks and knit together a surface layer which can be supported with an expanded pattern of cablebolts.

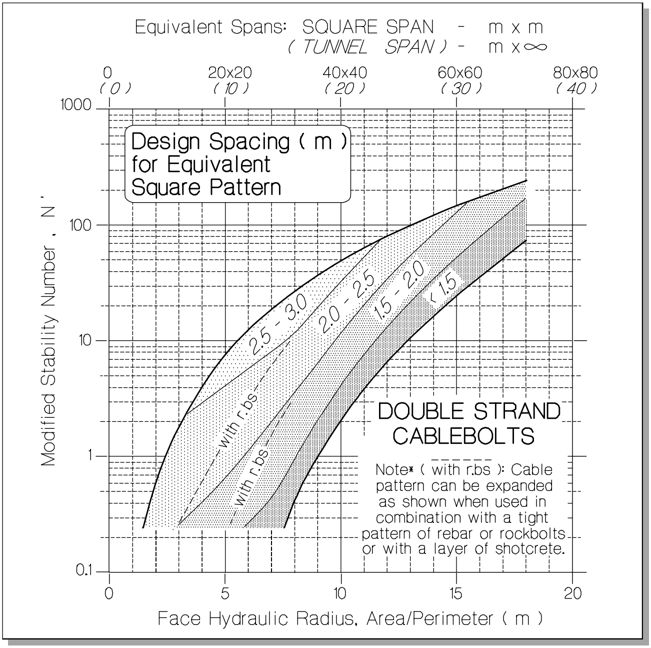


Figure 2.17.15: Recommended spacings for double strand cablebolts

Minimum Design Length for Cablebolts (Single/Double Strand)

Support design at the outer limits of the *Reinforce*ment and the *Support* zones illustrated in Figure 2.17.13 are based on limiting conditions of arch/beam reinforcement and deadload estimation respectively. Based on parametric analysis using conservative parameters derived from N', these analyses yield the bounding values for spacing discussed in the previous sections and for length as shown below in Figure 2.17.16.

Recommended lengths for cement grouted cablebolts differ from resin grouted or mechanical bolt recommendations in the literature. This is due to the necessary addition of a reliable anchor length beyond the zone of supported rock. In the case of beam analysis and deadload estimation, this corresponds in the figure below to 2m beyond the stabilized beam or failed zone respectively. Note that increasing length does not always imply increased capacity (controlled by strand density). These lengths are based primarily on cable coverage of the supported zone.

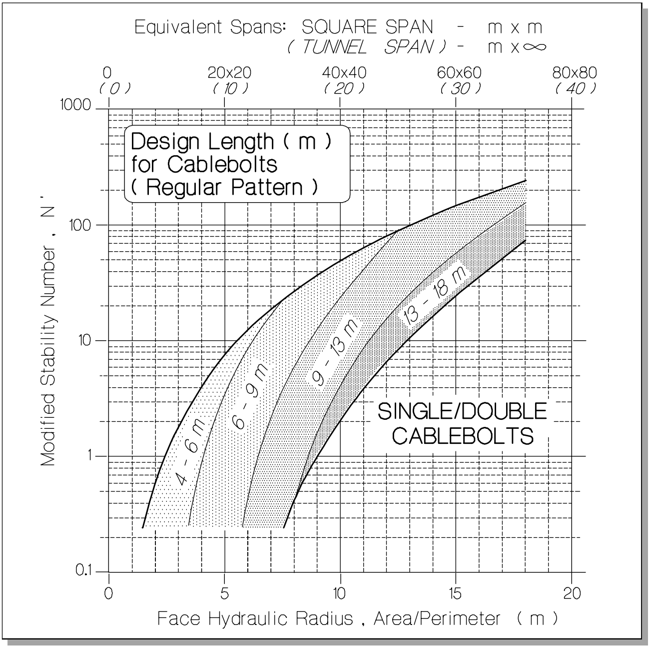


Figure 2.17.16: Recommended minimum lengths for grouted cablebolts

Cablebolt Spacing and Length of Regular, Uniform Arrays

All of the preceding discussion concerning the modified stability graph and recommendations for cablebolt spacing and length apply to a regular or patterned array of cables; a constant distribution of bolts across the face area of the stope and an arrangement behind the face such that neighbouring cables are within 40 degrees of being mutually parallel. The example cablebolt patterns in Figure 2.17.17 illustrate the ideal application of these guidelines.

Cables should be spaced as close to square as possible if designed using the recommendations in this section. For example, patterns such as 1.5m x 2m (equivalent square = 1 7m x 1.7m) or 2m x 2.5m (= 2.2m x 2.2m) are acceptable, whereas a pattern of 1m x 3m may not perform as well as the equivalent square pattern (1.7m x 1.7m).

Cable spacing should also be uniform (i.e. spacing should not vary more than 20% over the stope face). The density of tight clusters of cables bounding larger areas of unsupported stope face cannot be averaged over the whole area and equated to an average density or equivalent spacing. The design of this type of system is handled differently as shown on the following page.

Cable lengths are specified for cablebolts which are within 30-40 degrees of perpendicular to the stope face. Normally the length refers to the perpendicular distance between the face and the end of the cables. Actual cable length will depend on the cable angle.

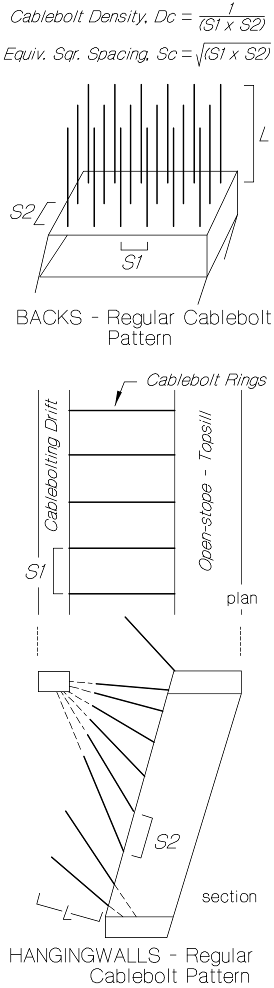


Figure 2.17.17: Regular Patterned Support

Line or Point Anchor Arrays - Sub-Span Design

In many cases in mining, access constraints do not allow the installation of a regular uniform pattern of cablebolts in a back or hangingwall. In addition, mining influences such as blasting, induced stress change or rock relaxation may limit the effectiveness of a distributed cable pattern (Section 2 6). This is particularly the case in foliated hangingwalls. Often it may be preferable, therefore, from both an operational and an engineering viewpoint to install line anchors as shown below at prescribed intervals. These anchors reinforce a local volume of rock, limiting internal displacements and preventing dilation. This artificial rockmass block or rib then acts as an effective abutment for adjacent spans (Fuller, 1983).

This support system should only be used in rockmasses dominated by a single lamination parallel to the stope face or joint sets perpendicular to this face (few oblique joints). Blast control is critical to avoid damage to the unsupported span and displacement rate monitoring may be a useful design verification tool here.

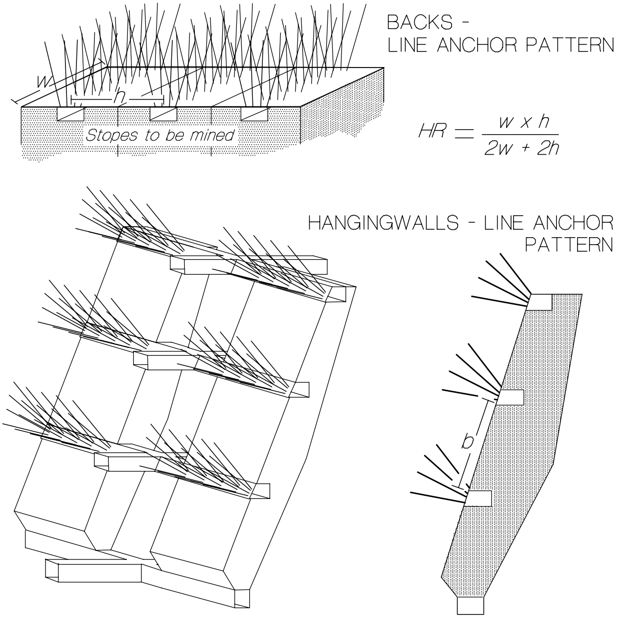


Figure 2.17.18: Line anchor support system geometry

As such these anchors must have a locally dense arrangement (<1.5 m spacing at collar) and 4-6 cablebolts in each ring. These cables should then be plated. This is to ensure limited internal movement within this reinforced "abutment". The Modified Stability Graph can then be used directly to dimension the unsupported sub-spans (a x b in Fig. 2.17.18). These sub-spans (unsupported spans) may be strung together providing a huge operational benefit by allowing a much larger stope to be opened without immediate backfilling. There is a limiting relationship, however, between the unsupported sub-span and the overall "supported" span (or hydraulic radius of total open stope face). Nickson (1992) compared 13 case histories of line anchored hangingwalls and proposed the crude relationship illustrated in Figure 2.17.19.

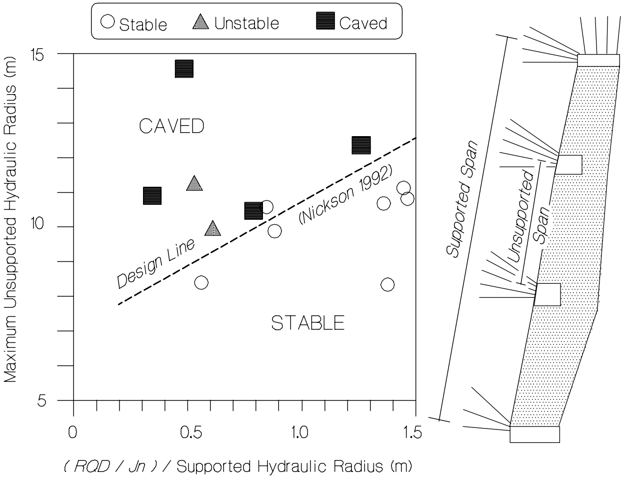


Figure 2.17.19: Crude relationship relating overall (supported) span to unsupported sub-span. Applicable to hangingwalls only (data from Nickson,1992).

Note that the database is extremely limited and so caution must be exercised when using this graph. Calibration to local conditions will be necessary.

The relationship above should not be applied to shallow dipping hangingwalls or backs. This method is designed for non-entry stopes should not be applied to stope faces in areas where regular human access is necessary without additional primary support such as rockbolts and screen to control small block fallout.

2.17.5 Stability Graph - Examples

Consider the following examples of open stope scenarios. These four cases have been deliberately chosen to result in the same hydraulic radii, *HR* and the same values of Modified Stability Number, *N′*.

Table 2.17.2: Four example applications of the Stability Graph

	CASE A Hangingwall	*CASE B Back*	*CASE C Hangingwall*	*CASE D Back*
Problem Description				
Depth	200 m	600 m	150 m	1000 m
Wall Stress	10 MPa	20 MPa	8 MPa	60 MPa
RQD	40	60	85	90
Joint Sets	2	2 + random	3 + random	2 + random
Joint Surface	Smooth Planar; Slightly Altered	Rough Undulating; Unaltered Stained	Rough Planar; Slightly Altered	Slickensided Undulating; Unaltered Stained
Rock Type	Foliated Schist	Bedded Limestone	Gneiss	Massive Sulphide
Rock Strength	80 MPa	115 MPa	160 MPa	180 MPa
Input Parameters				
Wall Dimensions	20 m X 40 m	18 m X 55 m	25 m X 30 m	22 m X 34 m
RQD/Jn	40 / 4 = 10	60 / 6 = 10	85 / 12 = 7.1	90 / 6 = 15
Jr/Ja	0.5	3.0	0.75	1.5
A	0.78	0.52	1.0	0 21
B	0.3	0.3	0.3	1.0
C	8.0	2.0	6.0	2.0
Stability Graph Coordinates				
HR	6.7	6.8	6.8	6.7
N'	9.4	9.4	9.6	9.6
Status	STABLE	CAVED	UNSTABLE	CAVED

These examples are illustrated in Figure 2.17.20 on the following page. Note the obvious differences in stope dimensions and in geometrical and geomechanical environment. Yet the plotted results for the four cases are indistinguishable. This illustrates both a strength and a weakness of the Stability Graph method. Like all empirical methods it is a general design method which allows us to formulate preliminary designs in the face of limitless variety and complexity. Other design techniques are normally very problem specific and cannot be universally applied. Once the preliminary design is established, however, the method does not provide for the fine tuning which must occur to adapt the design to the specific problems encountered in each case.

Case A, for example, is a thinly laminated rockmass with a second discontinuous joint set at 90 degrees from the main lamination. Even though the RQD is low due to the foliation, the stope wall is vertical and as such should be inherently stable unless disturbed by poor blasting or excessive span development. Cables are unlikely to improve stability within economic limits in this case. The stress is low compared to the rock strength so gravity is likely to be a dominant control. This case is suitable to Voussoir beam analysis (Section 2.18.12).

Case B is a competent blocky rockmass above a relatively wide sill span. The main lamination would suggest beam analysis. The cross jointing, however, is oblique to the back and is unlikely to allow complete arch development in the horizontal back. Patterned roof bolting will be necessary in this case

Case C represents a strong gneiss with moderate structural density. Even though the block size is small, the joint surfaces are very rough and tightly interlocked. The stresses are low but the steep dip of the wall will maintain compression and improve stability. Patterned cablebolting from a hangingwall drift should prove effective in this case.

Case D appears to be the highest quality rockmass as indicated by the large values for *RQD/Jn* and *Jr/Ja*. The stability graph analysis does not consider the sheared contact which forms the hangingwall. It is likely that this contact will shear due to stresses in over the back. These stresses are high and this slippage may be unstoppable. The vertical jointing will form release planes resulting in a large free full span wedge which must be supported. Cables must be designed to withstand large displacements or they will snap as the wedge slips.

These examples show that while the Stability Graph method is an invaluable tool for initial dimensioning and support design for open stopes, it is not the final word. If the stope plots well into the stable zone or well into the caving zone, then the respective result is fairly reliable. If the stope plots close to or within the cablebolt design (support required) zone, then further mechanistic analysis should be carried out to confirm the validity of critical assumptions and recommendations of the stability graph method.

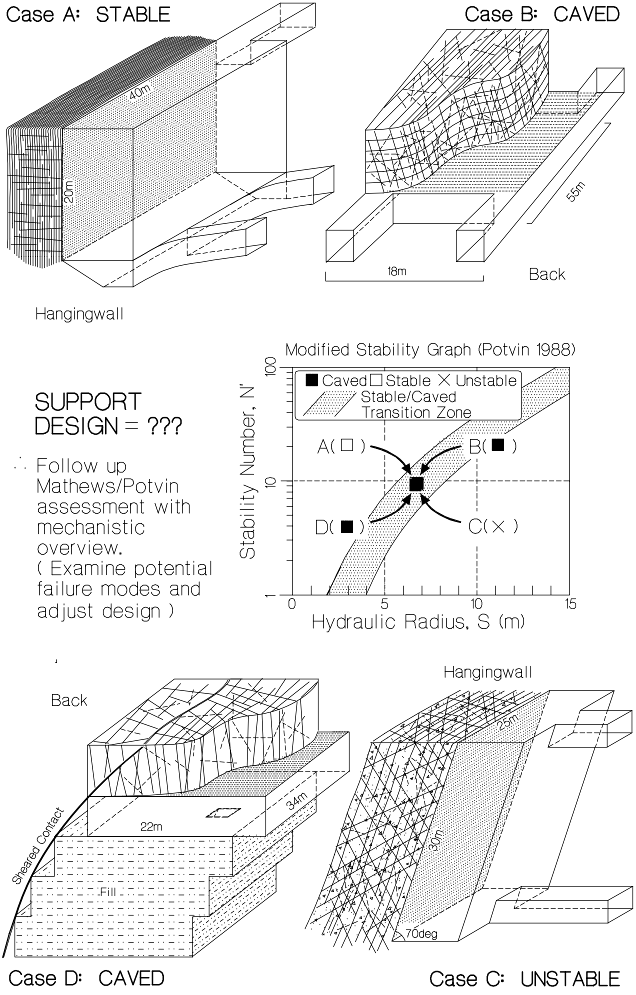


Figure 2.17.20: Four application examples for the Stability Graph Method. Note that very different design problems can result in the same position on the graph.

2.17.6 Stability Graph Method - Limitations

There are certain assumptions inherent in the application of the Stability Graph Method. Observe the following limitations when using the method for stope and support design.

Inadequate Fill

The estimation of stable hydraulic radii determined from the graph or used as input for stability evaluation assumes that the span being considered is fully bounded. This assumption is valid for unfilled stopes which are surrounded by fill (as in alternate block sequencing, for example). The surrounding fill must be tight to the walls and back of the stopes in order to be considered supporting elements. If this is not the case as shown in Fig. 2.17.21a), the true effective span for analysis may be much larger than the nominal stope panel. The same is true if the fill is highly compressible. In such a case, the Stability Graph Method is not applicable to the design of the unmined panel 7 in Fig 2.17.21b), for example.

Corners-Designed and Accidental

As shown in Fig 2.17.21c), corners or bulges can be created in stope walls though poor design ("chasing grade") or though the upward caving of mined stopes below. In either case, the stability graph cannot be used to evaluate the stability of either the span above the corner nor the overall span. The corner so created, will dominate the stability of the entire stope and will likely cause major stability problems. Such corners, either deliberate or accidental should be avoided.

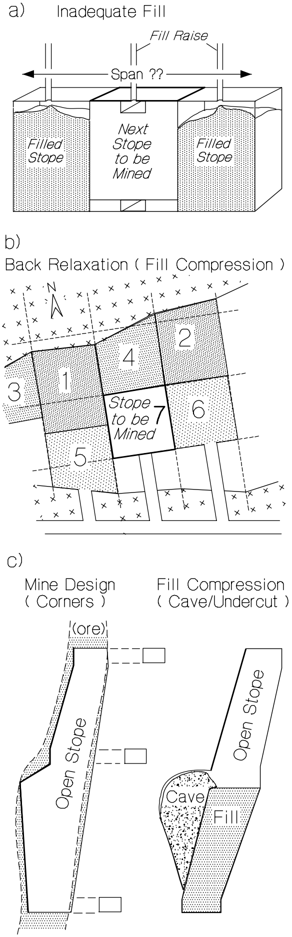


Figure 2.17.21: Limitations for use of Modified Stability Graph

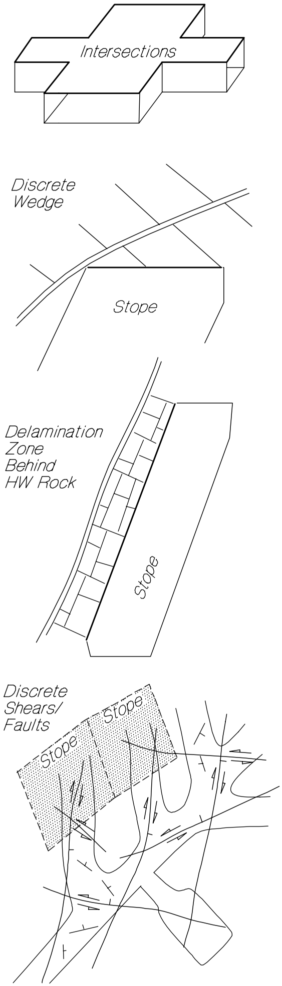


Figure 2.17.22: Limitations of the Stability Graph Method

Intersections

While the Stability Graph Method was calibrated for open stopes, it can be used for large mining tunnels and sills provided that a conservative approach is taken for safety reasons. The method should not be used, however, to design intersections. The assumption of a bounded span is not valid here. Intersections are normally less stable than the associated tunnels. In addition, it is not possible to calculate an equivalent *HR* for an intersection.

Discrete Wedges

The stability graph design approach is applicable to moderately structured rockmasses with distributed or ubiquitous structure. Discrete structural features such as large wedges which may form in sill backs must be considered separately.

Delamination Zones

Large stable spans may be predicted in cases with structurally sound wall rock. If this wall rock is bounded by a weak layer close (within 20% of the span) to and parallel to the wall as shown at left, the stability of the resultant beam will be reduced. Beam analysis methods may be more appropriate for design.

Discrete Shear Structures

Large scale structure (length > stope dimensions) will control stope stability under stress and gravity. Discontinuum analysis methods must be used for design. The Stability Graph results will not accurately predict stability.

2.17.7 Stability Graph-Calibration to Local Conditions

The initial database of approximately 350 case histories is impressive in size and scope. It is, however, incapable of accurately predicting stability in every possible situation. The database, for example, reflects Canadian practice. This immediately implies a bias towards Canadian conditions. In short, the method provides an excellent starting point for design but it must be calibrated on-site in every new mining environment. This involves maintaining an up-to-date database of stope dimensions, rockmass parameters, and stability status.

Bawden (1993) uses a data set from Greer (1989) to demonstrate this concept as illustrated in Figure 2.17.23 below (note the truncated axes for more detail). In this case, due to unique conditions at the mine site, significant caving and instability was observed in stopes which the method predicted to be stable.

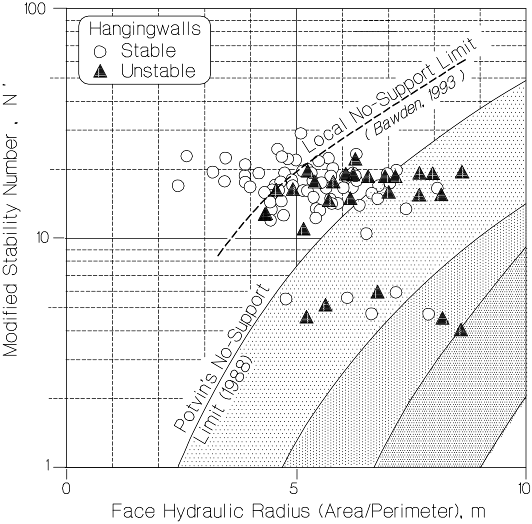


Figure 2.17.23: Example of local site calibration. New design line (dashed) can be used for future stope design (after Bawden, 1993).

If such a local database is maintained, then the Stability Graph can be calibrated for local conditions. The dashed design line proposed by Bawden for the above data bounds the caved and unstable stopes. This line should now be used as the no-support limit for future mine design at this site.

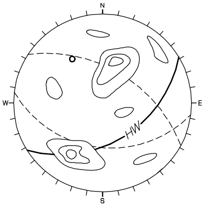
2.17.8 Parametric Analysis

The quality of a rockmass is never well defined. For overall mine design and budgeting purposes at a preliminary stage it may be adequate to design based on average rockmass conditions. Assuming worst case parameters may prove impractical from an economic perspective while designing based on the best possible conditions would clearly be imprudent.

In order to understand the consequences of this variability at a given site, it is useful to employ a bounded analysis for the Stability Graph method by tabulating reasonable ranges for the input parameters (limit ranges to one tabulated category or one standard deviation for each parameter and only use variability as required or impractically large solution ranges will result) and then calculating an expected range for N'. Consider the following example input for the hangingwall of a mine employing a modified AVOCA mining method:.

Table 2.17.3: Data sheet for *parametric* design example

DEPTH	500m		
STOPE HEIGHT	20m		
NOMINAL PANEL WIDTH	30m		
WALL STRESS	20 - 30 MPa		
HW DIP	65 degrees		
HW ROCK	gneiss		
UCS	120 - 180 MPa		
JOINTS	2 + random	rough & planar	stained - slightly altered



PARAMETER	***LOWER BOUND***	***EXPECTED***	***UPPER BOUND***
RQD	70	75	80
Jn	6	6	6
Jr	1.5	1.5	1.5
Ja	2	2	1
A	0.3	0.4	0.5
B	0.2	0.3	0.4
C	5.5	5.5	5.5
N'	**2.9**	**6.2**	**22**

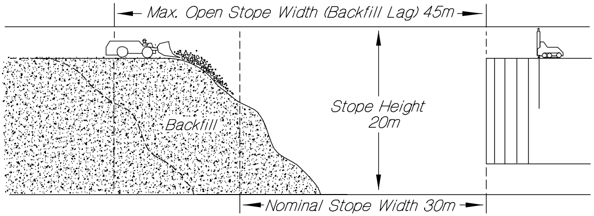


Figure 2.17.24: AVOCA Stope example - Fill Lag can be up to 1/2 stope width

Due to operational delays and scheduling problems, it is expected that the backfill front as illustrated in Figure 2.17.24 can lag behind the design position (relative to the blasting face) by as much as 1/2 of the nominal panel width. It must be considered then that the width can vary from 30m to 45m. The hydraulic radius, HR, must therefore be assumed to vary in the range:

From: $\frac{20 \times 30}{40 + 60} = 6$ To: $\frac{20 \times 45}{40 + 90} = 7$

The combined range of N' and of HR can be plotted on the Stability Graph as shown in Figure 2.17.25. Support is clearly required in this case. When worst case conditions occur, significant stability problems may result if support is inadequate. The decision to enhance support beyond average requirements must be based on risk to personnel and equipment and on the costs, losses and delays associated with unexpected dilution. This method can be expanded to involve probabilistic methods similar to those outlined in Hoek et al. (1995) and Harr (1987).

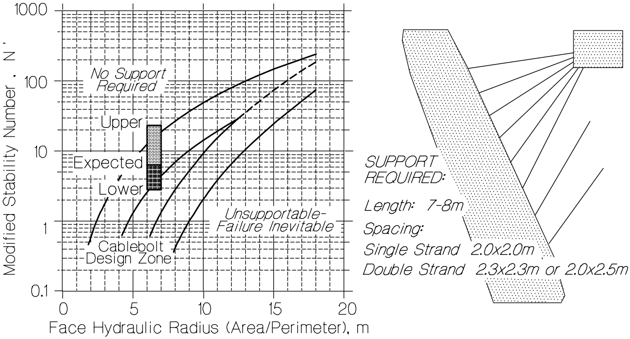


Figure 2.17.25: Example design range and recommendations

2.17.9 Probabilistic Analysis

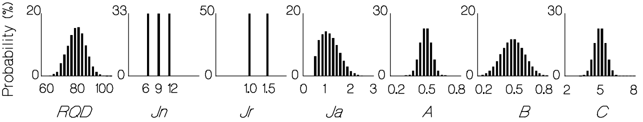


Figure 2.17.26: Example input distributions resulting from in situ variability

Another approach to incorporating input variability into the analysis is through the use of a probabilistic approach. Input parameters can be assigned distributions as shown in Figure 2.17.26. If a single recorded value for any parameter is sampled at random from the database, these histogram distributions show the relative likelihood of the sample equating to a particular value or range. Distributions can be obtained from real field data using statistical techniques (Harr, 1987; Pine, 1992; Rosenbleuth, 1981) or commercial simulation software (Hoek et al , 1995; Carter, 1992; Diederichs and Kaiser, 1996), and can be used in a Monte Carlo style analysis. In this analysis, a large number of calculations for *N*' are generated from different combinations of values for the above parameters. The frequencies with which each parameter falls within different ranges for use in the calculation are reflected in the distributions in Figure 2.17.26. Several hundred such calculations result in the distribution for *N'* shown in Figure 2.17.27 a). If this distribution is superimposed on the instability limits at a given *HR*, the *probability of instability* or of *caving* (Figure 2.17.27.b) is equal to relative area of the distribution which falls below the respective limit.

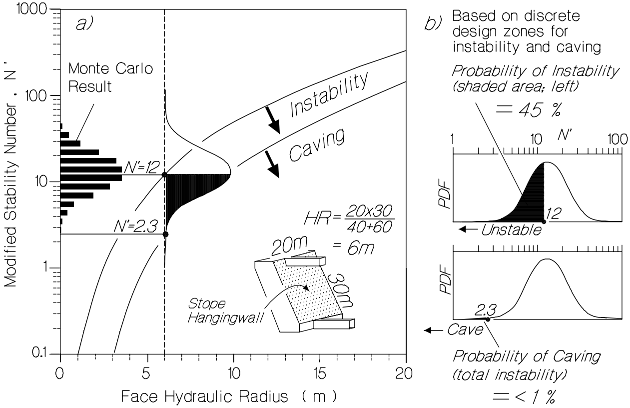


Figure 2.17.27: Probabilities of (b) instability and caving based on (a) Monte Carlo analysis (after Diederichs and Kaiser, 1996)

2.17.10 Dilution and the Stability Graph

The instability and caving limits in the Modified Stability Graph are based loosely on the apparent area of instability across the stope face. If the volume of failure is considered and divided by the volume of the ore in the stope, a value for dilution is obtained (Section 1.2). For a simple rectangular geometry, and if the stope thickness does not change, it is possible to plot contours of expected average dilution on the Stability Graph (Figure 2.17.28). Note that these contours are likely to be site-specific and depend on the stope thickness (5m in the example below). Based on local site experience, a *dilution vs HR* relationship for any rock quality *N′* can be obtained and used in economic analyses to optimize stope dimensions (Elbrond, 1994; Planeta et al., 1990; Diederichs and Kaiser, 1996).

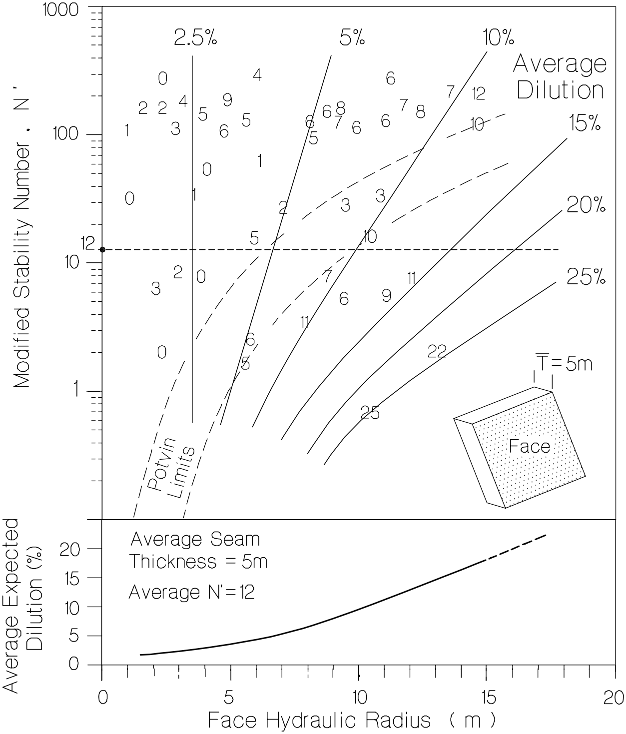


Figure 2.17.28: Site-specific average expected dilution (data from Pakalnis et al., 1995)

2.18 A Mechanistic Toolbox: Customizing the Design

While empirical design methods typically produce general preliminary recommendations to cover a wide variety of rockmass behaviour (within a given rock quality range), a mechanistic approach considers specific failure mechanisms and adjusts design accordingly. Hoek and Brown (1980), Hoek et al. (1995), Brady and Brown (1993), and others give detailed treatment to many of these mechanisms and to the appropriate support strategies. A modest selection is covered here.

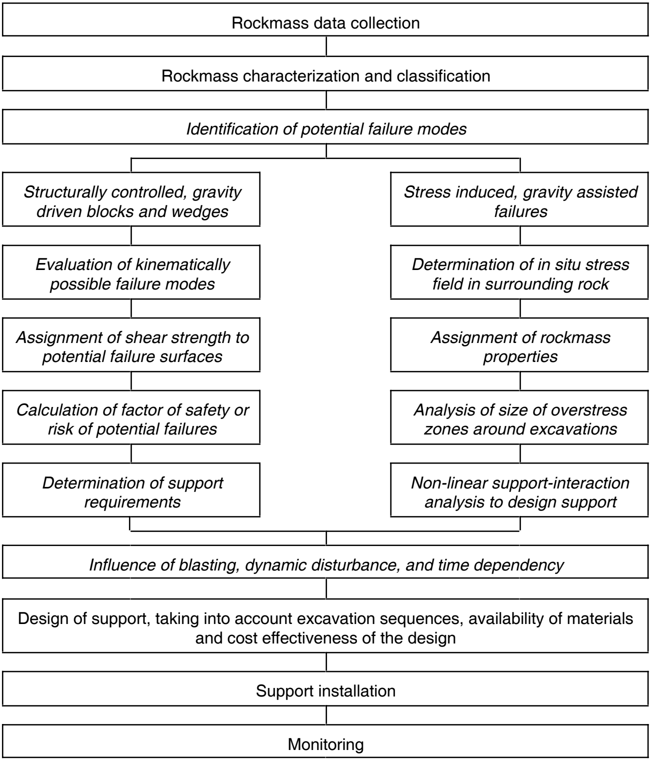


Figure 2.18.1: Mechanistic Design (*Italics*). After Hoek et al. (1995)

2.18.1 Stress Induced Boundary Crushing

Hoek et al. (1995) describe a methodology for analysis of excavation induced stresses for the purpose of support design. Two-dimensional or three-dimensional elastic analysis may be used to evaluate the induced stresses around complex excavation shapes. These stresses can be compared to an appropriate strength criteria to determine the extent of rockmass failure (Sections 2.13 and 2.15).

In hard and brittle rockmasses, the maximum compressive stress can be contoured around an excavation boundary (where the minor or least compressive stress approaches zero) and compared with the uniaxial compressive strength of the rock. Where the calculated compressive stress exceeds one-half (Section 2.13.3) of the strength determined from testing of laboratory samples, it can be assumed that, in the long term, the rockmass will become significantly damaged and may require support. This method of analysis is most appropriate in highly stressed, hard, brittle rockmasses with moderate to low initial fracturing or structure. Failed rock in these environments is unlikely to possess much residual strength and will require full support after the creation of the failed zone. For regions within one excavation radius, compare the induced stress difference (σ_1 - σ_3) to 0.5 times the laboratory *UCS*. Areas where the stress difference exceeds this value may be prone to damage and eventual weakening and rupture.

In softer and more fractured rockmasses, the use of a confinement dependent criteria such as Hoek-Brown (Hoek et al , 1995) is warranted. Plastic analysis may be used to assess the potential for progressive failure and to investigate the role of stress redistribution and self-stabilization. Once a zone of potential failure has been established from such models, it may prudent to simply design cablebolt support to sustain the deadload of this failed zone (Factor of Safety or Strength Factor <1). Often this will prove economically impractical and may not be necessary, since failed rock often retains limited ability to support itself. If plastic analysis has been employed, the support must extend into the zone of confinement or the zone above the back (e g. in a roof support problem) where the minor (least compressive) stress begins to increase. This indicates the development of self-stability. The location of this boundary will, however, be highly dependent on the strength and deformation parameters used in the analysis.

It should be noted in any case that cablebolts are unlikely to arrest the onset of rock failure under high stress and may do little to alter the progression of such failure into the rockmass. The objective here is to hold the failed material in place so that the broken rock itself can generate the necessary confinement to reduce the extent of progressive damage and instability. In highly plastic (deformable) rockmasses under high stress, it is also unlikely that cables will be effective in arresting the progression of failure. In addition, in these environments, the induced displacements may be too great for the system to handle and cable strand rupture may be inevitable in pre-installed systems.

2.18.2 Stress Shadowing and Relaxation

Low stresses can pose as much risk as high stresses in a fractured or jointed rockmass. When confinement is present, joint surfaces remain mated and even limited surface roughness provides adequate dilational and frictional resistance to slip. When joints are subjected to shearing in a confined environment, the asperities on the surface interact and interfere with each other as slip progresses. This results in a tendency for the joint to open which in turn generates increased pressure on the asperities, increasing the frictional resistance to further slip. The only avenue for further slip becomes shear through of the asperities requiring large stresses. In this way, joint slip under confining stress may be self-limiting.

When confinement is removed as in a late stage stope boundary after significant stress and blast damage has occurred and after neighbouring excavations have provided a stress shadow, the fractures are free to dilate and slip resulting in a destabilized rockmass. While even modest stresses across a back or sidewall can serve to clamp the rockmass blocks in place, gravity will dominate once these stresses decrease.

In an elastic model, zones of "tension" or zones of near-zero stress in, for example, a stope wall indicate potential problem areas (refer to Figure 2.13.10). This applies to highly fractured rockmasses, blocky rockmasses and also to areas where discrete, large scale structure is present. Large wedges and blocks can be liberated in otherwise stable areas by reductions in field stress.

At particular risk are zones of rock which have previously undergone overstress failure (confined) and which are, at a later stage in mine life, undergoing relaxation. The previously self-stable rockmass may tend to unravel and cave in this situation. Cablebolt support in these areas can be designed to support the deadload of such destressed zones. Roof bolts in cut and fill stopes, for example, must be designed in this way. Where the ore is weak, ensure that the cables penetrate into the hangingwall (Cassidy, 1980). If this is not possible, then approaches such as beam building (Voussoir) can be employed to optimize design. In any case, it is usually prudent to design cable lengths to penetrate into a zone of confinement.

Zones of relaxation pose an additional hazard for cablebolting. As discussed in Section 2.6, stress decreases across a cable array can seriously impair the bond strength of plain strand cablebolts. Rockmass stiffness is also dependent on confinement in fractured rockmasses, and decreases with relaxation. This has a compounded detrimental effect on cable capacity - just when bond strength is needed the most.

It is for this reason that plating and the use of modified strand cablebolts are recommended in fractured-destressed rock.

2.18.3 Limiting Displacement - Reinforcement

Undisturbed, fractured rockmasses in underground environments are generally inherently stable, provided that excavation can proceed with little or no additional disturbance and if internal displacements (block or particle shifting) can be minimized. Rockmass quality, stiffness and strength generally degrade with displacement.

Consider the example illustrated in Figure 2.18.2. As the hangingwall displaces (at midspan) the constituent blocks shift with respect to each other and lose the essential interlocking required for stability. Rockmass joint surfaces typically have a characteristic roughness scale (the basis for Jr/Ja in the Q system for example) or asperity height. This determines the degree of interblock slip which can be tolerated before significant interlock is lost. Rockmasses will rapidly disintegrate at this time as shown in Figure 2.18.2. This behaviour suggests a limiting displacement which can form the basis for cablebolt pattern and element type selection. In general, rockmasses with small block sizes or low roughness fracture surfaces will have smaller critical displacements than large block size, rough jointed rockmasses.

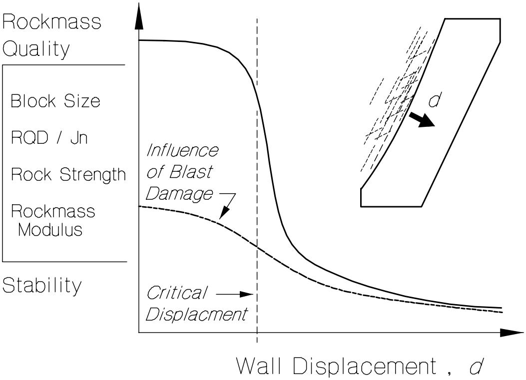


Figure 2.18.2: Effect of displacement on rockmass integrity

When the limiting displacement is small, such as in the case of a highly schistose hangingwall, a very stiff system such as multiple bulbed strand on tight spacings may be required.

This critical displacement can be determined by systematic monitoring from stope to stope (Chapter 4). Design can be optimized in this way.

Displacements in Hard Rock

The yield or damage strength and rupture strength of hard rock can be several orders of magnitude higher than the shear strength contribution of a distributed pattern of cablebolts. It is unlikely therefore that cablebolts can prevent the onset of fracturing and damage in hard rockmasses. In practice, however, cablebolted excavations in overstressed rockmasses normally perform better than if left unsupported. It is well established in rock mechanics, that ductile rockmasses (Figure 2.13.11.b) are more self-supporting after yield than brittle rockmasses of equivalent peak strength. The role of cablebolts in a hard and brittle rockmass must therefore be to restrain the dilation of existing discontinuities and of stress induced fractures so that the post yield strength of the rockmass is maximized and ductility is achieved. Open or dilating joints or fractures can drastically reduce the ultimate strength of a rockmass.

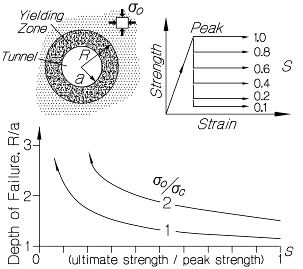


Figure 2.18.3: Depth of failure for rockmasses with varying ductility

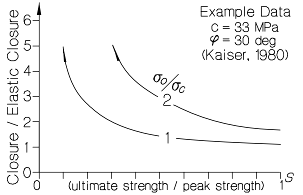


Figure 2.18.4: Relative surface displacement vs rockmass ductility

Analyses by Kaiser (1980), show the extent of the failure zone (Figure 2.18.3) and the surface displacements (Figure 2.18.4) around a circular opening for different degrees of rockmass ductility (s = ultimate strength/peak strength; s=0 for perfectly brittle and s=1 for perfectly ductile) for an example rockmass. Limiting displacement through support creates a more ductile rockmass which carries load after yield, and maintains confinement away from the opening, limiting the extent of failure. This, in turn, further limits the total displacement.

Displacements in Soft Rock

In a soft and weak rockmass, cables perform in a similar fashion as described above. A yielding rock such as phyllite or schist can form either discrete or distributed slip surfaces throughout the yielding zone. These opposing surfaces must dilate (open or separate slightly) in order to shear past each other. Suppressing this dilation increases the shear strength. Single discrete and persistent surfaces require less total dilation to slip and therefore shear more readily than a combination of many small distributed surfaces. Cablebolts act to restrain discrete weakness planes in the soft material. Yielding can then only occur evenly and continuously throughout the rockmass. Even a weak and yielding continuum is stronger than a discretely discontinuous rock of similar material properties.

2.18.4 Stress Induced Joint Slip

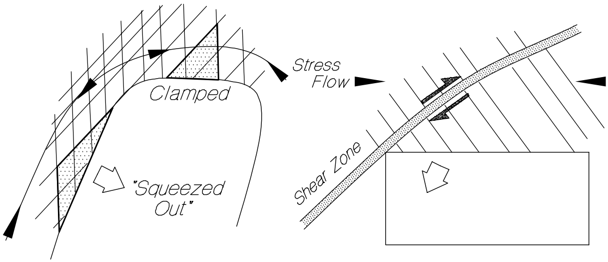
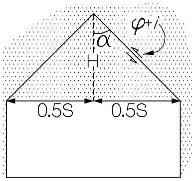


Figure 2.18.5: Stress induced slip along smooth discontinuities

Where slickensided planar joints intersect an excavation boundary at a low angle of between 5 and 45 degrees, there is potential for stress driven displacement or slip across these surfaces. The driving force behind these displacements will normally be beyond the capacities of conventional support in hard rock at depth. These movements are normally shortlived, however, as the stresses are relieved by the slip and transferred out into the rockmass. The goal of pre-support, in this case is to "go along for the ride" during initial slip, maintaining load capacity to support the relaxed blocks created by this deformation. In a highly stressed environment with several unfavourable shears converging at an excavation boundary, a required displacement capacity of 100mm would not be extraordinary.

Once induced stresses around the excavation are obtained using elastic analysis, the procedure summarized in Figure 2.13.7 in Section 2.13.2 can be used to determine the tendency for inclined discontinuities to slip or to be clamped by the induced stress field.



This analysis can be used in conjunction with simple gravity wedge analyses (Sections 2.18 8 and 2.18.9). Wedge analyses can often reveal enormous and steep sided wedges in the back of an excavation. This can be cause for alarm. It can be shown, however, that even with a modest degree of lateral confinement (> 1 Mpa), a rock wedge with apex angle, α, less than the effective friction angle (Section 2.13.3) will be stable (i.e. $\alpha > \phi+i$). This explains why, at depth and in hard rock with rough joint surfaces (i.e. $\phi+i > 45°$), failures involving wedges with heights, *H*, greater than 1/2 of the excavation span, *S*, are rare (i.e. tan(45)=1; $\therefore H_x$=0.5S).

2.18.5 Dynamic Loading

Support displacement capacity is critical in areas of seismically induced or blast generated dynamic loading (Ortlepp, 1983; Kaiser et al., 1995; Kaiser and McCreath, 1992). Cablebolts can be effective where fractured ground has relaxed and become vulnerable to vibration. Support loads generated over larger displacements absorb more of the kinetic energy of such a ground mass. Stiff supports will rupture before the mass can be decelerated and stabilized.

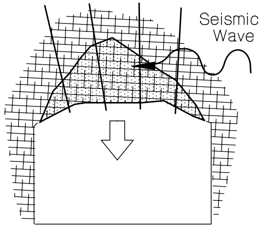


Figure 2.18.6: Dynamic Loading

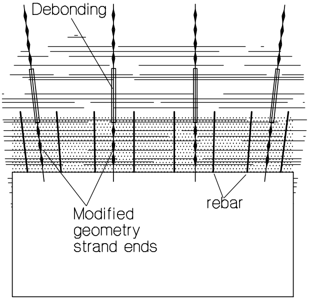


Figure 2.18.7: Composite cablebolt - rebar support (Plain strand, plated cablebolts can also be used in less severe conditions.)

It may be advisable to combine long, partially debonded cables, with modified geometry or plated end lengths, with closely spaced primary support such as grouted rebar (Kaiser et al., 1995). Under dynamic loading the rebar will maintain a reinforced skin at the excavation surface. The large displacement of this skin can be accommodated by the cables, maintaining ultimate holding capacity after the disturbance has passed. Without the rebar, the surface skin may loosen and disintegrate, rendering the cablebolts ineffective.

Tannant and Kaiser (1995) describe an innovative support element for support of blocky ground under dynamic loading. A cable is anchored in a borehole using a Swellex friction bolt (Stillborg, 1986). A length of cable extends beyond the downhole extent of the Swellex and has an optional button or swage (Section 2.9 3) at the end. Even if the Swellex breaks in several places under loading, the cable remains frictionally anchored within the Swellex segments. The load-displacement characteristics are ideal for dynamic conditions; a consistent yield load and a large displacement capacity before rupture.

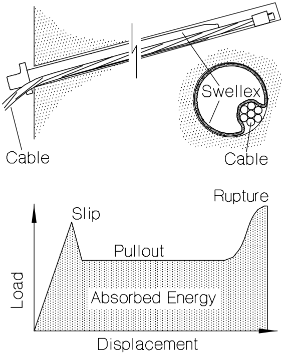


Figure 2.18.8: Cable-Swellex bolt

2.18.6 Surface Unravelling

The spacings for cablebolt elements as calculated in designs based on deadload and support pressure are greater than those calculated for rock bolts, due to the higher capacity of the cables. In addition, cablebolts are not tensioned (although proper plating provides up to 5 tonnes of loading on the surface of the rock - comparable to a rockbolt). The increased spacing between pattern cables and the lack of active loading (unplated cables) provides a greater opportunity for blocks of rock between the cablebolts to fall freely from the surface. In highly fractured rock, supplementary support and face restraint systems will have to be used in conjunction with cablebolts.

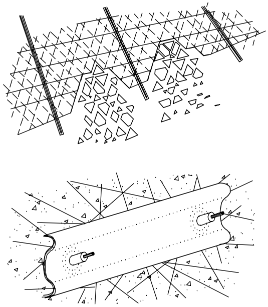
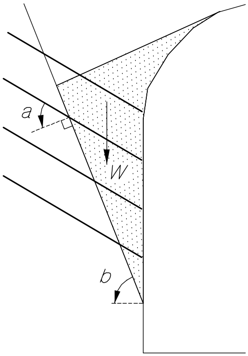


Figure 2.18.9: Surface retention

2.18.7 Sliding Wedge



$$N = \frac{W(f\ \sin(b) - \cos(b)\tan(\phi) - cA)}{B(\cos(a)\tan(\phi) + f\ \sin(a))}$$

f = 1.5 for grouted bolts
2.0 for non-grouted bolts

Definitions:

N = number of cablebolts
W = weight of the wedge
f = factor of safety
B = cablebolt capacity
b = dip of sliding surface
c = cohesive strength of sliding surface
A = base area of sliding surface
a = angle between the plunge of the bolt and the normal to the sliding surface
ϕ = joint surface angle of friction

Figure 2.18.10: Bolt support for a sliding wedge (after Choquet and Hadjigeorgiou,1993)

2.18.8 Two-Dimensional Wedge

In many inclined tabular ore bodies, it is common to find two dominant joint sets; one parallel to the hangingwall (or footwall) and one cutting obliquely across the back. Vertical release planes normal to the ore zone complete the definition of two-dimensional prismatic wedges. These wedges can and often do form across the full span of the stope and must be supported. Consider the bolt spacing and total capacity calculations in Figure 2.18.11.

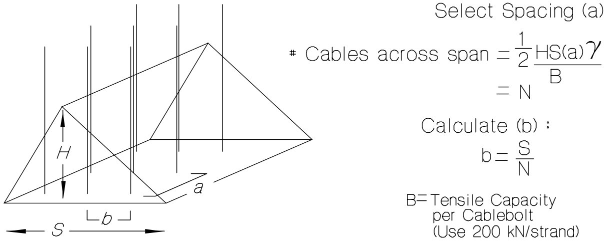


Figure 2.18.11: Bolt spacing and total load for a prismatic gravity wedge

When unplated cables are to be used for this application, it is important to be aware of stress change effects inside the wedge which may lead to bond capacity loss (Section 2.6). In addition, all of the cables within a regular pattern will not have the same load response curve when supporting a gravity wedge. Cables on the outer edges may not have embedment lengths greater than their c*ritical embedment length* (length required to break steel during pullout). These cables will have a much softer response and will not take on load at the same rate as the other cables towards the centre of the wedge. As a result, the centre cables may become overloaded and fail in tension even though the total tensile capacity of all of the cables may be more than the weight of the wedge. It may be necessary to move the outer cables closer towards the centre (use rockbolts at the wedge perimeter), to plate the cables or to use modified strand.

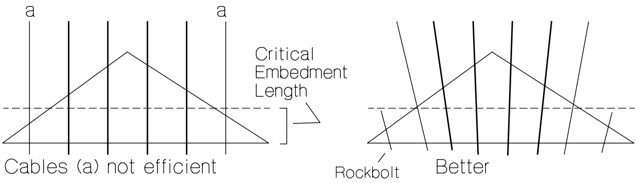


Figure 2.18.12: Critical embedment length adjustment

2.18.9 Three-Dimensional Wedge

The analysis of three-dimensional wedges is based on limit equilibrium analysis methods similar to the two-dimensional wedge analysis discussed in the previous section. The definition of arbitrary wedges, based on joint orientation information, and the calculation of weights and safety factors is more complex than in two-dimensions and therefore routine treatment by hand, while possible (Hoek and Brown, 1980), is not normally practical. Computer software is available to perform the necessary stability calculations.

Three-dimensional wedges are easily visualized using the program UNWEDGE (Hoek et al., 1995). The program allows the calculation of the weight of the wedge, the input of support elements and calculation of the factor of safety against failure (support capacity / gravity demand) using limit equilibrium techniques. The output of the program is similar to that shown in Figure 2.18.13.

The same critical embedment length criteria for non-plated cablebolt support of two-dimensional wedges also apply to the design of support for three-dimensional wedges. A denser pattern in the centre of the wedge may be required.

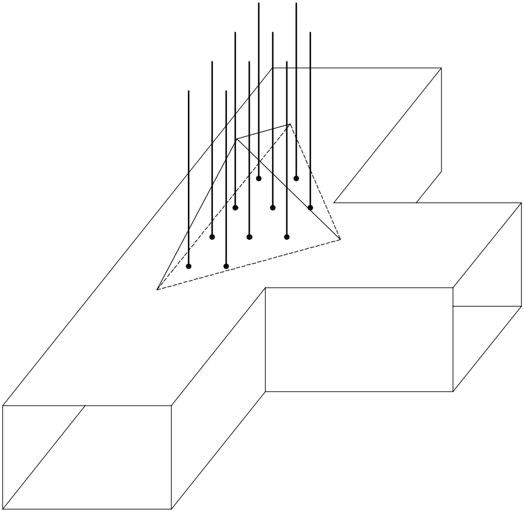


Figure 2.18.13: Visualization of excavation and three-dimensional wedge with cablebolt pattern support

2.18.10 Stress Induced Buckling: Euler approach

In highly anisotropic (foliated) ground at high stress the foliation produces thin slabs of rock which may be parallel to the excavation wall. These slabs can potentially fail at a stress level much lower than the compressive strength of the rock due to the instability phenomenon of buckling, in much the same way as a thin sheet of strong steel will bow and collapse under a minimal load parallel to the sheet. The calculation of a critical stress required for buckling is based on slab geometry and rock stiffness and with the following assumptions:

- The out-of-plane dimension of the slab is the largest dimension.
- The thickness of the slab can be defined as the minimum spacing of potential foliation parting of laminar joints.
- The foliation slabs are sufficiently intact to warrant the use of the intact modulus and compressive strength for stability calculations.
- There is no inclined cross structure which could lead to alternate failure modes.

E = Intact rock stiffness (parallel to foliation)
S = In-plane slab span (long dimension)
T = In-plane slab thickness (short dimension)

$$\sigma_b = \frac{\pi^2 E}{12(S/T)^2}$$

The role of stiff or tensioned cables in this situation is to reduce the active spans (i.e. $S' = S / N$; N is the number of evenly spaced bolts across the span).

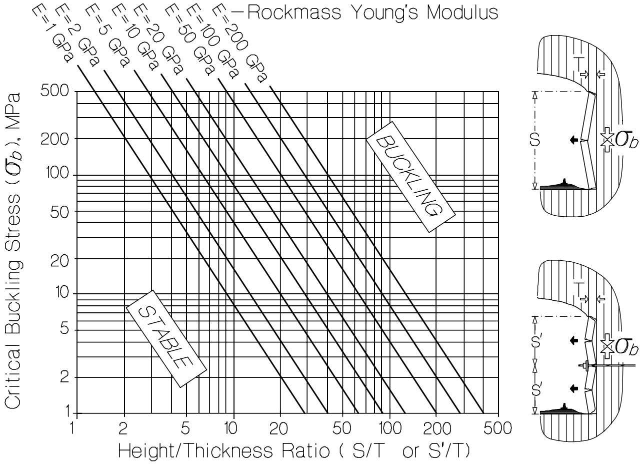


Figure 2.18.14: Buckling Analysis

2.18.11 Drift and Intersection Support

Cablebolts can be used to enhance primary support systems such as rockbolts or grouted rebar and screen, or shotcrete in drifts and tunnels. While primary support systems hold the surface of the excavation together, cablebolts perform the function of tying this reinforced skin back into sound rock. Cablebolts also provide added security under seismic conditions.

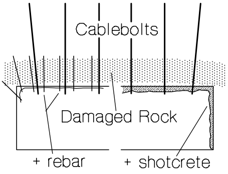


Figure 2.18.15: Drift Support

When cablebolts are deemed necessary for drift stability, ensure that their length is at least 3 metres beyond the maximum extent of yielding rock or beyond the apex of the maximum expected rock wedge. The design load capacity of the cablebolt system depends on the expected support function. If the cablebolts are expected to carry a full deadload of yielded rock, then the load capacity should be equivalent to a slab thickness of 30% to 50% of the span. This can be reduced to 10% to 20% of the span if the primary reinforcement can be expected to successfully create a beam (Section 2.18.12). The empirical tunnel support guidelines in Section 2.16 should also be of assistance. For excavations at depth, follow the logic described in Section 2.18.1 and in Section 2.13.3. Hoek and Brown (1980) present charts of induced stress, around various excavation shapes and in various stress fields, for comparison to rock strength. Detournay (1988) calculates depth of yielding around a circular opening in non-uniform stress fields. Wedge analysis in structured ground is described in Sections 2.18.8 and 2.18.9. Stability and support design in laminated ground is described in the next section.

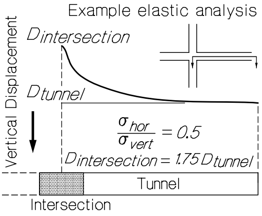


Figure 2.16.16: Vertical deflection of tunnel and intersection roof

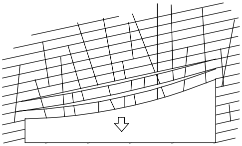
Intersections pose a special design problem. A major function of support is to transfer rock loads from the centre of the span to the abutments. At an intersection, the definition of the abutments is not immediately apparent and so additional logic is needed. Figure 2.18.16 illustrates typical vertical displacements along the centreline of a tunnel approaching an intersection. The displacement in the intersection is equivalent to that in a tunnel 1.75 times wider than the actual tunnel span. Since displacement ultimately affects stability, the following rule of thumb can be adopted.

Wedge stability analysis can be assessed directly using geometric techniques (Hoek and Brown, 1980) or using software as in Section 2.18.9. For other failure modes, analyze an intersection roof as an equivalent tunnel 1.75 to 2 times the maximum adjacent tunnel span or as a square with a width 2.5 to 3 times the span. Alternatively, for complex shapes, extend the entire perimeter of the intersection radially outwards by 1 tunnel span for the purposes of stability analyses.

2.18.12 Gravity Bending/Buckling: No-Tension Slab - Voussoir Approach

Open stope mining often involves excavation of large span openings parallel to laminated or bedded structure. When the discontinuity forming the lamination is the sole structural feature to be considered and the excavation technique is selected to minimize damage to the rock, standard beam analysis can be used to evaluate the stability of the roof or hangingwall. It is more often the case, however, that structures crosscut the main laminations reducing or eliminating the ability of the rock to carry tensile loads parallel to the lamination making standard elastic beam analysis inapplicable. In this case an alternative technique must be employed.

The problem is similar to the design and construction of mortarless masonry arches. The solution technique, which has become known as Voussoir analysis was first applied in rock mechanics by Evans in 1941. It has been modified over the years (Beer & Meek, 1982; Brady & Brown, 1985), correcting some earlier assumptions and improving the solution technique. The solution presented here represents a further development by the authors in order to correctly incorporate arch deflections and to incorporate more acceptable design confidence limits into the solution.



Assumptions:

- The out-of-plane depth of the beam is very large compared to the in-plane span. Only a unit depth is considered with all deformations occurring in plane.
- Cross-cutting structure is angled from the wall normal at significantly less than the minimum angle of friction assumed for the jointed surfaces.
- The beam is not capable of sustaining tension.
- As the beam deflects, a parabolic compression arch develops in the beam.
- Deflection of the beam occurs before slippage at the abutments. Stability against slippage is determined after the compression arch develops.
- Initial lateral stress resulting from in situ stress and excavation geometry is not considered in this analysis. The beam is assumed to be initially stress free.
- The abutments are stiff - they do not deform under the arching stress. For large span to lamination thickness ratios, the deformation of the abutments can normally be assumed to be negligible compared to the shortening of the roof beam.

The problem is statically indeterminate. This means that there is no explicit solution and that the iteratively obtained solution is approximate. A factor of safety of 1.5 to 3 is advisable. In addition, the solution is highly sensitive to rockmass modulus. The lowest expected value should be used. For a horizonal beam, the problem geometry is as shown below.

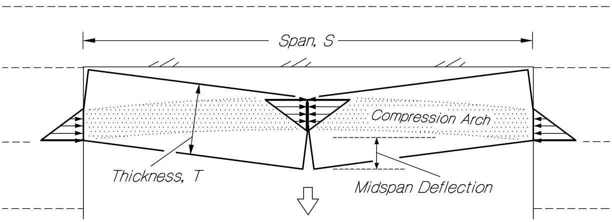


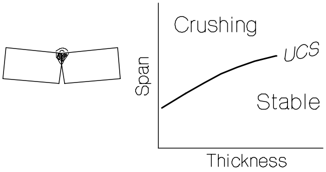
Figure 2.18.17: Problem geometry for Voussoir stability analysis

The following input parameters must be specified:

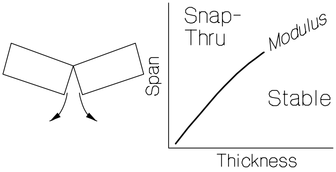
E = Rockmass stiffness (parallel to excavation surface), (MPa)
UCS = Uniaxial compressive strength of rock, σ_c, (MPa)
S.G. = Specific Gravity, (dimensionless) or
γ = Specific weight of rock
T = Thickness of continuous laminations parallel to surface, (m)
S = Span of excavation surface being analyzed, (m)
In the case of a long excavation, *S,* is the short dimension.
α = Inclination or dip of excavation surface, (degrees from horizontal)

Two failure modes are analyzed:

a) Crushing at the top and bottom of the beam resulting in beam failure when the compressive strength of the rockmass is exceeded.



b) Snap-thru at the middle of the beam resulting in immediate collapse. Controlled mainly by geometry.



Both failure modes are dependent on inclination and density and are most sensitive to rockmass modulus.

Calculation Procedure

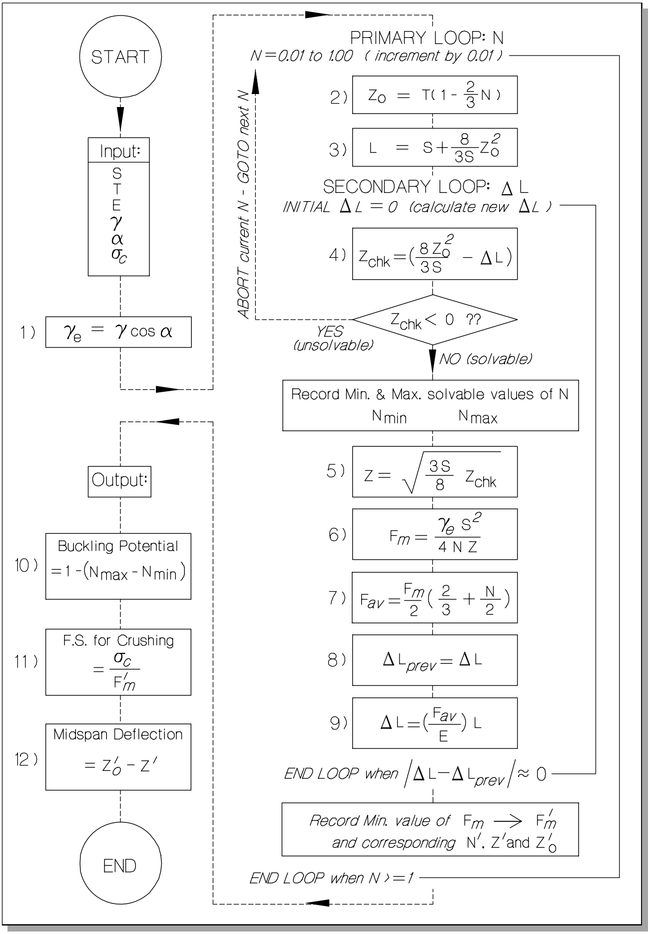


Figure 2.18.18: Calculation flow chart for the iterative Voussoir solution. Auxiliary variables include: *z*=arch thrust moment arm between centre and abutments); *Fm, Fav*=maximum and average arch stress; *ΔL*=arch shortening; *N*=ratio of arch thickness to beam thickness (0 to 1.0). Note that the *Buckling Limit* = proportion of unsolvable cases for *N*.

Deflection and Stability

Previously documented presentations of this solution have used *an absolute snap-thru limit* which is defined as the limit of stable deflection according to the mathematical formulation. This limit (*Buckling Limit* = 1) is extremely sensitive to lamination thickness, a difficult parameter to reliably estimate and one which may change as deflection and layer separation occurs. As a result, large safety factors have been recommended (Beer and Meek, 1982; Brady and Brown, 1985).

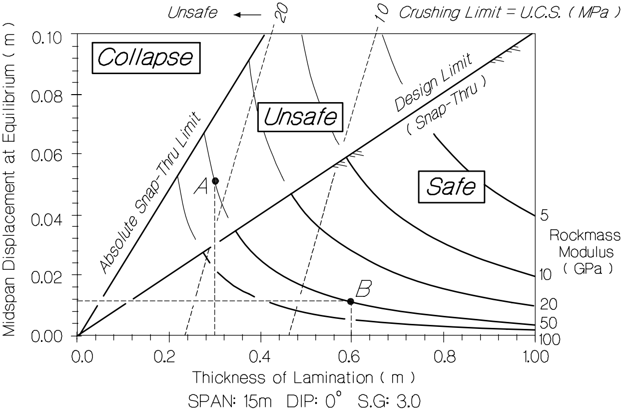


Figure 2.18.19: Limiting beam deflection for buckling and crushing failure modes

The stability charts which follow utilize a *design limit* for snap-thru which is based on a sensitivity or design confidence limit equivalent to a *Buckling Limit* of 0.35 in Figure 2.18.18. Beyond this limit (i.e. 0.35 to 1.0), small differences in thickness have an unacceptably large influence on stability. As a result of this adjustment, the charts which follow can be used with greater confidence in design.

Figure 2.18.19 above and Figure 2.18.20 also illustrate an interesting component of the analysis which becomes useful for excavation monitoring and design verification. Notice that for any span, inclination or rock modulus:

- The design snap-thru limit is reached when midspan displacements reach 10% of the lamination thickness. Beyond this deflection as in the case of example A in Figure 2.18.19, stability is unlikely.

This critical displacement (deflection at failure) may be further reduced by low compressive strength of the rock as crushing failure becomes dominant. Actual midspan displacement at equilibrium for a stable excavation surface is dependent on all of the input parameters (see example point B in Figure 2.18.19).

Deflection and Stability (cont)

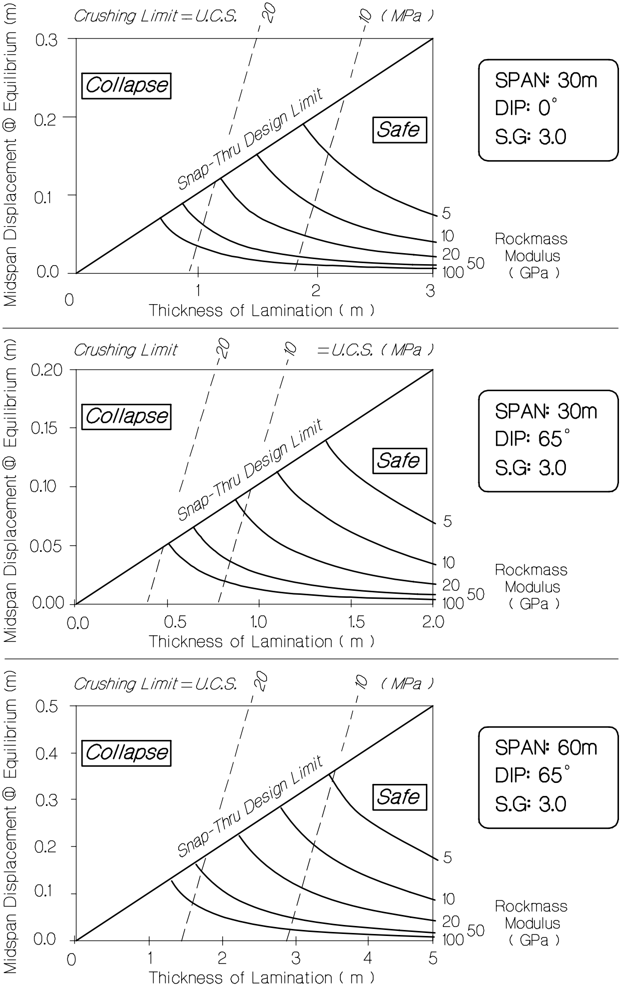


Figure 2.18.20: Limiting deflections for a variety of beam configurations

Span vs Thickness: Horizontal surfaces

Considering separately the two failure modes of snap-thru and crushing, design charts can be obtained as shown below for a horizontal excavation surface. A stable span plots below the design curve for the appropriate rockmass modulus, E_{RM} (snap thru), and for the appropriate compressive strength, *UCS* (crushing). In these charts, specific gravity is constant (S.G.=3.0) and ***b*** is the tunnel length.

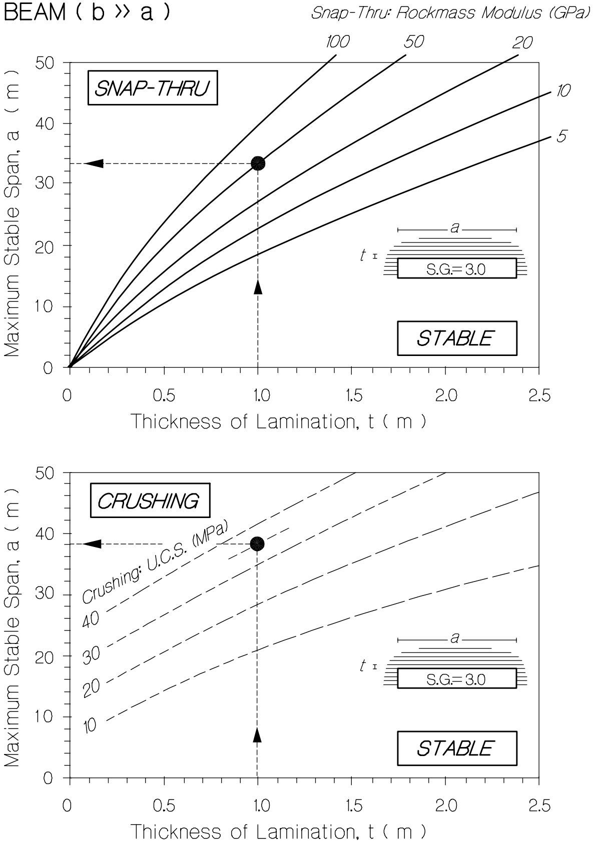


Figure 2.18.21: Critical (maximum stable) span for laminated horizontal backs

Span vs Thickness: Inclined surfaces

Voussoir analysis can be applied to inclined surfaces as well. Certain simplifying assumptions must be made which do not consider the distribution of pressures due to self-weight acting parallel to the beam. Nevertheless, a reasonable solution may be obtained and applied with the appropriate factor of safety (> 2). The following charts are for laminated walls inclined at 65 degrees.

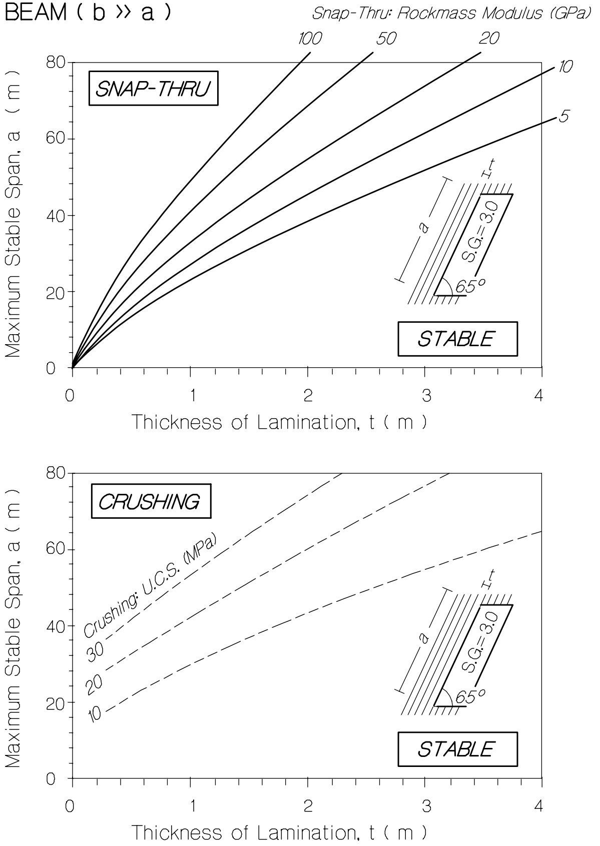


Figure 2.18.22: Critical spans for laminated hangingwalls inclined at 65 degrees

Span vs Thickness: General solution

The method can be generalized for any inclination and for any specific gravity. First calculate an *Effective Specific Gravity, S.G.*$_{eff}$, based on the actual specific gravity of the rock and on the inclination as shown below in Figure 2.18.23.

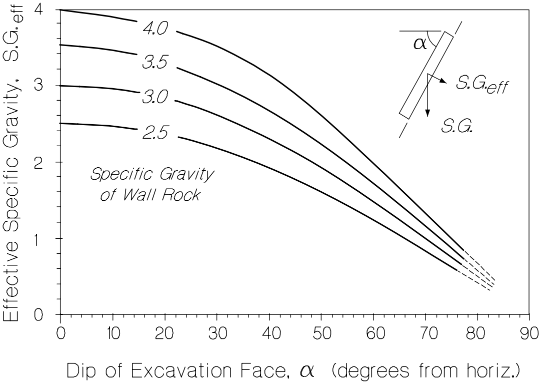


Figure 2.18.23: Effective specific gravity for generalized Voussoir analysis

The next step is to obtain the normalized modulus E' by dividing the actual modulus by the effective specific gravity. Next, the normalized compressive strength *UCS*' is obtained by dividing the real *UCS* by the effective specific gravity. Finally the maximum stable span for a beam can be found from the assumed thickness using Figure 2.18.24. Note that this chart and those on the preceding pages are applicable to a long stope wall with one dimension significantly longer than the other. The span used in the analysis is the short span. The results will be conservative.

A solution can also be obtained for a square stope surface (Brady and Brown, 1985). In this case, all four abutments contribute to the confinement of the beam or plate. As a result this analysis will give less conservative results (e.g. larger safe spans). The results for the general analysis are also given in Figure 2.18.24. Note that both crushing and snap-thru failure modes are combined on each plot.

Most excavation spans will be rectangular. These two charts, therefore, serve to bound the actual solution. Use both to obtain an upper and lower bound design. Note that these charts are based on a *Buckling Limit* of 0.35 (Figure 2.18.18). Critical spans based on a Buckling limit of 1.0 as in Beer and Meek (1982) and in Brady and Brown (1985) will be up to 20% larger.

Critical Span vs Thickness - General Voussoir Solution

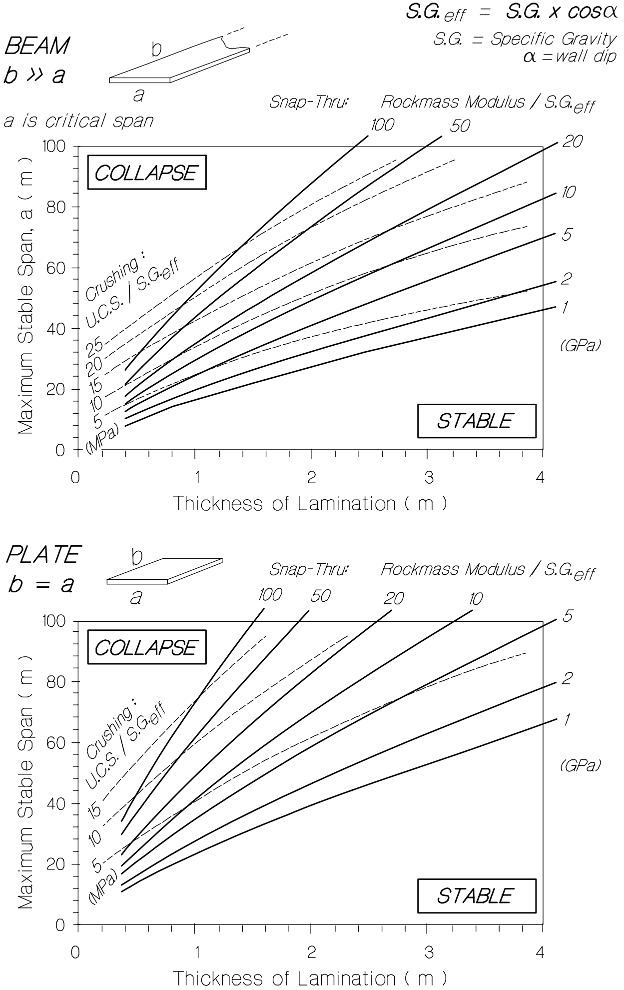


Figure 2.18.24: General solutions for Beam (infinite depth) and Square plate

Support Rationale - Voussoir beam

If the assumptions inherent in this analysis can be validated, it is possible to develop a support pattern to create a laminated beam or plate which will then prevent further destabilization of the wall. The role of cables here is two fold.

Firstly, cables near the abutments act to reinforce the joint surfaces, increasing resistance to internal shear which could lead to delamination and destabilization (smaller thicknesses have smaller critical spans). Topsill and bottomsill cable arrays (not shown in Figure 2.18.25) perform this role (Bywater and Fuller, 1983).

Secondly, cablebolts installed normal to the laminations and covering the span area should be designed as stiff reinforcement within the zone of rock equivalent in thickness to a self-supporting beam as calculated by this analysis. This is to prevent delamination though the central portion of the beam (Roko and Daemen, 1983; Stimpson, 1983; Snyder, 1983). Beyond this limit, an optimum cable array would have a more ductile response to allow the beam to deflect a small amount to generate the required compression for stability. Beyond this should be a suitable anchor length. Spacing as shown in Figure 2.18.25 is based on the deadload of the beam. If the cables can hold the weight of this beam, then stability should be assured. This result is usually much more efficient than a pure deadload estimate on a relaxing hangingwall (no beam formation).

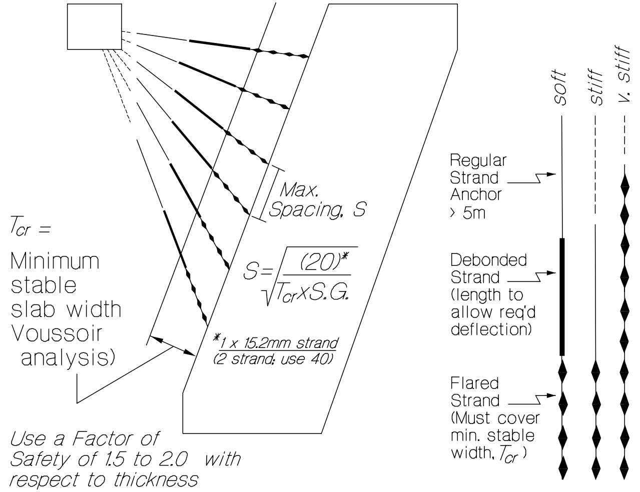


Figure 2.18.25: Cable spacing and length guidelines using Voussoir approach

2.18.13 Other Applications

Mandolin Bolting

Cablebolts are ideally suited for axial loading. For this reason cablebolt patterns are normally laid out to maximize the axial component of cable strain as compared with the shear component (Section 2.8.3). If the shear strength and stiffness of the cablebolt can be increased, then cable strand can be used as shear restraint parallel to the excavation face (Cutjar et al., 1985; Lappalainen and Antikainen, 1987). *Mandolin* style cablebolts installed in this way may also provide tensile capacity to the underside of a rock beam, similar to reinforcing strand in concrete structures (Beer and Johnston, 1992; Nickson, 1992).

For example, several cables can be inserted into steel pipe and lowered down a drillhole behind the stope wall as in Figure 2.18.26. Grout is pumped into the pipe and around the outside of the pipe. The cables extend beyond the collar and are tied back into a second lateral hole in the wall. This creates a rigid shear pin which in the case in Figure 2.18.26 provides shear reinforcement along a set of sliding joint surfaces. This method, while expensive, can eliminate the need for additional hangingwall development as required for conventional cablebolt fans. Note that without the pipe in this case the cables alone are unlikely to have enough shear stiffness to be effective support.

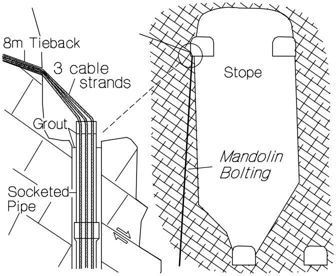


Figure 2.18.26: Mandolin bolting approach used in Australia (after Cutjar et al., 1985)

Mandolin bolting is used in Figure 2.18.27 in combination with other patterns to optimize reinforcement in areas where access for conventional bolting is limited. In this case the mandolin cablebolts provide both shear restraint and tensile strength. A rock beam with tensile strength on the excavation side is stronger than a no-tension beam (Section 2.18.12).

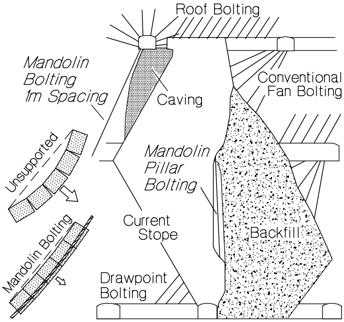


Figure 2.18.27: Mandolin bolting in combination with conventional cablebolt patterns (after Lappalainen and Antikainen, 1987)

Cable Slings

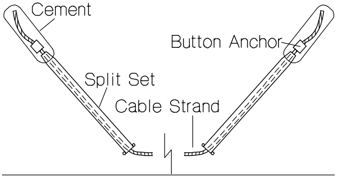


Figure 2.18.28: Cable sling components (after Castle and Scott, 1981)

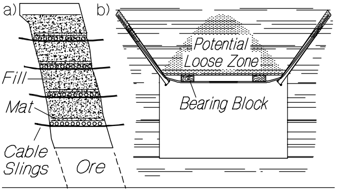


Figure 2.18.27: Cable sling applications
a) U/H cut and fill stopes ; b) roof support

Another approach to beam building is the cable sling (Figure 2.18.28) described by Raju and Ghose (1980), by Scott and Castle (1981) and Castle and Scott (1982). The cable sling can be used to support fill mats and cemented backfill in underhand cut and fill stopes (Figure 2.18.29.a) or as tunnel roof support (Figure 2.18.29.b). In the Scott system, a cable is inserted through two opposing Split Sets (Scott, 1976; 1983). Button anchors are fitted to the extreme ends of the cable. Two boreholes are drilled up and out from the corners of a tunnel roof and cement is placed in the ends of the holes. The split sets are pushed into the holes with the cables, tightening the intermediate cable length across bearing blocks to support the tunnel roof.

In the cut and fill application, the cable slings are installed across the floor of the current sublevel. Reinforced fabric mats, timber beams and a thin layer of strong cemented fill is placed on top. After set, the remaining backfill is then placed on top, and excavation can occur on the next level down, below the slings and backfill. This technique requires diligence and expertise to be executed safely.

Ore handling systems

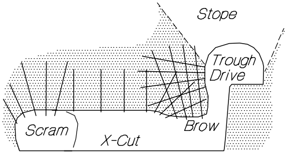


Figure 2.18.28: Drawpoint support (modified after Stillborg, 1986)

Cablebolts can be used to provide reinforcement and support of drawpoints and ore handling systems. Drawpoints normally encompass many of the more unfavourable conditions for rockmass integrity as well as plain strand cablebolt load transfer. They are typically overstressed during construction and relaxed during service. This reduces the rockmass stability and is detrimental to the capacity of plain strand cablebolts (Section 2.6). In addition the vibration and abrasion serve to destabilize existing and induced structural weaknesses. The design philosophy here, in short, is to use modified geometry strand, to use plates in backs where possible and to cablebolt from all accessible directions (Figure 2.18.28). Cablebolt support of orepasses has also been attempted (Clegg and Hanson, 1992) with limited success.

3 IMPLEMENTATION: Making the Design Work

3.1 Introduction

The implementation of the cablebolt design involves a variety of mine personnel working together through a series of tasks. The key steps in the implementation of the cablebolt system are:

- Assemble the cablebolting crew. The number of crew members and the skills required will depend upon the equipment in use and the tasks assigned to the crew. Establish the payment structure (hourly wage and bonus, or per contract item), and the contractual conditions for payment of the crew. (Section 3.2).

- Plan and conduct a training program for the underground personnel involved in all aspects of the cablebolt operation. Training sessions should be repeated on a regular basis. The personnel attending the courses should include the ground control or rock mechanics engineers and technicians, the cablebolting crews, the drilling crews, the underground supervisors, and the purchasing department. (Section 3.3).

- Communicate the cablebolt system design and installation procedure to the surveyors, drillers, cablebolters, underground supervisors and technicians. (Section 3.4)

- Install the cablebolt system, with good control on the quality of the installation. Any deviations from or problems with the design must be documented and reported. (Section 3.4)

- Monitor the quality of the installation procedure, both during the various installation procedures, and by observing the finished installations. Any problems with the quality of the installation must be reported and documented. (Section 3.5).

- Rectify any problems or difficulties found with the installation procedure, as soon as they are identified. Problem installations that are likely to compromise the performance of the cablebolt system should be replaced. (Section 3.5).

The information presented in this chapter has been compiled from a variety of sources, including observations of practice at underground mines, discussion with mine personnel about cablebolt design and installation and a review of the literature. Every effort has been made to reference published and accessible information. However much of the detailed information contained in this chapter can not be referenced in this manner. Therefore, thanks are given here to the numerous people who provided the practical hints, copies of company installation procedures and other information upon which this chapter is based.

The procedures and information contained in this chapter have been kept as general as possible, so as to be applicable to the widest variety of mine sites. The sample instruction and procedure sheets presented here are intended as guidelines, that after some modification should be suitable for the specific cablebolt applications, equipment usage and special circumstances of each site.

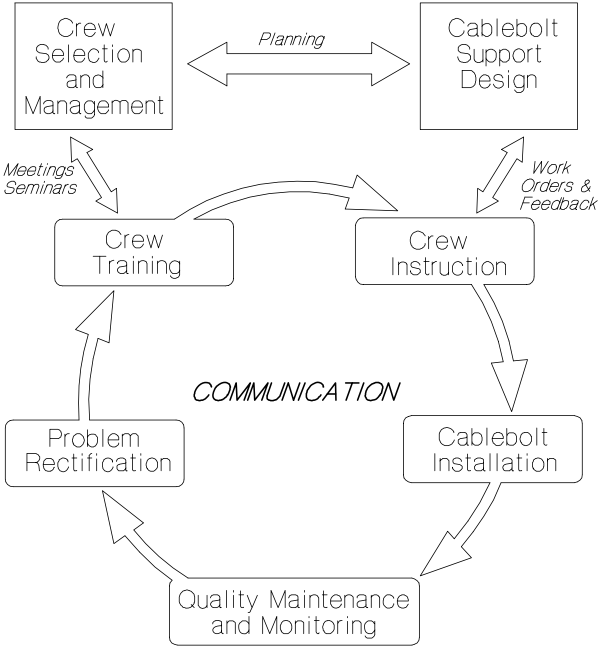


Figure 3.1.1: Key steps in the cablebolt implementation cycle

3.2 The Cablebolting Crew

The cablebolting crew is the most important component of the cablebolt installation process. If they do not install the system as designed and the cablebolts fail, then it is often very difficult to determine the cause of failure: Was the major cause of the failure poor installation practice or an inadequate cablebolt design?

There are several factors that influence a cablebolt crew's performance. They must be well trained, well equipped and supplied, be assigned to the job on an ongoing basis and work well together as a team. The best examples of crew conscientiousness and ability come from mine sites where cablebolting is one of the best paid underground jobs, where the crew have been well trained and stay with the job for a long time, and where there is frequent communication between the crew members, the supervisors and the engineers, so that any problems can be solved in a practical manner as soon as they are identified.

3.2.1 Crew Tasks

The list of operations that may be included in a cablebolt installation is:

1) Locate and orient the hole according to the layout sheets.
2) Drill the hole.
3) Cut the cablebolt strand length specified on the layout, or remove the pre-cut cablebolts from the palette.
4) Attach spacers at the designated places along the cablebolt strands.
5) Attach or form the end anchor or hanger.
6) Attach the breather and/or grout tubes.
7) Insert the cablebolt(s) into the hole.
8) Form the collar plug, or secure the cablebolt at the hole collar.
9) Thoroughly mix the grout to the specified water:cement ratio.
10) Pump the grout until there is visual proof of return of good quality grout.
11) Seal the end of the breather and grout tubes immediately after grouting.
12) Clean up the grouting equipment and working area.
13) Install the surface fixtures.
14) Report on the installation quality and any problems.

Surveying the hole locations, drilling the boreholes, and installing the cablebolts are often done by different mine crews. If possible, it is better to have the cablebolting crew drill the holes, so they are aware of any drilling problems that have occurred. Adding the responsibility of drilling the holes to the cablebolt crews' duties may increase the profitability and the appeal of the job as well (Nickson, 1992).

3.2.2 Crew Composition

The cablebolting crew can consist of one to three people, or more, depending upon the equipment in use and the installation procedure.

- A single person can complete the installation of the cablebolts alone when working with a well equipped, truck mounted, drilling / cable pushing / grouting system. Some mines have designed and outfitted their own cablebolting trucks, while others have purchased pre-assembled units such as the Tamrock Cabolter.

- A two person crew will be necessary when a scissor lift or fork lift mounted working platform is provided where the mechanical controls are not conveniently located on the platform. Two people are also required in situations where one person is operating the grout pump, while the other grouts the hole. It is better to have two people placing the cablebolts where long cables are inserted manually, so that they can share the work load.

- Three people are required where more than one of the conditions requiring a two person crew are found. For example, the members of a three person crew might: 1) operate the mechanical controls for the scissor lift platform, 2) place the cablebolts and tubes, plug the collars, and grout the holes and 3) assist with placing the cablebolts and operate the grout mixer and pump.

The crew leader should have the most experience with cablebolting and/or have demonstrated good problem solving abilities. The leader has the responsibility to monitor the quality of the cablebolt installation, and to report on any deviations from or problems with the established procedure. Promotions or transfers of crew members should be made on a staggered basis so that at least one trained and experienced person will remain on the crew at all times.

3.2.3 Crew Training

All members of the crew must be taught the correct procedure for all aspects of the installation procedure, as is discussed in Section 3.3. An important part of the crew training is to make the crew members aware of the ways in which the quality of the installation can be compromised, the consequences of quality control problems and what can be done to improve any problems with the installation.

Crew members must feel free to approach the underground supervisor(s) and the engineer(s) to discuss any problems with the cablebolting operation. As soon as any problems are identified, practical and workable solutions must be generated by discussion between engineering, supervision and the crew.

3.2.4 Crew Payment

The cablebolting crew must be paid at a reasonable level with respect to the other jobs at the mine. If cablebolting is one of the lowest paid jobs, then the crew members will request transfer to a better paying, cleaner job, and there will be no continuity in the crew members' training, leading to poor quality installations.

Crew members should keep a record of reasons for unavoidable down time so that their pay will not be reduced unfairly. This record should be reviewed periodically by the supervisors so that any problem areas are identified and solved.

The pay structure for the cablebolting crew at most mines is not linked to the quality of the job, but is based on the total length of cablebolts installed and grouted per shift. This policy often results in poor quality installations where time is saved by mixing the grout continuously instead of in batches, or by pumping the grout into the holes before it is completely mixed, using dirty cablebolts, by incompletely plugging the collar, or by not waiting for the return of design grout consistency along the breather tube or from the collar of a grout tube installation.

If the mine payment structure is such that a bonus is applied to the job, then consider implementing a system where the bonus is based both on the length of cablebolts installed and on the quality of the installation. It is important that the crew not be penalized financially for reporting problems associated with the installation and quality control of the installation, so ensure that they are well trained and proficient with the equipment, that the equipment is adequate, and that all quality control problems have been solved before instituting a quality control bonus. The quality control checks that can be made are discussed in Section 3.11.

When the cablebolting crew is employed by a contractor, a payment structure based on detailed quality control specifications should be implemented, and a quality control checking procedure established and agreed upon. Provision must be made for the mine engineer to reject any materials or installations that do not meet the standards established in the contract.

In some South African mines, the payment for support installed is based on the contracted crew meeting some readily achievable quota of cablebolts and on the bolts passing quality checks. The quality of the installation is inspected at least once a day by a Senior Supervisor employed by the mine to monitor quality control. The supervisor points out any problems to the contractor as soon as they are observed. The quality is also checked monthly and if the installation does not meet the level of quality control required, then no payment is made for the work, and the installation must be brought up to standard at the contractors' expense. A bonus is paid if the quality of the installation is good. Additional bonus is paid if the cablebolt quota was exceeded during the month (Thompson, P., 1993, personal communication).

3.3 Training

Training is an essential component of the cablebolt implementation process. The training course should educate everyone involved in the design and the implementation of the cablebolt system so that they understand the function of the cablebolts, the proper procedures for all of the equipment in use, and the consequences of poor quality control. It is also important to establish the responsibility of each department for each step of the implementation process.

The information to be imparted during the training sessions can be structured by answering the questions Why?, What? and How? with regard to the cablebolting process. Methods for providing feedback on the installation process and safety issues must also be discussed during the course.

A suggested table of contents for the training course is given in Table 3.3.1. The table makes reference to the sections in this book where further information on each subject in the training course can be found.

The training material must be updated as new techniques and products become available, if problems with installation of regularly observed or as company policies are altered. The training program should be simple and easy to conduct so that it can be done at any time that it is thought necessary.

Anyone who is involved with designing, purchasing materials for, supervising, checking or implementing the cablebolt system should attend the training sessions. There may be as many as three different underground crews working on the cablebolt installation on each shift, including the surveying crew, the drilling crew and the cablebolt installation crew. All members of these crews should attend the courses.

Training courses should be conducted frequently. The entire course should be taught annually so that the information discussed during the course remains fresh in the minds of the personnel involved with the cablebolting process. Periodic repetition of the training courses also provides the opportunity for review and discussion of the design and implementation of the cablebolt system. Whenever new people join the cablebolting crew, or if the performance of the crew appears to be declining, the appropriate sections of the course should be repeated.

A "break in" period should be established after the first course has finished to give the crew time to become proficient with the equipment and procedures. During this period of time, the engineers, technicians and supervisors should visit the crew more often than usual to observe their work and to solve any problems with the installation procedure that might arise.

Whenever different materials or equipment are to be used in the cablebolt operation, the engineer should work with the crew to check that the new items will perform as expected, and to sort out any problems with the installation procedure.

Copies of relevant drawings, tables and procedural instructions should be provided to course attendees and be regularly updated as procedures or equipment change.

Table 3.3.1: Suggested table of contents for the cablebolt training course.

Module 1: *Why use cablebolts?* Section 3.3.1

1) Purpose of cablebolting.
2) Application(s) of cablebolting at the mine.
3) Function of cablebolts

Module 2: *What is a cablebolt?* Section 3.3.2

1) What is a cablebolt?
2) The components of a cablebolt element.
3) The optimum cablebolt element for the site.
4) How does a cablebolt work?

Module 3: *How are cablebolts installed and checked?* Section 3.3 3

Classroom discussion:

1) The cablebolting cycle, with emphasis on where the implementation process fits into the cycle.
2) The steps in the implementation process.
3) Overview of the cablebolt installation method(s).
4) Cablebolt layout and design specification sheets.
5) Implementation procedures and safety.
6) Quality control guidelines.
7) Reporting on the cablebolt installation process.
8) Monitoring the cablebolt installation process.

Hands on training:

1) Purchasing and handling the materials.
2) Installing the cablebolts using layout, procedure and observation sheets.

Module 4: *Safety.* Section 3.3.4

1) Review and discussion of safety issues.

Module 5: *Feedback on the installation procedure.* Section 3 3 5

1) Review and discussion of feedback recorded on the layout sheets.
2) Review and discussion of feedback recorded on the observation sheets.
3) Review and discussion of the quality control check list.

3.3.1 Why Use Cablebolts?

The intent of this module is to explain in general why support is used at the site and in particular why cablebolts are used in certain environments. This discussion should be very specific to the mining environment and also to the trainees' level of knowledge. Some points that should be included in the discussion are:

1) Purpose of cablebolting. This may include such items as improving the safety of workplaces and the stability of the stope boundaries as well as reducing dilution and oversize (Section 1.2).
2) Application(s) of cablebolts, including a general discussion (Section 1.3), site specific support patterns and the intended support mechanism of the cablebolt systems in use at the mine site.
3) An introduction to cablebolt functions (Section 1 6) is useful to avoid some on-site adjustments to procedure which may impair support effectiveness.

3.3.2 What is a Cablebolt?

This module describes a cablebolt, including:

1) A description of a typical cablebolt (Section 1.1), including the basic steel strand, the cablebolt toolbox (Section 1.5), grout and any surface fixtures. Detail can be added from the information presented in Chapter 2.
2) An introduction to the components of the cablebolt element in use at the site. The cablebolt element includes the cablebolt strand(s), the installation components such as hangers and tubes, the grout mix and any surface fixtures. Describe all of the combinations of the element components that will be used in different applications at the mine.
3) A description of the optimum installed cablebolt element. This will depend on numerous factors and can only be fully developed once the cablebolt system has been designed and selection of the best implementation procedure has been made. As an example though, an optimum cablebolt element could be a clean cablebolt installed at the exact position designed, in the centre of the hole and with a full column of completely mixed grout of the correct water:cement ratio. The optimum cablebolt could also include a clean plate of the design dimensions securely fastened with a clean, matched barrel and wedge anchor to the end of the correctly tensioned clean cablebolt strand. Some suggestions for the conditions for optimum installation are given in the left hand column of the quality control guideline tables in Sections 3 8.3 and 3.9.3. A discussion of the items which are applicable to your site and usage of cablebolts would provide useful information for the crew and supervisors.
4) A discussion of how a cablebolt works, including the interaction of plain, multiple and modified strand with the surrounding grout. Include a brief discussion on the impact of stress change (Section 2.6).

3.3.3 How are Cablebolts Installed and Checked?

The cablebolt implementation module should include a general overview of the implementation process, a detailed discussion of the implementation steps and associated instruction documents, and a hands on training session.

Samples of the sheets used in this module are given in Sections 3.7 to 3.9:

Design specifications:	Material purchasing Cablebolt layout: Plan and Section Cablebolt installation: Layout and Notes
Procedure and safety:	Material handling Borehole drilling Cablebolt placement Grout mixing and pumping Surface fixture installation
Quality control guidelines:	Cablebolt borehole preparation Cablebolt placement Grout mixing and pumping Surface fixture installation
Feedback:	Drilling observation report Cablebolt installation observation report Cablebolt quality control check list

The *classroom session* should include discussion of:

1) The need for effective communication throughout all aspects of the work (Section 3.4).
2) How the implementation process fits into the cablebolting cycle. This discussion should provide the trainees with an overview of the cablebolting cycle (Section 1.4), including general information about the design and verification processes, so that they will understand the need for feedback.
3) The steps in the installation process (Section 3.6). Everyone involved in the process should know where they fit in to the whole picture.
4) The installation method(s) that will be used at the site. Discuss the different installation methods (Section 1.8), and the advantages and disadvantages of each.
5) Describe the range of equipment available on the site. A sample list is provided in Section 1.7.

6) The cablebolt layout and design specifications: Sections 3.4, 3.6, 3.7.1, 3.8.1, and 3.9.1. Work through the sample sheets during the course to make sure that everyone understands the information contained in the sheets and when feedback is required.
7) Implementation procedures and safety, modified from samples provided in this book and modified to reflect conditions at the site: Sections 3.6, 3.7.2, 3.8.2, and 3.9.2. Discuss the contents of the sheets briefly, emphasizing the allowable tolerances for deviation. This information can also be reviewed and is easiest to teach in detail during the hands on training session.
8) Quality control guidelines: Sections 3.5, 3.7.3, 3.8.3 and 3.9.3. This discussion must emphasize that the cablebolt will only perform as designed if the installation quality control is good. Discuss the correct procedures for good quality control and solutions for commonly encountered quality control problems.
9) Reporting on the cablebolt installation process: Sections 3.4, 3.5, 3.6, 3.7.4, 3.8.4 and 3.9.4. Discuss the observation sheets that are to be completed by the crews, and the possible uses of the information recorded on these sheets.
10) Monitoring the cablebolt implementation process: Sections 3.5 and 3.11. Describe the methods used for monitoring the grout quality during installation and the check lists that the supervisors will complete after each visit to the work site.

The *hands on training session* can be conducted in a working area that is to be cablebolted, or anywhere else using pipe pumping tests. In the latter, a length of pipe simulates the borehole. One of the advantages of the pipe pumping tests is that the pipes can be cut apart after the tests to check the completeness of the grout column. A description of the materials required for and the procedure to be followed during a pipe pumping test is given in Chapter 2.

The installation trials provide the opportunity to catch quality control problems before they become established installation procedures. In addition, it will be apparent during the trials whether the equipment and materials are adequate for the specified design and work site conditions. The hands on training sessions should expose all of the people involved in the implementation process to:

1) Selection and handling of materials following the procedures discussed in Section 3.7. Collect samples of the different materials available and create examples of well stored and handled materials and of materials that should be rejected due to deterioration of their quality. It is instructive for everyone involved in the cablebolt implementation process to see the differences between the well stored and poorly stored samples.
2) Installation of cablebolts using the sample layout, procedure and observation sheets that were discussed in the classroom sessions. Everyone should complete the feedback sections on the layout and specification sheets, the observation reports and the quality control check lists.

3.3.4 Safety

Safety pointers are included in the procedures but should be discussed during the installation training as well. A list of some safety points is given in Section 3.6.1. Review the safety guidelines after the hands on training to reinforce the ideas, and to ask the trainees for additional safety suggestions. Add useful suggestions to the list and to the procedure instructions.

3.3.5 Feedback on Installation Procedures

Feedback is essential, and this module reviews how it is recorded and explains how the reports are used. The following points of discussion should be covered during the training course:

1) *Daily feedback.* The crew is required to provide feedback on a daily basis about the drilling and installation process on the layout sheets, including any deviations from design due to insurmountable problems, and usage of cablebolts, cement, and other materials. Notes of layout changes must be recorded so that they can be considered when back-analyzing the performance of a cablebolt layout or system design. If layout changes occur frequently, then the reason for the changes should be determined and the problem solved. For example, if the grout rarely returns down the breather tube, then the diameter of the breather tube, the grout fluidity, and the pump capacity should all be examined to try to determine the source of the problem.
2) *Periodic feedback.* The underground supervisors should also complete a check list for quality control issues after they have observed the crew(s) working underground. A sample quality control check list is given in Section 3.11 an should be modified to reflect the specific situation at the site prior to use in the training session at the working face. The information recorded on the check list should be used to determine if there are any recurrent problems in the installation process which need to be worked on, and where applicable, to calculate the payment of bonus to the crews.

It would be instructive to work through an example of how data recorded on feedback or observation forms could be used by the engineer. After the introduction of these forms, as everyone at the site becomes familiar with the cablebolting cycle, some examples from the site should be incorporated into the course material. In the meanwhile though, the case history of quality control improvement given in Section 3.12 could be given to show the utility of communication and feedback. In the case example, the quality control problems only came to light when large wedges fell from the drift back. Had the drilling. installation and quality control guidelines been in place prior to the failures, the mine personnel might have been alerted to the problem sooner.

3.4 Communication

Communication between all people involved in the cablebolt implementation process is important to ensure that the operation runs smoothly and efficiently, and that any problems are resolved as soon as they arise.

There are two levels of communication that should occur: regular and periodic.

Regular communication should include:

1) The engineer communicating the design layout and specifications to the drilling and installation crews: *Layout and specifications*.
2) The engineer instructing the crews in the procedures to be followed during the implementation process. Safety issues must be included in these instructions: *Procedures and safety guidelines*.
3) The engineer and supervisors informing the crews of the potential quality control problems and their solutions: *Quality control guidelines.*
4) The crews providing daily feedback on the installation process, including usage of materials, any problems with equipment and materials and any deviations from the design as specified on the layout: *Feedback.*

Periodic communication involves:

1) The supervisors, engineers or technicians recording their observations of the quality of the installation during spot checks: *Quality control check list.* This information should be reviewed frequently by the engineer.
2) The engineer discussing, with the crew, any problems that have been identified with the installation process . The presence of problems could be indicated by the comments recorded on the feedback forms, on the *Quality control check list* or by the results of grout tests.
3) All personnel involved in the cablebolt cycle discussing the design, installation procedure and verification of the process during *monthly meetings.* The monthly meetings should provide a forum for the transfer of information and discussion on the installation method, equipment and materials. A suggested agendum for the monthly meetings is:

    - General review of operations for the month, including the total length of cablebolts installed, crew productivity and materials consumed. These items should be compared to the budgeted work.

    - A review of the work planned for the next month.

    - Review of post-installation quality control checks. Resolutions must be found for any quality control problems found during these checks.

- Discussion of any quality control problems reported by the crew, supervisors or engineers on the feedback reports or on the quality control check list. Review of the solutions adopted and discussion of other possible solutions that might also work.

- Discussion of any procedural problems, and possible solutions.

- Description of any changes to the installation procedure.

- Suggestions for improvement to the cablebolt system design or installation procedure.

- Discussion of any new products under review or available: advantages and disadvantages.

A spreadsheet listing the annual cablebolt budget, which details the metres of cablebolts required for each planned stope, should be produced and available to crew members (Boaro, J., 1993, personal communication). This spreadsheet would aid cablebolt planning and progress review.

The communication of feedback is essential so that any problems with the installation equipment or any suggestions for improvements to the cablebolt design or installation procedure will be considered as soon as they arise. Changes to the cablebolt layout due to problems encountered during the installation process should be recorded for back analysis of any subsequent problems with support performance. If it is felt that the installation problems are severe enough to compromise the capacity and designed function of the cablebolts, then additional cablebolts should be designed and installed in the problem area.

Copies of the original design specification sheets, and the feedback comments must be kept in a readily accessible place. Then if a rockmass or support system failure occurs, a description of any problems encountered during the installation will be available as basic information for the evaluation of the cause of the failure. A failure report documenting the volume of and suspected reason for the failure, assessing the performance of the cablebolts, and discussing anticipated influences of the failure on adjacent unmined stopes should also be filed.

The crew productivity can be assessed periodically from the information recorded on the reporting sheet. Reasons for drops in productivity should be determined and corrected, and most importantly, suggestions for productivity enhancing changes evaluated and implemented quickly.

3.5 Quality Control Practice

The cablebolts must be installed with the best quality control possible. The designed function of the cablebolts can be severely reduced if the quality of the installation is poor, as is discussed in further detail in Section 3.11. The maintenance of good quality control during installation is the responsibility of everyone involved in the implementation process:

1) The crews should be encouraged to monitor the quality of the materials, equipment and the installation process, and report any problems as soon as they arise even if the crew has solved the problems during the shift. The *Quality control guidelines* presented in this handbook are intended to be given to the crews by the engineer along with the design specifications and procedures. The guidelines list good practice procedures and solutions for some installation problems. Samples are given in Sections 3.8 3: *Cablebolt borehole preparation* and 3.9.3: *Cablebolt placement*, *Grout mixing*, *Grout pumping*, and *Surface fixture installation*. The guidelines should be reviewed by the crew periodically, and at any time when the installation procedures change. Additions and improvements to the guidelines are encouraged as merited by unique site conditions or by experience and procedural improvements.
2) The underground supervisors, technicians and engineers should also monitor and report on the quality of the materials, equipment and the installed cablebolts during spot check visits to the crews. Observations of the installation process should be recorded on the *Quality control check list* (Section 3.11). Solutions to any problems should be found as soon as possible in consultation with the crew and implemented immediately.

When quality control problems are found, their impact on the cablebolt system function should be evaluated as discussed in Section 3.11. If the system capacity is severely compromised by poor quality control, then additional cablebolts may have to installed in the working area.

Where the payment of bonus to the crews is based in part on the quality of the installed cablebolts, reported, unavoidable problems should not be penalized by reduction of the bonus payment. Solutions to these problems for future installations should be investigated and implemented as soon as possible.

New equipment and hardware is constantly being developed and supplied to the mining industry. These improvements are welcome but greet with skepticism any claims that quality control is no longer required. Old quality issues may become less critical but new ones will arise. It is better to maintain a high standard of quality regardless of the technology in place. Diligence costs little and always brings rewards.

3.6 Installation

The cablebolt implementation process involves several steps as shown in Figure 3.6.1. Instructions for each of these steps are given in the procedural guidelines presented in the following pages. Procedures are also discussed by Hunt and Askew (1977) and Schmuck (1979).

The guidelines presented here have been kept general, in an attempt to cover all aspects of the procedure for each of the installation methods presented in Chapter 2. Create the following set of procedural guidelines and feedback report forms, specifically for your site and use of cablebolts and equipment:

1) Material purchasing specification.
2) Material handling procedure.
3) Borehole drilling procedure.
4) Drilling observation report.
5) Cablebolt placement procedure.
6) Grout mixing and pumping procedure.
7) Surface fixture installation procedure.
8) Cablebolt installation observation report.

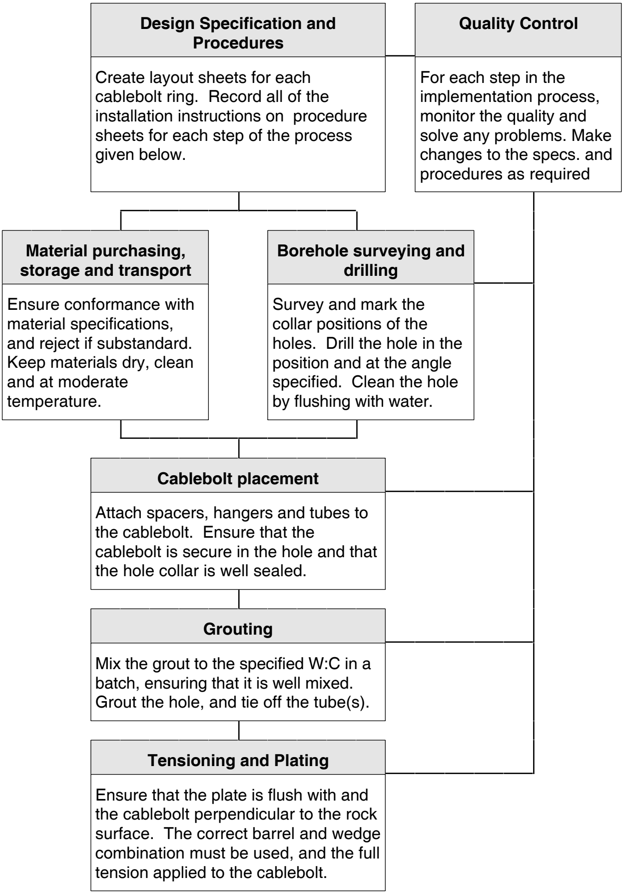
In the sample procedures, a number of entries have been printed in italics. These and any other items should be changed to reflect the practice and procedures appropriate to your usage of cablebolts before the sheets redistributed.

In any installation, problems may arise at the working face, such as not being able to set up the drill rig in the exact position required by the design. In the general procedure guidelines, an attempt has been made to give some allowable tolerances for deviations from design. Then if problems do arise, the crew can alter the design within the specified limits at the working face, and will not be delayed. Allowable tolerances have been included in the following guidelines in italicized text. When creating the set of procedural instructions for the crew(s), alter these tolerances to reflect the conditions of your site. The specific instructions for the installation of each cablebolt are presented on the following design specification sheets.

1) Cablebolt layout plan.
2) Cablebolt layout section.
3) Cablebolt installation layout specification and notes.

These sheets provide all of the information required by the drilling and cablebolting crews to completely install the cablebolts designed for a particular working area. These sheets also provide opportunities for the crew(s) to provide feedback on the installation process.

Implementation



Design Specification and Procedures

Create layout sheets for each cablebolt ring. Record all of the installation instructions on procedure sheets for each step of the process given below.

Quality Control

For each step in the implementation process, monitor the quality and solve any problems. Make changes to the specs. and procedures as required

Material purchasing, storage and transport

Ensure conformance with material specifications, and reject if substandard. Keep materials dry, clean and at moderate temperature.

Borehole surveying and drilling

Survey and mark the collar positions of the holes. Drill the hole in the position and at the angle specified. Clean the hole by flushing with water.

Cablebolt placement

Attach spacers, hangers and tubes to the cablebolt. Ensure that the cablebolt is secure in the hole and that the hole collar is well sealed.

Grouting

Mix the grout to the specified W:C in a batch, ensuring that it is well mixed. Grout the hole, and tie off the tube(s).

Tensioning and Plating

Ensure that the plate is flush with and the cablebolt perpendicular to the rock surface. The correct barrel and wedge combination must be used, and the full tension applied to the cablebolt.

Figure 3.6.1: Steps in the implementation process.

3.6.1 Safety Guidelines

There are numerous safety issues that should be considered during the installation of cablebolts, which must be stressed during the training course. The following guidelines provide a partial list only, and must be expanded to cover all aspects of safety at the site. The safety guidelines should be repeated in the appropriate procedures to stress their importance. Review these guidelines regularly to ensure that they are up to date.

- Ensure adequate ventilation is turned on.

- Collect all required supplies, tools and safety equipment.

- Review all hand and light signals with new crew members.

- Make the working area safe by scaling down loose rock.

- Clear the floor of the working area, to allow for good footing.

- Store tools, materials and equipment in safe, out of the way places to avoid tripping hazards.

- Use a certified safety restraint system (e.g. rope, lanyard or full body harness) when working on platforms.

- Stand in the middle of the roll of cablebolts when cutting the packing straps. Cut the straps in the correct order to allow the cablebolt strand to unroll slowly, in a controlled manner. Ensure that all other crew members are far enough away to be out of the range of any springing or lashing cablebolts.

- Wear a face shield, safety glasses and work gloves when cutting cablebolts with an abrasive cutter, since small shards of metal and sparks are created.

- Adopt "safe-lifting" practices when pushing cablebolts into holes. Guide the free end of the cablebolt element as it is being inserted into the hole so that it doesn't whip about.

- Do not stand directly beneath holes, especially when sealing the collar with grout or resin or when grouting the hole.

- Ungrouted cablebolts can fall out of upholes, with the potential to injure people working below. Post a warning sign, which indicates that there are ungrouted cablebolts overhead, at all entrances to the work area.

- Grout can severely burn exposed skin. Wear glasses, gloves and protective clothing when working with cement powder and grout. Barrier cream should be used on any exposed skin, and on the hands. If any grout does touch bare skin, the area should be washed immediately with soap and water and completely rinsed. If grout gets into your eye, flush with fresh water, and immediately report to the First Aid station.

- Ensure that water and air supplies are turned off while assembling and disassembling the mixer and pump.

- All clothing and hands must be clear of the mixer paddles before starting the mixer motor.

- Wear a dust mask when breaking open and handling bags of cement.

- Ensure that the grout and breather tubes are of the correct pressure rating, and have not been damaged in any way.

- The pressure created during the grouting operation has the potential to burst the grout hoses or tubes. Turn the pump on slowly and keep the pumping rate at a slow speed to prevent excess pressure build up. On the other hand, do not reduce the pressure too much, or else the pump will stall.

- Use caution when disconnecting hoses and tubes, because pressurized grout may still be present in them.

- Clean up all empty cement bags and cotton waste, as they are a fire hazard.

The safety clothing and equipment provided to the cablebolting crews should include:

- Oiler or other tough waterproof pants.

- Long water proof gloves and barrier cream.

- Safety glasses, full face shield or protective goggles, and antifog solution.

- Dust masks.

- Hearing protection.

- General safety equipment including work gloves, safety ropes, lanyards, well fitting safety belt and correctly sized full body harness.

3.7 Material Purchasing and Handling

The equipment required for the cablebolting job, such as grout mixers and pumps or tensioning jacks, is discussed in Chapter 2, and is generally purchased once every few years. This section covers the materials that are consumed on a regular basis by the cablebolting operation.

The general list of materials required for cablebolt installation should be specified by the engineer so that the crews can ensure that they have all of the materials required for a particular job. A sample list of required materials given in Section 3.7.1 is fairly general, and should be discussed with the crew to ensure that it is complete.

Quality Control

As the materials arrive on the site, they should be checked for quality, and rejected if the quality is poor, as discussed in Section 3.7.2. The materials are generally stored in a cool dry location on surface, and transported underground as required. The storage period at the mine site should be minimized by ordering smaller quantities of materials more frequently, to reduce the chance of deterioration of the quality of the material. All people working on the storage, transport and handling of the materials must endeavour to keep them as clean and dry as possible.

The quality of the materials purchased is very important as well. All materials must meet or exceed the specifications supplied by the engineer. For example, the cheapest cement or cablebolt strand may not have the performance characteristics required for the design. Where cost savings appear to be substantial for an alternate material, the purchasing department should consult with the engineer before trying a new product. If the specifications of the product are not proven, then field trials of the product should be arranged and conducted prior to volume purchases.

Feedback

Lapses in quality of newly shipped materials should immediately be brought to the attention of the supplier so that the problem cab be dealt with immediately.

It is advisable for the purchasing department or the engineer to keep abreast of the new development of new materials or equipment. The last few years have seen great changes in the cablebolting market. As more experience is gained and as more mines use cablebolts in different environments, manufacturers of cablebolting hardware and equipment can implement improvements based on this experience. Feedback from the mines to the suppliers and manufacturers should result in continued product development and refinement.

3.7.1 Design Specifications

Specification: Material purchasing (Example)

Stope:

Material storage location:

Date materials required at the storage location:

General:

* Axe, hammer, hacksaw, vice grips, crescent wrench, pipe wrench, electrical or duct tape, knife, air powered cutter or grinder with blades, string, tape measure.

Hole drilling:

Long hole drill.
3' to 4' drill rods, rigid couplings and one centralizing coupling.
Steel rod rack.
* Sample bags for drill cuttings.
* *2 1/2"* diameter drill bits.
* Air and water hoses.

Placement: (This example is for a breather tube installation of 300 cablebolts, with a 10% surplus contingency, and using Procedures A1, B3, D2, E3, F1, G1, H1, I1, J1, K2 and L1 in Section 3.9.)

330 pre-cut plain strand cablebolts of *10 m* length, *with nut for hanger attached*.
660 m of *19 mm* I D. *250 psi* pressure rated grout tube.
4300 m of *13 mm* I.D. *100 psi* pressure rated breather tube.
990 7 5 cm by 2 cm spring steel clips.
330 4 5 cm diameter washers.
330 wooden wedges.
Cotton waste.

Grouting:

* *Spedel 6000* grout mixer and *3100* pump.
* *2 x 20 litre* clean, empty pail.
* 2 cement scoops.
35 grout tube connectors.
660 25 kg bags of Portland Type 10 cement, supplied on wooden palettes wrapped in plastic.

Surface fixture installation:

330 20 cm by 20 cm by 1 cm flat plates.
330 wedge and barrel sets.
* Tensioning jack.

* Note that the items marked with a * are purchased periodically and will not normally be specified for purchasing on a stope by stope basis. In addition, these items are generally kept with the crew at all times. It is the crew members' responsibility to request from the purchasing department these materials when they are required.
# The items marked with a # are purchased once every few years and are generally with the crew at all times.

3.7.2 Procedure and Safety

Procedure: Material handling

Procedure:

1. Order the materials listed on the purchasing specification sheet. If the exact materials specified are not available, discuss the alternatives with the engineer.

2. Check the materials on arrival at the mine site for quality. Reject any shipments of sub-standard materials and record the reason. Standards are listed below.

3. Store the materials in a dry, clean, shaded area which remains at a moderate temperature (between 20 and 40 C). Do not stock pile the materials for too long, otherwise their quality will start to deteriorate.

4. Deliver the materials required for each working area at least one shift before the start of work in that area.

5. If the crews report any problems with the quality or ease of use of the materials. This should be discussed with the engineer.

Material quality standards:

Cablebolt strand:

- Clean and free of rust and grease or oil.
- No nicks or surface damage to the steel strand.
- Coils no smaller than the minimum specified diameter.

Cement:

- Stacked on a wooden palette which is well water proofed with plastic wrapping.
- Do not stack more than 2 palettes high.
- Lump free.
- Do not leave in the sun for long.

Placement materials:

- Grout and breather tubes must arrive well secured in a roll, be clean and have no kinks or bends.
- Fabric for collar plugging must be soft, and not too stiff.

Surface fixtures:

- Plates, wedges and barrels clean and free of rust.
- The wedges and barrels must match - when placed over a piece of cablebolt, the narrow ends of the wedges must not protrude through the end of the barrel, and the tops of the wedges must be at least 10 mm above the top of the barrel.

Please make a note of the date and supplier of any materials that are rejected due to sub-standard quality and the reason for the rejection.

3.8 Cablebolt Borehole Preparation

Sample versions of the *Cablebolt borehole preparation* sheets for:

Design specification:	Cablebolt layout plan and sections.
Procedure and safety:	Borehole drilling procedure.
Quality control:	Cablebolt borehole preparation quality control guideline.
Feedback:	Drilling observation report.

are given in the following text. These sheets are samples only, and should be modified to suit the particular cablebolt application, specific drilling equipment in use, and safety concerns at your mine. At some sites, several versions of these sheets will be required to cover all possible combinations of cablebolt application and installation procedure.

The *Cablebolt layout* design specification presents the information required for locating the cablebolt holes. This sheet is used by the survey crew when locating and marking the collar position of the boreholes on the rock face. The surveyors will either mark the collar location of every hole, or just the middle of each cablebolt ring, depending upon the accepted practice at the site. If there are any problems with locating the hole collar positions as designed, the surveyors should note on the layout sheet the new distance between the reference point and the centreline (℄) of the ring.

The cablebolt boreholes are then drilled by the drilling or cablebolting crew at the angle and to the length detailed on the *Cablebolt layout* sheet, following the instructions given on the *Borehole drilling procedure* sheet. If the crew have any problems with drilling the holes at the specified pattern, the layout can be altered within the allowable deviations given in the procedure. Changes that are made to the collar location or the borehole length or inclination should be noted and drawn on the *Cablebolt layout section* sheet. Changes to the position of the cablebolt ring and the date on which each ring of boreholes was drilled should be recorded on the *Cablebolt layout plan* sheet.

During borehole drilling the crew should monitor the following items:

1) *Loss of drilling water.*

Drilling water loss indicates the presence of an open structure such as a joint, fault or void. Where there are open joints, there may be problems with grouting the borehole, if the grout flows away into the rockmass. Section

3.9.2: J3 to J5 describes techniques for grouting in jointed rock. In addition, open structures or voids can have a great deal of influence on the rockmass behaviour. Reports by the drilling crew of such features must be evaluated by the engineer to determine if unexpected rockmass failure could occur.

2) *Change in drilling rate.*

An accelerating drilling rate indicates that softer rock has been encountered. Softer rock provides less confinement for the cablebolt, reducing the bond strength of the cablebolt. A soft, easily drilled layer may also indicate the presence of a fault zone, which could lead to unanticipated rockmass failure. A decelerating drilling rate will occur when harder rock is encountered, but is generally of no concern.

3) *Changes in the appearance of the cuttings.*

The appearance of the cuttings should be uniform for a hole drilled in a single lithology rockmass. The expected rock types along the drill holes should be shown on the *Cablebolt layout section* sheets. Then if the cuttings are different than expected, the drillers should describe the cuttings and take a sample of the cuttings for later inspection. If the cuttings indicate a much softer or weaker zone of rock than was considered during the design of the cablebolt, then the capacity of the cablebolt system will be reduced. It may be important to assess the influence of different lithologies on the cablebolt bond strength.

4) *Holes producing water.*

Excess water in the hole may make grouting difficult, and is likely to mix with the grout, increasing the water:cement ratio (*W:C*) above design levels. Assess the possible increase in the *W:C* of the installed grout. If the increase in *W:C* is likely to be significant, reduce the amount of water in the mix design to account for the presence of water in the borehole.

Any changes in these items should be recorded on the *Drilling observation report*. The engineer must review the report periodically and determine if any of the observations recorded indicate unexpected rockmass failure modes, or problems with the cablebolt installation or with the expected cable performance.

A discussion of the quality control problems that can be encountered when preparing the borehole is given in the *Cablebolt borehole preparation quality control guidelines*. There may be several additional quality control issues that should be added to the guidelines produced for each site. It is important that the drilling crew review these guidelines frequently until they are fully aware of the contents, so that they will be able to assess the influence of problems and understand the importance of reporting on quality control problems.

3.8.1 Design specifications

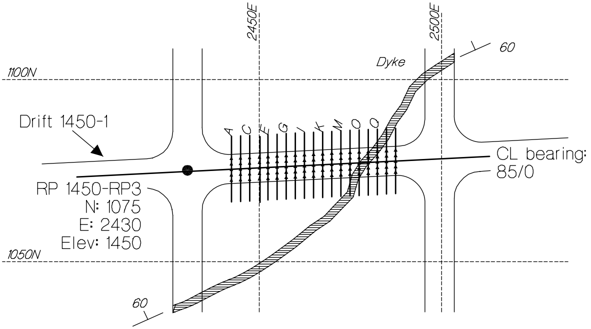
Cablebolt layout: PLAN (Example)

Plan #: *1450/-1* *"Uphole Cablebolts"*

Stope #: *1400/C* Cablebolt ring #s: *A to S*

Reference point (RP): #: *1450 - RP3* Location: N: *1078* E: *2430* Elev.: *1450*

Reference line: Azimuth: *85/0*



Ring spacing: 2.5 m
Cablebolt spacing within ring: @collar - 1.5 m: @ toe - 2.5 m
Cablebolt length: 10 m

Cablebolt ring	Specified Distance: RP - ring	Feedback				
		Surveying *			Drilling	
		Distance: RP - ring	Date	Crew Leader	Distance: RP - ring	Crew Leader
A	*12.5*					
B	*15*					
C	*17.5*					
D	*20*					
E	*22.5*					
....						

** Mark the reference line and ring position on the back or walls of the drift.*

Design Specifications: 3.8.1 continued

Cablebolt layout: SECTION (Example)

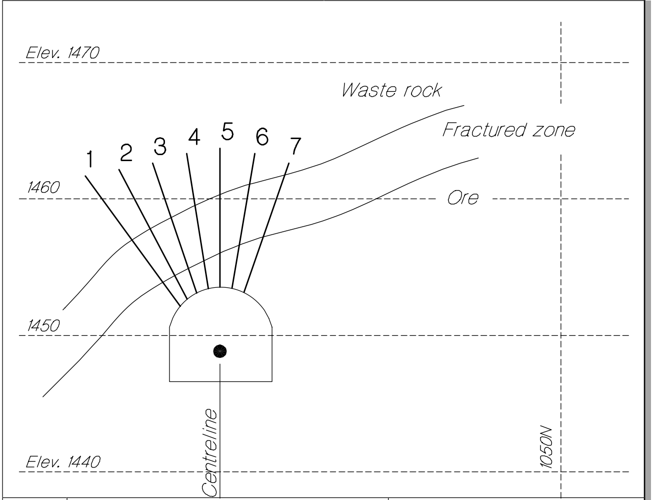
Stope #: *1400/C*

Plan #: *1450/-1*

Cablebolt ring: *A*

Drilling crew leader:

Date holes drilled:



Cable-bolt #	Specification:					Feedback:			
	Dist. from ℄ (m)	Hole Diam (mm)	Dip	Dump	Length (m)	Dist. from ℄	Dip	Dump	Length
1	*-2.8*	*54*	*-36*	*0*	*11*				
2	*-2.3*	"	*-27*	"	"				
3	*-1.6*	"	*-18*	"	"				
4	*-0.9*	"	*-9*	"	"				
5	*0*	"	*0*	"	"				
6	*+0.9*	"	*+9*	"	"				
7	*+1.9*	"	*+18*	"	"				

3.8.2 Procedure and Safety

Procedure: Borehole drilling

Procedure:

1. Make the working area safe by scaling down any loose rock. Clear the floor of the working area of any debris to allow for good footing.
2. Collar each hole as close as possible to the location specified on the *Cablebolt layout plan and sections*. Definitions of the terms "Dip" and "Dump" are given at the bottom of this sheet. If there are hole obstructions, remove them; for example trim the screen, scale the loose or move the pipes. Don't drill through any steel hardware, such as straps or plates.
3. Select the drill bit for the hole diameter specified. Make sure it is not too worn.
4. Drill each hole at the angle and to the length specified, keeping hole wander to a minimum.
5. Sample the drill cuttings for the holes as specified, and at any time that the cuttings look different than expected.
6. Blow the borehole clean, with compressed air, as soon as you have finished drilling. If necessary, flush the hole with water to ensure that it is clean.
7. Block the collar of any downholes against the inflow of mud or dirt.

Allowable deviations and Feedback:

If there are problems, and it is impossible to drill the holes as specified:

- the position of the collar may be moved by up to *100 mm*,
- the borehole angle can deviate by *± 2.5* °, and
- the length of the borehole can be exceeded by *1 metre*, but can't be too short.

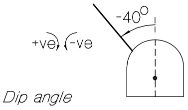
Record any changes to the layout on the *Cablebolt layout plan and sections.*

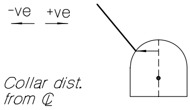
Observations and Feedback:

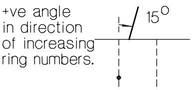
If you observe any of the following changes while drilling the boreholes, please note them on the *Drilling observation report*, indicating the position of the changed zone:

- Loss of drilling water.
- Drilling rate faster or slower than usual - estimate the difference in the rate.
- Hole cuttings different than usual - describe the appearance of the cuttings.
- Holes producing water - estimate the volume of water in litres / second.

Definitions:







3.8.3 Quality Control

Quality control guidelines: Cablebolt borehole preparation

Quality control		Consequences of poor quality control	How to achieve good quality control
Good	Poor		
Hole size correct	Hole size too large or too small	If the borehole is too large, additional grout will have to be pumped into the hole, and the spacers and hanger will be too small. If the hole is too small, it will be difficult to insert the cablebolt, spacers and hanger into the borehole.	Drill the borehole to the specified diameter, and ensure that the bits in use are not too worn.
Correct hole length	Hole too long or too short	If the borehole is too long, then extra grout will have to be pumped into the hole. If it is too short, then the full length of the cablebolt will not fit into the hole. This could result in flared sections of the cablebolt being left hanging out of the collar of the borehole creating plating problems.	Drill the borehole to the specified length. Report any boreholes that are drilled to a length different from that specified.
Straight drill hole	Hogged drill hole	The cablebolt will be in contact with the borehole wall in some places over the length, and so the circumference will not be completely embedded in grout. This may result in reduced capacity of the cablebolt. The toe end of the cablebolts will not be in the designed positions, which may lead to failure of the cablebolt system.	Drill the borehole as straight as possible.
Borehole flushed clean	Borehole not clean	Any mud or cuttings remaining in the borehole will reduce the length or diameter of the grouted column. Any water left in the hole prior to grouting is likely to dilute the grout. Any oil left in the hole will reduce the cablebolt bond strength.	Flush the borehole clean with water and keep it clean by plugging or capping the collar of downholes until ready to install the cablebolts and grout the borehole.

3.8.4 Feedback

Feedback: Drilling observation report

Plan #:

Stope #: | Cablebolt ring #s:

Date: | Drilling crew Leader:

Cable ring #	Cable hole #	Drilling water loss	Change in drilling rate and/or Change in appearance of cuttings: Distance from collar (m) & Description		Borehole water
		@ (m)	@ (m)	Description	(l/min)

Drilling rate: Estimate the change: e.g. 2+ = 2 x faster, 2- = 2 x slower
Appearance of cuttings: Describe the cuttings.

Any changes to the borehole layout must be recorded on the *Cablebolt layout plan and section* specification sheets.

3.9 Cablebolt Installation

The sheets required to fully communicate the cablebolt design are:

Design specification:	Cablebolt layout plan and sections. Cablebolt installation layout specifications and notes.
Procedure and safety:	Cablebolt installation procedure, including Cablebolt placement, Grout mixing, Grout pumping, and Surface fixture installation.
Quality control guidelines:	Cablebolt placement. Grout mixing. Grout pumping. Surface fixture installation.
Feedback:	Cablebolt installation observation report.

A sample version of each of these *Cablebolt installation layout* sheets is given in the following text. These sheets are samples only, and should be modified to suit the particular cablebolt application, specific drilling equipment in use, grouting method selected, safety concerns and allowable deviation tolerances at your mine. The *Cablebolt installation layout specification* sheet provides all of the information required to implement the cablebolt design in the pre-drilled boreholes. A layout sheet should be prepared for each ring of cablebolts to allow the crew to record feedback about each cablebolt installation. At some sites, several versions of these sheets will be required to cover all possible combinations of cablebolt application and installation procedure. In addition, all of the columns on the sheet may not be required, for example, where only one type and length of cable are used these sheets could be simplified. Wherever possible, the safety guidelines listed in Section 3.6.1 have been repeated here to emphasize their importance on a daily basis to the crews.

The installation instructions are detailed for the crew on the *Cablebolt installation procedure* sheet, which includes information about the cablebolt placement, grout mixing, grout pumping and surface fixture installation.

Quality control problems that could be encountered during the installation of the cablebolts and their solutions are listed in a set of guidelines for each of the steps in the installation process in Section 3.9.3. These guidelines are by no means complete, and should be added to if quality control problems arise at your site.

Feedback on the installation process is made by the crew on the *Cablebolt installation layout* sheet and on the *Cablebolt installation observation report.*

3.9.1 Design Specifications

Specification: Cablebolt installation layout *Example* (See attached NOTES)

Plan #: *1450/-1*

Stope #: *1400/C*

Ring #: *A*

✔ See attached *Cablebolt drilling observation* report.

Installation method:

A .... Installation method, BT: Collar finishing, GF; *W:C* = 0 4; Additives, None.

Cablebolt #	As drilled		Design specification				Feedback: As installed						
	Collar dist. from ℄	Drilled hole length	Cablebolt length	Cablebolt type	Installation method	Surface fixture type	Cablebolt length	Cablebolt type	Installation method	Surface fixture type	Grout batch #	Grout flow	Comment #
1	*-2.8*	*11*	*10*	*PS*	*A*	*Pl*							
2	*-2.3*	*11*	"	"	"	"							
3	*-1.6*	*11*	"	"	"	"							
4	*-0.8*	*11*	"	"	"	"							
5	*0*	*11*	"	"	"	"							
6	*0.9*	*10*	"	"	"	"							
7	*1.9*	*11*	"	"	"	"							
...													
...													

▨ A shaded box indicates an acceptable change from the original design.

Date cablebolts placed: Crew Leader:

Date cablebolts grouted: Crew Leader:

Date surface fixtures installed: Crew Leader:

See the attached *Installation layout notes* for additional information about this sheet.

In the feedback section of this sheet, please indicate where the installation followed design with a check mark, ✓, or record the change(s) made. Any comments about or problems with the equipment or procedure, as well as feedback on the grouting procedure must be noted on the *Cablebolt installation observation report*.

Design Specifications: 3.9.1 continued

Specification: Cablebolt installation layout NOTES		
Collar dist. from ℄		The hole collar distance from the ℄ is measured horizontally at the drift back and along the sidewall.
Cablebolt type	PS	Plain 15.2 mm diameter strand.
	TS	Twin or double 15.2 mm diameter strand.
	TS+Sp	Twin 15.2 mm diameter strand with 56 mm diameter spacers placed every 1 metre along the length of the cablebolts.
	BC	Single birdcaged strand.
	TBC	Twin birdcaged strand: two BC strands wired together.
	DBC	14 wire birdcaged strand.
	BA	*25* mm diameter bulbed or nutcaged anchor formed from 15.2 mm diameter strand.
	TBA	Twin bulbed anchor: two BA strands offset and wired together.
Installation method	BT	Uphole, breather tube installation.
	UGT	Uphole, grout tube installation
	DGT	Downhole, grout tube installation
Collar finishing	F	Fabric collar plug: burlap, cotton waste, shredded rag.
	GF	Grout soaked fabric collar plug: burlap, cotton waste, rag.
	GP	Grout collar plug.
	WW	Wooden wedge.
	EF	Expansive foam collar plug.
	RP	Rubber plug.
	VP	Victaulic pipe plug.
	RSP	Resin collar plug.
Surface fixture type	Pl	Plate of *x* by *y* surface dimensions and *z* thickness.
	BPl	Butterfly plate of *x* by *y* surface dimensions and *z* thickness.
	DPl	Domed plate of *x* by *y* surface dimensions and *z* thickness.
	Str	Strapping of *x* by *y* surface dimensions, *z* thickness, *s* spacing between holes, and *d* hole diameter.
Grout batch #		A record of the batch mixing details must be kept on the *Cablebolt installation observation report* sheet. The grout batch # must be recorded on the *report* sheet and the *layout specification* sheet.
Grout flow observation		*BT installations*: Indicate the holes for which there was grout flow out of the end of the breather tubes with a check mark, ✓. *GT installations*: Record the time between the appearance of watery grout at the collar of the hole and of grout of design consistency, (min). Indicate any holes for which grouting was incomplete with a cross, ✗.
Comment #		Any comments about the performance of equipment, materials or procedures should be noted on the *Cablebolt installation observation report* sheet. Cross-reference to the *layout* sheet with a comment #.

3.9.2 Procedure and Safety

Procedure: Cablebolt placement			
1	CB-A: Prepare the borehole.	A1	Upholes.
		A2	Downholes.
2	CB-B: Prepare the cablebolt strand.	B1	Cablebolt strand in uncut coils on a palette.
		B2	Pre-assembled, pre-cut cablebolts in coils on a palette.
		B3	Pre-assembled, pre-cut lengths of cablebolts.
		B4	Cablebolt strand in uncut coils in a reel or dispenser to be inserted directly into the borehole, prior to cutting the strand.
		B5	Cablebolt strand in uncut coils in a reel or dispenser to be cut into lengths, prior to insertion in the hole.
3	CB-C: Assemble the cablebolt element.	C1	Twin strand cablebolts with spacers.
		C2	Twin strand cablebolts without spacers.
		C3	Twin strand flared cablebolts.
4	CB-D: Attach the hanger	D1	Spring steel hanger.
		D2	Pre-attached nut for spring steel hanger.
		D3	Bent wire hanger at the toe of the hole.
		D4	Bent wire hanger at the collar of the hole.
		D5	Pre-attached external fish hook hanger.
		D6	No hanger required.
5	CB-E: Prepare and attach the tube(s).	E1	Grout tube to toe of hole; to be left in the hole during and after grouting.
		E2	Grout tube to toe of hole; to be retracted during grouting.
		E3	Breather tube to toe of hole, and short length of grout tube in collar of hole.
6	CB-F: Place the cablebolt element.	F1	Upholes with breather tubes.
		F2	Upholes or downholes with grout tubes; Grout tube to be left in the hole during and after grouting.
		F3	Upholes or downholes with grout tubes; Grout tube to be inserted with the cablebolt element, and to be retracted from the borehole during grouting.
		F4	Upholes or downholes with grout tubes; Grout tube to be inserted into each hole just prior to grouting, and to be retracted from the borehole during grouting.

Procedure and Safety: 3.9.2 continued

Procedure: Cablebolt placement continued

7	CB-G: Finish the borehole collar.	G1	Cotton waste or dry burlap collar plug.
		G2	Grouted burlap collar plug.
		G3	Grout collar plug.
		G4	Wooden wedge.
		G5	Rubber collar plug.
		G6	Resin collar plug.
		G7	Victaulic pipe collar plug.
		G8	Expansive foam collar plug.
		G9	No borehole collar finishing required.

Feedback:

- Any changes to the cablebolt length or type must be recorded on the *Cablebolt installation layout specification* sheet.
- Any comments about the installation procedure, materials or equipment should be recorded on the *Cablebolt installation observation report* sheet, and cross referenced to the *Cablebolt installation layout specification* sheet.

Procedure: Grout mixing and pumping

1	CB-H: Mix the grout.	H1	Paddle mixer, Drum mixer or Colloidal mixer.
2	CB-I: Clean the grout mixer.	I1	Paddle mixer, Drum mixer or Colloidal mixer.
3	CB-J: Pump the grout.	J1	Breather tube installation method.
		J2	Grout tube installation method.
		J3	Breather tube installation method in fractured rockmass: fractured zone extent is known in advance.
		J4	Grout tube installation method in fractured rockmass: fractured zone expected during grout pumping.
		J5	Grout tube installation method in fractured rockmass: fractured zone extent is known in advance.
4	CB-K: Clean the grout pump.	K1	Cleaning all grout pumps.
		K2	Piston pump.
		K3	Progressing cavity pump.

Procedure and Safety: 3.9.2 continued

Procedure: Grout mixing and pumping continued

Feedback:

- Any changes to the installation method must be recorded on the *Cablebolt installation layout* sheet.
- The grout batch # and observations of the grout flow from the breather tube or collar of the hole, as discussed on the *Cablebolt installation layout Notes* sheet, must be recorded for each cablebolt on the *Cablebolt installation layout* sheet.
- Information about each grout batch must be recorded on the *Installation observation report* sheet.
- Any comments about the installation procedure, materials or equipment should be recorded on the *Cablebolt installation observation report* sheet, and cross referenced to the *Cablebolt installation layout* sheets.

Procedure: Surface fixture installation

1	CB-L: Fixture installation.	L1	Plain plates, Domed plates, Butterfly plates, Straps.

Feedback:

- Record any changes to the surface fixture type on the *Installation layout* sheet.
- Any comments about the installation procedure, materials or equipment should be recorded on the *Cablebolt installation observation report* sheet, and cross referenced to the *Cablebolt installation layout* sheet.

To create a cablebolt placement procedure for the specific conditions at the site, determine which of the separate procedures listed in the preceding tables and in the following text are required. Most of the procedures will likely require some edits and additions to accurately reflect the conditions at the site, and text shown in italics within the sample procedures should be checked to ensure that it is accurate for the equipment and conditions at the site.

As an example, procedures A1, B1, D1, E3, F1 and G5 could be used to specify the installation of a single plain strand cablebolt to be installed in an uphole. Grouting procedures could be created from H1, I1, J1, K1 and K3.

The list of the cablebolt placement options given here may seem extensive, but is most probably not even complete. The authors observed a great deal of innovation with cablebolting procedures at the mine sites visited during the course of the project. In a number of cases, a new technique had taken quite some time and trial and error to develop, however the final product worked well enough to justify the expenditure. Often development of the technique was necessitated by failure or lack of efficiency of the old method.

CB-A: Borehole Preparation

A1: Upholes.

1) Make the working area safe by scaling down any loose rock, following safe scaling practice.
2) Clear all debris from the floor of the working area.
3) Measure the length of the boreholes. Make a note of any holes that are longer or shorter than the specified length on the *Cablebolt installation layout* sheet. (If the specific hole length is critical for the design, specify a hole length deviation that should not be exceeded. For example, if a 10 m hole is < 8 m in length, in a specific design this cablebolt might be useless, and therefore not worth installing in the short hole. In this case the cablebolt should be installed at a later time in a new hole drilled to the correct length. To assess the hole length allowable deviation, consider the effect of the short cablebolt on the performance of the cablebolt system, the likelihood of the hole being redrilled, the effect of a shorter hole on the effectiveness of modified geometry cablebolts, and any other factors relevant to your site.)
4) Make a note on the *Cablebolt installation observation report* sheet of any boreholes which are producing water.

A2: Downholes.

1) Make the working area safe by scaling down any loose rock, following safe scaling practice. Clear all debris from the floor of the working area.
2) Clean the floor of the drift around the cablebolt holes of any debris or dirt that could fall into the borehole during the installation process.
3) Blow the boreholes clean and dry. If the holes are very dirty, fill them with water and blow them dry again.
4) Measure the length of the boreholes. If a hole is greater than *1 metre* too long, the cablebolt could support the ore as well as the waste rock. In this case, mark the hole as bad, make a note on the *Cablebolt installation layout* sheet and do not install the cablebolt. (If a too short cablebolt will greatly reduce the effectiveness of the cablebolt pattern, specify a hole length deviation that should not be exceeded. To assess the allowable hole length deviation that is specified here, consider the effect of the short cablebolt on the performance of the cablebolt system, the likelihood of the hole being redrilled, the effect of a shorter hole on the effectiveness of modified geometry cablebolts, and any other factors relevant to the site.)

CB-B: Cablebolt Strand Preparation

B1: Cablebolt strand in uncut coils on a palette.

1) Place the coil of cablebolts in a clean, open working area away from other people and equipment.
2) Stand in the middle of the coil, and cut the binding straps with an *air powered cutter* in the correct order so that the coil of cablebolts will unroll in a controlled manner. Ensure that no one is in the range of any springing or lashing cablebolts.
3) Cut the length of cablebolts detailed on the *Cablebolt installation layout specification* sheet, using an *abrasive cutter*. Wear a face shield and leather gloves when cutting cablebolts, since small shards of metal and sparks are created. Lay the cablebolts out on a clean, dry surface, such as a *wooden platform*, a *row of PVC tubes*, or a *series of saw horses*. Where plates or other surface fixtures are to be used on modified geometry cablebolt elements, cut the strands so that the last *1 m* at the collar end of the strand is straight.
4) If the cablebolts are dirty or rusty, clean them with a *wire brush* or a *pressurized water jet*. If it impossible to clean them, reject them.

B2: Pre-assembled, pre-cut cablebolts in coils on a palette.

1) Place the coil of cablebolts in a clean, open working area away from other people and equipment.
2) Stand in the middle of the coil, and cut the binding straps *with an air powered cutter* in the correct order so that the coil of cablebolts will unroll in a controlled manner. Ensure that no one is in the range of any springing or lashing cablebolts.
3) Separate apart the individual pre-assembled cablebolts, and lay them out on a clean, dry surface, such as a *wooden platform*, a *row of PVC tubes*, or a *series of saw horses*. Ensure that all components of the pre-assembled element remain intact and are clean.
4) If any of the cablebolts are dirty or rusty, clean them with a *wire brush* or a *high pressure water jet*. If it is not possible to clean them, reject them.
5) Where pre-assembled modified geometry elements are to have surface fixtures attached after grouting, make sure that the end of the cablebolt element with the straight section of strand will be used at the hole collar.
6) When hangers are included with the pre-assembled cablebolt elements, lay the cablebolt out with the hanger closest to the drill hole collar.

CB-B: Cablebolt Strand Preparation continued

B3: Pre-assembled, pre-cut lengths of cablebolts.

1) Separate apart the individual pre-assembled cablebolts, and lay them out on a clean, dry surface, such as a *wooden platform*, a *row of PVC tubes*, or a *series of saw horses*. Ensure that all components of the pre-assembled element remain intact and clean.
2) Where pre-assembled flared cablebolt elements are to have surface fixtures attached after grouting, ensure that the end of the cablebolt element with the straight section(s) of strand is furthest away from the borehole collar.
3) When hangers are included with the pre-assembled cablebolt elements, lay the cablebolts out with the hanger end nearest the collar of the borehole.
4) If the cablebolts are dirty or rusty, clean them with a *wire brush* or a *high pressure water jet*. If it is not possible to clean the cablebolts, reject them.

B4: Cablebolt strand in uncut coils in a reel or dispenser to be inserted directly into the borehole, prior to cutting.

1) Place the reel or dispenser in a clean area as near to the working face as possible. Ensure that the cablebolt strand will not drag on the floor while being dispensed.
2) If the strand within the coil is rusty, check with the engineer to see whether the surface can be cleaned, or whether the whole coil should be rejected.
3) After the hanger and tubes have been attached to the cablebolt, and the cablebolt has been inserted into the hole (F), cut off the end of the steel strand using an *abrasive cutter*. Wear a face shield and leather gloves.

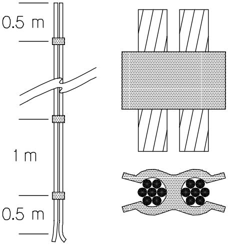
B5: Cablebolt strand in uncut coils in a reel or dispenser to be cut into lengths, prior to insertion in the hole.

1) Place the reel or dispenser in a clean area as near to the working area as possible. Ensure that the strand will not drag on the floor while being dispensed.
2) If the strand within the coil is rusty, check with the engineer to see whether the surface can be cleaned, or whether the whole coil should be rejected.
3) Pull the required length of cablebolt from the dispenser onto a clean, dry platform, and cut using a *tungsten grinder blade*. Wear a face shield and leather gloves when cutting cablebolts, since small shards of metal and sparks are created. If plates or other surface fixtures are to be used on modified geometry cablebolt strand, cut the strands so that the last *1 m* of the strand at the collar end of the strand is straight.

CB-C: Cablebolt Element Assembly

C1: Twin strand cablebolts with spacers.

1) Place two individual lengths of strand on a clean, dry working space.
2) Insert the cablebolt strands into the *56 mm diameter green plastic double* spacers. The first spacer should be placed at *0.5 m* from the toe end of the cablebolts. The rest of the spacers should be placed every *1 m* along the length of the cablebolts. The last spacer should be placed *0.5 m* from the collar position.

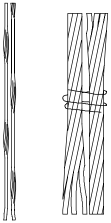


C2: Twin strand cablebolts without spacers.

1) Place two individual lengths of strand on a clean, dry working space.
2) For twin strands to be installed in upholes, place the cablebolts so that the ends of the separate strands are offset slightly. The protruding strand should be long enough to accept the hanger.
3) Wire the toe end of the cablebolts together *in two places.*

C3: Twin strand flared cablebolts.

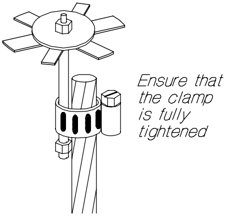
1) Place two individual lengths of strand on a clean, dry working space.
2) Position the individual strands so that the flared sections are offset at even spacings along the length of the cablebolt element.
3) Wire the toe end of the cablebolts together, leaving enough space for the hanger. Tie the strands together with wire every *2 metres*.



CB-D: Attachment of Hanger

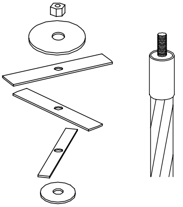
D1: Spring steel hanger.

1) Wear leather gloves as the sharp steel strips can cut your hands.
2) Ensure that the spring steel strips are evenly spaced around the outside of the hanger, and that the nuts are tight.
3) Attach the spring steel hanger to the end of the cablebolt using a *steel band hose clamp*, so that the spring steel strips on the top of the hanger are above the end of the cablebolt.



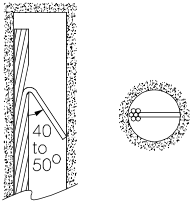
D2: Pre-attached bolt for spring steel hanger.

1) Wear leather gloves as the sharp steel strips can cut your hands.
2) Remove the nut from the bolt.
3) Slide *3 7.5 cm* by *2 cm* steel strips over the end of the bolt, spacing the strips evenly around the outside of the hanger.
4) Place the washer over the strips.
5) Screw the nut back onto the end of the bolt and tighten.



D3: Bent wire hanger at the toe of the hole.

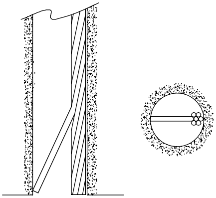
1) Working at the end of the cablebolt nearest the hole collar, bend over a *75 mm* length of *one* wire so that it forms a hook, using the *bending tool*. The hook should point towards the collar end of the cablebolt and be at a *40 °* to *50 °* angle away from the cablebolt strand.
2) The hook must be long enough to grip the borehole wall.



CB-D: Attachment of Hanger continued

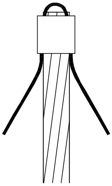
D4: ***Bent wire hanger at the collar of the hole. (This type of hanger is not applicable when surface fixtures are to be used, and may not be very effective in certain circumstances.)***

1) Working at the end of the cablebolt furthest from the hole collar, bend *75 mm* of *one* wire of the end of the cablebolt away from the strand, using the *bending tool*. The hook should point towards the collar end of the cablebolt and be between *40 °* and *50 °* away from the cablebolt element.
2) The hook must be long enough to grip the borehole wall when it is inserted.



D5: ***Pre-attached external fish hook hanger.***

1) Using the *bending tool*, bend out the wires of the pre-attached hanger. The bent wire should be between *40 °* and *50 °* from the cablebolt.
2) The wire(s) must be long enough to grip the borehole wall. If the wire(s) are too long, they may not be at a great enough angle from the cablebolt to grip the hole well.



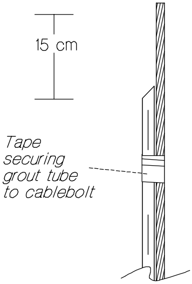
D6: ***No hanger required.***

The hanger must be strong enough to support the full weight of the ungrouted cablebolt, and be long enough to grip the borehole wall securely. If the borehole diameter has changed from design, or if the bent wires are shorter or longer than specified, the hanger may fail to support the cablebolt, leading to very dangerous working conditions. The selection of adequate hangers is discussed in greater detail in Section 2.11.4.

CB-E: Tube Preparation and Attachment

E1: Grout tube to toe of hole: to be left in the hole during and after grouting.

1) Cut the end of the *19 mm I.D.* grout tube at a 45° angle, using a *hack saw*.
2) Cut *two 10 cm* long slots into the side of the grout tube. This is especially important for downhole cablebolts where the end of the tube can become easily plugged with cuttings.
3) Position the grout tube beneath the arm of the hanger for protection. Using *duct tape*, attach the grout tube to the cablebolt element, so that the end of the grout tube is *15 cm* to *30 cm* from the toe end of the cablebolt.
4) Straighten the tubes so there are no kinks or twists in the plastic.

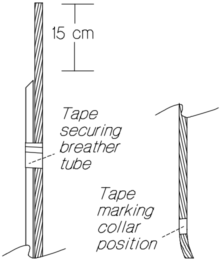


E2: Grout tube to toe of hole: to be retracted during grouting.

1) Cut the end of the *19 mm I.D.* grout tube at a 45° angle, using a *hack saw*.

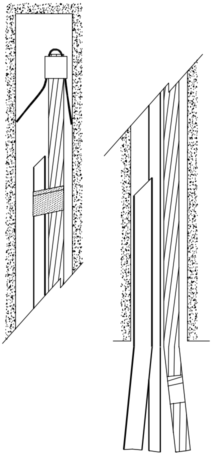
E3: Breather tube to toe of hole, and short length of grout tube at the collar of the hole.

1) Cut the end of the *10 mm I.D.* breather tube and the *19 mm I.D.* grout tube at a 45° angle, using a *hack saw*.
2) Position the breather tube beneath the arm of the hanger for protection. Using *duct tape*, attach the breather tube to the cablebolt element, so that the end of the breather tube is *15 cm* below the toe end of the bolt.
3) Mark the collar position on the cablebolt. When surface fixtures are to be used, *1 metre* of one strand should be below this mark.
4) Straighten the tubes so that there are no kinks or twists in the plastic.



CB-F: Cablebolt Placement

F1: Upholes with breather tubes.



1) Remove any obstructions or loose near the borehole collar.
2) Insert the assembled cablebolt element with attached breather tube into the hole. Adopt "safe lifting" practices when pushing the cablebolts into the hole, and don't overexert yourself. One crew member must guide the free end of the cablebolt. While pushing the cablebolt into the hole, ensure that the breather tube is not crushed, kinked or twisted. Stop pushing the cablebolt when the marked collar position is still 1 m away from the rock face.
3) If it is impossible to insert the cablebolt into the hole, push a grout tube up the hole and flush the hole with water. If the hole is still obstructed, redrill it, and then place the cablebolt.
4) Insert the grout tube a few cms into the collar of the hole. Tape the grout tube to the cablebolt element, leaving the breather tube free.
5) Pull on the cablebolt to check that it is securely anchored.
6) Flush water through the breather tube to clean the borehole and cablebolt.
7) Place a plastic cap on the end of the cablebolt.

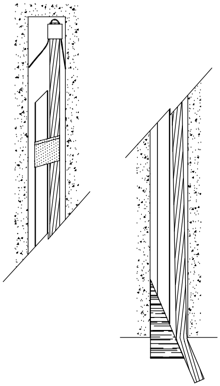
8A) If cotton waste or burlap is to be used to seal the collar of the hole, leave the cablebolt protruding from the hole, and proceed to CB-G: the *Borehole collar finishing* instructions.

8B) If cement or expansive foam collar packing are to be used to seal the collar, push the cablebolt the rest of the way into the hole.

9) Cut off the protruding end of the breather tube, leaving enough length of tube outside the hole to allow easy observation of the return of the grout, and to tie off the tube after grouting is finished.
10) Cut off the protruding end of the grout tube, leaving enough length of tube outside the hole to tie off the tube after grouting is finished.
11) Roll up the tubing that is left protruding from the hole so that it is out of the way until you start grouting.

CB-F: Cablebolt Placement continued

F2: Upholes with grout tubes: Grout tube to be left in the borehole during and after grouting.



1) Remove any obstructions or loose rock from the borehole collar.
2) Insert the assembled cablebolt element with grout tube into the hole. Adopt "safe lifting" practices when pushing the cablebolt into the hole. Someone must guide the end of the cablebolt, so that it does not whip around in the air. While pushing the cablebolt into the hole, ensure that the grout tube is not crushed or kinked. Try to keep the grout tube straight, as twists and bends in the tube will create points where grout pressure builds up.
3) Cut off the protruding end of the grout tube, leaving enough length of tube outside the hole to be attached to the grout pump hose, and to tie off the tube when grouting is finished. Roll the end of the tube up and secure it out of the way.
4) Pull on the cablebolt to ensure that it is securely anchored.
5) Place a plastic cap on the end of the cablebolt. Then if the cablebolt does fall from the hole, there will be less chance of damage.

F3: Upholes or downholes with grout tubes: Grout tube to be inserted with the cablebolt element and to be retracted from the borehole during grouting.

1) Remove any loose or obstructions from around the borehole collar.
2) Holding the end of the grout tube at the end of the cablebolt element, insert both into the hole. Adopt "safe lifting" practices when pushing the cablebolts into the hole. Ensure that the reel of grout tubing is able to unroll easily, and the tubing will not kink or be crushed during the cablebolt placement. Keep the grout tube as straight as possible and prevent it from twisting as it enters the hole. Roll up the rest of the tube and secure it out of the way.
3) Pull on the cablebolt to make sure that it is securely anchored.

CB-F: Cablebolt Placement continued

F4: Upholes or downholes with grout tubes: Grout tube to be inserted in each hole just prior to grouting and to be retracted from the borehole during grouting.

1) Remove any loose or obstructions from around the borehole collar.
2) Insert the cablebolt element into the hole. Adopt "safe lifting" practices when pushing the cablebolts into the hole. Someone must guide the end of the cablebolt, so that it does not whip around in the air.
3) Pull on the cablebolt to check that it is securely anchored.
4) The grout tube will be inserted into the hole during the grouting procedure. However, after the first couple of cablebolts have been placed in a working area, attempt to insert the grout tube into the boreholes. If there are problems with inserting the grout tube into the hole, stop installing the cablebolts in that area, and inform the underground supervisor or engineer about the problem.

As is mentioned in all of these cablebolt placement procedures, it is very important that the grout and/or breather tubes are not damaged during insertion of the element into the borehole. Damage is most likely to occur when thin walled, weak tubing is used. It is worth spending a few extra cents for each unit length of tubing to reduce the chances of damaging the tube. Damaged tubing may not transmit the grout during pumping, thereby potentially rendering the cablebolt installation in that borehole useless.

Some mine cablebolting crews observed by the authors had devised reels for dispensing the tubing, for much increased ease of handling. At these sites, one of the crew members inserted the steel strand into the borehole, while another member guided the tubing into the hole, keeping it straight and free of twists and kinks.

CB-G: Borehole Collar Finishing

G1: Cotton waste or dry burlap collar plug.

1) Wrap cotton waste or burlap around and between the cablebolt, grout tube and breather tube, above the collar position of the cablebolt element. *20 cm* of the cablebolt above the collar position mark should be fabric covered.
2) Push the cablebolt the rest of the way into the hole, tamping the material into the borehole collar tightly, while keeping the cablebolt(s) in the centre of the hole. Double cablebolts that will both be plated or strapped, must be far enough apart so that a barrel can fit over each of them.
3) Cut off the protruding end of the breather tube, leaving enough length of tube outside the hole to allow easy observation of the return of the grout down the tube, and to tie off the tube when grouting is finished.
4) Cut off the protruding end of the grout tube, leaving enough tube outside the hole to be attached to the grout hose, and to tie off the tube after grouting.
5) Blow on the end of the grout tube. You should feel air coming out of the end of the breather tube. If you do not feel air return, push some more fabric into the collar of the hole. If this doesn't lead to air return, the rock around the collar or along the borehole could be fractured. Do not grout the hole, and inform the engineer at the end of the shift.
6) Before grouting, flush water through the breather tube to saturate the fabric.

G2: Grouted burlap collar plug.

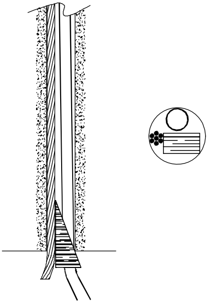
1) Dip the burlap into a bucket of thin cement grout paste. Wear safety glasses, long sleeves and water proof gloves to prevent cement grout burns.
2) Wrap the burlap around and between the cablebolt, grout tube and breather tube at and above the collar position of the cablebolt element. At least a *20 cm* length of the cablebolt above the marked collar position should be covered by the burlap. Double cablebolts that will both be plated or strapped, must be far enough apart so that a barrel can fit over each of them.
3) Push the cablebolt the rest of the way into the hole, tamping the burlap into the borehole collar tightly, keeping the cablebolt centred in the borehole.
4) Cut off the protruding end of the breather tube, leaving enough length of tube outside the hole to allow easy observation of the return of the grout down the tube, and to tie off the tube when grouting is finished.
5) Cut off the protruding end of the grout tube, leaving enough tube outside the hole to be attached to the grout hose, and to tie off the tube after grouting.
6) Blow on the end of the grout tube. You should feel air coming out of the breather tube. If you do not feel air return, push some more material into the collar of the hole. If you still don't get air return, the rock around the borehole could be fractured. Do not grout the hole, and inform the engineer.

CB-G: Borehole Collar Finishing continued

G3: Grout collar plug.

1) Insert a short section of grout tube into the collar of the hole. This section must be shorter than the grout tube that was inserted with the cablebolt element. Mark this grout tube, so that there will be no confusion between the two tubes. Make sure that the distance between the end of the longer grout tube and the collar plug is sufficient to create a large enough grout plug to support the full weight of the grout column.
2) Plug a short section of the collar of the hole with cotton waste or burlap, making sure that the cablebolt remains in the centre of the borehole.
3) Mix 0.40 *W:C* grout following the procedures given in CB-H.
4) Pump the grout into the hole through the shortest length of grout tube, following the procedures given in CB-J. Continue to pump until the grout flows back down the longer grout tube. Stop pumping the grout and kink over and tie off the shortest grout tube. Wash off the end of the longer length of grout tube, so that there will be no problems with attaching the grout pump hose to the tube when you return to grout the borehole.
5) Blow air through the longer grout tube to ensure that it is still open. If it is plugged, pump a small amount of water into the tube to flush it.
6) Ensure that the grout collar plug is placed early enough so that it has cured to an adequate strength to support the grout column when the cablebolt hole is grouted.

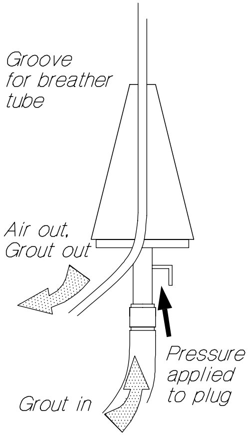
G4: Wooden wedge.



1) Push a wooden wedge into the collar of the hole to secure the cablebolt until ready to grout. The wedge should be placed between the cablebolt and the borehole wall, with the tubes pushed aside.
2) Make sure that the grout and/or breather tube(s) are not crushed or pushed out of round during this operation.
3) Pull on the end of the cablebolt to ensure that the wedge is secure.
4) Take a great deal of care with this operation, especially if no hanger has been used at the toe of the hole to provide added support to the cablebolt element.

CB-G: Borehole Collar Finishing continued

G5: Rubber collar plug (used at Mines Gaspé and Brunswick Mine, Canada). Note: This method must be used with a work platform that can be raised, such as a scissor lift.



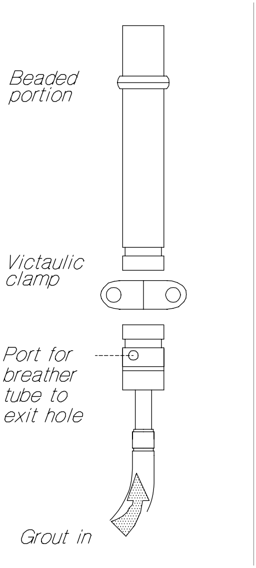
1) Place a wooden wedge in the borehole collar to secure the cablebolt (procedure CB-G4). Remove the wedge when ready for grouting.
2) Remove any obstructions or loose rock from the collar area, making the collar rock as smooth as possible.
3) Place the rubber plug onto the end of the 3/4" diameter pipe or rod that will be used to support it. Hold the end of the pipe or rod vertically with the lower end resting on the scissor lift platform.
4) Raise the platform until the rubber plug is tightly wedged into the hole collar. During this process, feed the breather tube into the slot on the side of the rubber plug.
5) If the rock around the collar of the borehole is rough or jagged, use some waste cotton around the rubber plug to help seal the collar.

G6: Resin collar plug (used at Macassa Mine, Canada).

1) Wear rubber gloves, long sleeved clothing and safety glasses to prevent resin burns. Never stand under the hole.
2) Lay a piece of paper towel or flexible rag out on the work space. Cut open a tube of resin and spread it out on the paper towel, mixing the two components of the resin well.
3) Wrap the resin soaked paper or fabric around the collar position of the cablebolt, and push the cablebolt the rest of the way into the borehole.
4) Push more paper towel or rag into the collar to completely seal the hole. The resin sets very quickly, so that it is possible to grout the hole almost immediately after sealing the collar.

CB-G: Borehole Collar Finishing continued

G7: Victaulic pipe collar plug (used at Campbell Mine, Canada: Bourchier et al, 1992). The cablebolt is inserted into the hole through the pipe collar.



1) Remove any loose or obstructions near the collar of the borehole.
2) Attach the 2" Victaulic clamp to the grooved end of the pipe casing.
3) Cut a 2" slot on the plain end of the pipe casing with a hack saw.
4) Crimp the slotted end of the pipe so that it fits into the hole.
5) Insert the crimped end of the pipe into the borehole and tap with a hammer until the pipe bead is inserted into the hole. Insert the cablebolt into the hole.
6) Remove the Victaulic clamp.
7) Coat the inside of the Victaulic reducer with a thin layer of grease.
8) Push the breather tube through the hole on the inside of the reducer.
9) Slide the reducer along the breather tube, and fasten to the casing using the 2" Victaulic clamp, which has been coated with grease.
10) Tighten the clamp.
11) Coat the outside of the clamp and reducer with grease.
12) 24 hours after grouting the hole, the victaulic clamp and reducer can be removed from the pipe and used again.

G8: Expansive foam collar plug.

1) The foam collar plug should be sprayed into the hole following the manufacturer's instructions. Ensure that the length of the plug is adequate to hold the grout column.
2) Wait at least 24 hours before starting to grout.

G9: No borehole collar finishing required.

CB-H: Grout Mixing

H1: Paddle, Drum and Colloidal mixers.

1) Determine how much water will be needed for the mix dependent upon the size of the cement bags and the grout mix design.

Size of cement bag (kg)	*Volume of water required (litres)*			
	W:C = 0.3	*W:C* = 0.35	*W:C* = 0.4	*W:C* = 0.45
10	3	3.5	4	4.5
25	7.5	8.75	10	11 25
40	12	14	16	18

2) Grout can burn exposed skin severely. Wear glasses, gloves and protective clothing when working with cement powder and grout. Barrier cream should be used on any exposed skin, and on your hands. If any grout does touch bare skin, the area should be washed with soap and water and completely rinsed. If grout gets into your eye, flush with fresh water and report to the first aid station immediately.
3) Wear a dust mask when breaking open bags of cement.
4) Cement should be stored on water proofed, plastic wrapped palettes close to the intended pump set up. You will need about *80 kg* of cement for each *15 m* borehole.
5) Ensure that the mixer and pump are clean with no dried grout lumps.
6) *Provide detailed instructions for assembling the mixer components and connecting the system to the air supply (see manufacturers instructions).*
7) Use *whip-checks* on all air hose connections, and blow the hoses clean before connecting.
8) Check to ensure all valves are off.
9) Turn the air on at the header.
10) Fill the hopper with water. Turn on the mixer to check that it is operating. Flush water through the grout pump. Remove all water from both.
11) Measure and pour the required amount of water for the batch. It is very important to mix the specified water:cement ratio. If the mix is too dry, it will be hard to pump and may lead to pressure build up in the grout tubes and eventually bursting of the grout tubes. If the mix is too wet, the grout strength with be reduced and the grout will be more fluid and will more easily flow out of the hole collar or into rockmass fractures.
12) Turn on the mixer.
13) Where specified, mix the correct volume of the wet additive into the water. Agitate the water and additive slowly to prevent foaming.

CB-H: Grout Mixing continued

14) If the mixer and pump are a single unit, start to recirculate the water through the pump and back into the mixing hopper. Recirculating the grout will assist in complete mixing, and lead to a smoother mix. If the pump doesn't have a recirculating valve, place the outlet end of the grout pump hose into the mixer hopper, keeping it free of the mixer blades, and start the pump.
15) Break each bag of cement powder on a *screen* placed on top of the *hopper* of the mixer. Add the dry powder to the water slowly, and make sure that any lumps of pre-hydrated cement are removed. If there are a lot of lumps, discard the bag, and note the problem on the *Cablebolt installation observation report* sheet.
16) Where specified, add the correct volume of dry additive.
17) Mix the grout completely so that there are no lumps, and the grout has a smooth consistency. There should be no lumps visible on the surface of the mix. Stop the mixer briefly and remove any build up of dry cement or lumps on the paddles or bin walls.
18) Whenever the mixer will be idle for more than *1/2 hour* (lunch time, shift end), clean it following the instructions given in procedure CB-I.

CB-I: Grout Mixer clean up

I1: Paddle, Drum and Colloidal grout mixers.

Mixer clean up is essential for all types of mixers to ensure efficient and clean operation. Detailed instructions for the clean up of the mixer should have been provided by the supplier when the mixer was purchased. *Add these instructions to this procedure.* The time after which the grout will become too difficult to clean will depend on the type of cement and on the additives in use.

Work with the crew to ensure that the instructions given in the procedure result in a completely clean mixer. In general though, it is important to:

1) Remove as much of the grout from the mixer as possible.
2) Fill the mixer hopper with fresh water and operate the mixer.
3) Pour this water out of the hopper.
4) Wash the hopper and paddles with more fresh water.
5) Scrape any accumulations of grout from all exposed surfaces.
6) After cleaning the mixer, fill it with fresh water and leave it full of water until the next time that it is used.

CB-J: Grout Pumping

J1: Breather Tube Installation Method.

1) Grout can burn exposed skin severely. Wear glasses, gloves and protective clothing when working with cement powder and grout. Barrier cream should be used on any exposed skin, and on your hands. If any grout does touch bare skin, wash the area with soap and water and rinse completely. If grout gets into your eye, flush with fresh water, and then report to the first aid station immediately.
2) Set up the grout pump as close as possible to the boreholes to minimize the length of grout tube required. Longer tubes result in greater fluid pressure in the tubes and higher power requirements for the pump.
3) *Add instructions for the regular maintenance of the pump. These details might include such items as lubrication or air filter replacement.*
4A) If you have not been recirculating the grout during the mixing process, pump grout from the grout pump hose onto the floor until it is the same consistency as the grout in the hopper. The first portion of the grout may be watery and must be emptied from the hose before grouting the hole.
4B) If you have been recirculating the grout through the grout pump hose back into the hopper, shut off the grout pump.
4C) If you have been recirculating the grout through a recirculation valve during mixing, turn off the valve, slow the grout pump down, but don't stop it.
5) Clean any grout off the outside of the grout pump hose, and attach the hose to the grout tube using the *quick screw connector* or the *modified vice grips*.
6) Turn the pump on slowly. The pump speed must be fast enough to force the grout through the tube, but not so fast that excess pressure builds up and bursts the plastic tubing, and not so slow that the pump stalls.
7) Place the end of the breather tube in a bucket of water. Air will bubble through the water as it is forced out of the hole by the grout front.
8) Estimate the volume of grout required to fill the hole from Figure 3.9.1.
9) Monitor the level of the grout in the pump hopper. If the grout volume that has been pumped into the hole is more than *twice* what it should be, the rockmass is fractured. Stop grouting, remove the collar plug and disconnect the grout hose from the grout tube to allow the grout to drain from the hole. If you are unable to remove the collar plug, pump water through the grout tube until you see clean water flowing from the end of the breather tube. Wait for at least 24 hours before returning to plug the collar again and grout the hole. Record the problem on the *Cablebolt installation observation report*, and make a note of the grout volume initially pumped into the hole.
10) Try to block any grout leaks out of boreholes or cracks with *foam* or *cotton waste*. Alert the engineer at the end of the shift about the problem.
11) Keep pumping until grout of design consistency flows from the end of the breather tube. Stop the grout pump.

CB-J: Grout Pumping continued

12) Kink over the end of the breather and grout tubes and tie them off.
13) Disconnect the grout tube from the grout pump hose.
14) Start the grout recirculating in the pump if you are not immediately ready to start grouting the next borehole.
15) Use all of each batch before starting to mix more grout. Never mix grout continuously. Do not start to grout a hole if there is not enough grout to fill it. Discard the remaining grout and mix a fresh batch.
16) If there are any delays which prevent the grout from being pumped into the boreholes within *1/2 hour* of the start of the mixing time, pump the grout out onto the floor of the working area away from any downholes, and clean the grout pump (Procedure CB-K). If additives or high early strength grout are being used, the time that the grout can remain in the pump will be less.
17) Whenever the pump will be idle for more than *1/2 hour*, make sure that the pump is completely clean and is left full of clean water.
18) Never stand under newly grouted boreholes. Any water dripping out of the holes can burn skin badly.
19) Where surface fixtures are to be used, wait *24 hours* and then follow the instructions in Procedure CB-L. If surface fixtures are not to be used, trim the ends of all cablebolts that could create a safety hazard.
20) 24 hours or more after grouting, cut off the breather and grout tubes.

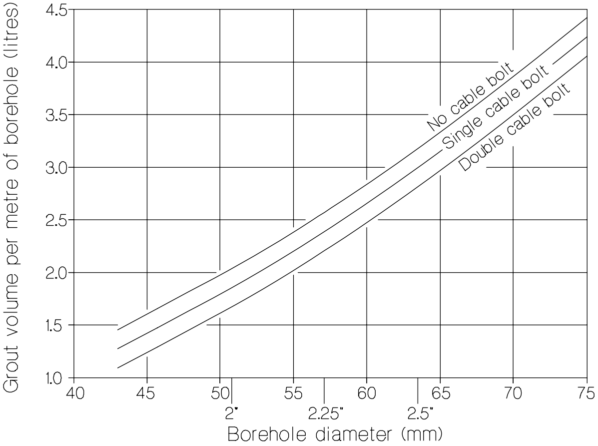


Figure 3.9.1: Volume of grout required per metre of cablebolt.

CB-J: Grout Pumping continued

J2: Grout Tube Installation Method.

1) Grout can burn exposed skin severely. Wear glasses, gloves and protective clothing when working with cement powder and grout. Barrier cream should be used on any exposed skin, and on your hands. If any grout does touch bare skin, the area should be washed with soap and water and completely rinsed. If grout gets into your eye, flush with fresh water, and then report to the first aid station immediately.

2) Set up the grout pump as close as possible to the boreholes to minimize the length of grout tube required. Longer tubes result in greater fluid pressure in the tubes and higher power requirements for the pump

3) Estimate the volume of grout required for the borehole from Figure 3.9.1.

4A) If you have not been recirculating the grout during the mixing process, pump grout from the grout pump hose until it is the same consistency as the grout in the hopper. The first portion of the grout may be watery and must be emptied from the hose before grouting of the hole starts.

4B) If you have been recirculating the grout through the grout pump hose back into the hopper, shut off the grout pump.

4C) If you have been recirculating the grout through a recirculation valve during mixing, shut off the valve, and slow the grout pump down, but don't stop it.

5) Clean any grout off the outside of the grout pump hose, and attach the hose to the grout tube using the *quick connector* or *modified vice grips*.

6) Start pumping the grout slowly. The pump speed must be fast enough to force the grout through the tube, without stalling, but not so fast that excess pressure builds up and bursts the plastic tubing.

7) Monitor the level of grout in the hopper. It is very important that a progressing cavity pump is never operated dry for more than *15 seconds*. If the grout level stops dropping in the hopper while pumping, stop the pump. Remove the grout hose from the grout tube and try to restore normal operation. If the grout has become too stiff to flow into the pump, add water to the hopper and pump the waste cement out onto the floor. If the pump stalls, and it is impossible to finish grouting the hole, estimate how much grout has already been pumped into the borehole and make a note. Stop the pump and detach the grout pump hose from the grout tube. Try to pump grout through the hose. If the hose is jammed, stop pumping the grout, and try to determine where the blockage has occurred. Manipulate the grout pump hose in the area of the blockage to try to free it. If all else fails, discard the hose and start with a new hose. If grout flows freely from the pump hose, it is likely that the grout "froze" in the hole due to the presence of fractured rock. Follow the instructions for grouting in fractured rock given in Procedure CB-J4.

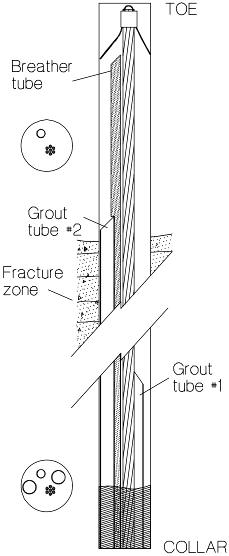
CB-J: Grout Pumping continued

8) Keep pumping until a "donut" of grout of design consistency squeezes out from the collar of the borehole. **A thin cement slurry will precede the thicker grout front. Do no stop pumping until the thicker grout appears.** Determine the time between the first appearance of grout, which may be watery, and the appearance of design consistency grout. Record this time on the *Cablebolt installation layout* sheet. If a "tongue" of grout which appears solid but does not completely encircle the cable (i.e. a "donut"), keep pumping until the exiting grout forms a complete encapsulating ring. Make a note in the installation report of any unusual grout flow.
9) If grout has appeared at the borehole collar before the expected full volume of grout has been pumped into the hole, make a note of how much grout has been pumped into the borehole on the *Cablebolt installation report* sheet, and inform the engineer of the problem.
10) Stop the grout pump.
11) Kink over the end of the grout tube and tie it off.
12) Disconnect the grout tube from the grout pump hose.
13) Start the grout recirculating again in the pump if you are not immediately ready to start grouting the next borehole.
14) Use all of each batch before starting to mix more grout. Never mix grout continuously.
15) Monitor the volume of grout in the hopper. If it appears that you will not have enough grout to fill the next borehole, discard the remaining grout and mix a fresh batch.
16) If there are any delays which prevent all of the grout from being pumped into the boreholes within *1/2 hour* of the start of the mixing time, pump the grout out onto the floor of the working area, and clean the grout pump using the procedures given in Procedure CB-K. If there are additives in the grout mix, or if high early strength grout is being used, the time that the grout can remain in the pump will be less.
17) Whenever the pump will be idle for more than *1/2 hour*, make sure that the pump is completely clean and is left full of clean water.
18) Never stand under newly grouted boreholes. Any water dripping out of the holes can burn skin badly.
19) Where surface fixtures are to be used, wait *24 hours* and then follow the instructions in Procedure CB-L. If surface fixtures are not to be used, trim any cablebolts protruding from the holes that could create a safety hazard.
20) 24 hours or more after the grouting has been completed, cut off the kinked over grout tubes.

CB-J: Grout Pumping continued

J3: Breather tube installation method in fractured rockmass: fractured zone extent known in advance.

Follow normal grouting procedures (CB-J1), with these additional instructions. This procedure will work when the longer grout tube has been inserted past the fractured section of the borehole. If grout continues to be lost into the rockmass when pumping through the longer grout tube, use a longer grout tube (#2) or even a third (longer still) as required.



1) When installing the cablebolt, insert two grout tubes into the borehole. The first grout tube, #1, should be inserted a short distance into the collar of the borehole following normal procedure. The second grout tube, #2, should extend just beyond the expected position of the fractured rockmass zone.
2) From Figure 3.9.1, estimate how much grout is required to fill the borehole between the collar and the end of the longer grout tube, #2, and between the end of grout tube #2 and the toe.
3) Pump grout through grout tube #1. Monitor the grout level in the hopper.
4) Stop grouting as soon as good quality grout flows out of the longer grout tube, #2, or after you have pumped *twice* as much grout into the hole as should be required to fill the hole.
5) Tie off grout tube #1.
6) Drain the grout out of grout tube #2. Clean and blow air in the end of the tube. If it is blocked, pump a small amount of water to clear the tube.
7) Detach grout tube #1 from the grout pump hose, and continue on to grout the next borehole or clean the pump.
8) Return after 24 hours, and pump grout through grout tube #2.
9) Keep pumping until grout of design consistency returns along the breather tube. Stop the grout pump, and follow normal procedures as in CB-J1.
10) If grout is flowing through the fractures and obstructing the grouting of other boreholes, drill, install and grout each cablebolt individually.

CB-J: Grout Pumping continued

J4: Grout tube installation method in fractured rockmass: fractured zone encountered during grout pumping.

The thicker grouts used in the grout tube method should flow along the borehole more readily than into the open discontinuities in the rockmass. Therefore there should be little problem with grouting in fractured rock with the grout tube method and grouts of $W{:}C < 0.35$. However there have been reports of the thicker grout "freezing" in uphole installations in very fractured rock (Oliver, 1992) due to water loss. This happens when the water lubricating the flow of the grout along the borehole wall flows away into the rockmass leaving the grout too dry to flow well. There are two options in this case. The first option is to push a second piece of grout tube into the borehole until it reaches the position of the frozen grout front, and to continue to grout the hole with the new tube. The second option involves washing the grout out of the hole and the grout tube, and trying to regrout the hole normally. In this case, it is essential that the grout is washed from the hole by water introduced at the collar, and that water is not pumped into the grout tube until the hole is basically clear of grout.

J5: Grout tube installation method in fractured rockmass: fractured zone recognized in advance of pumping.

In locations where the grout front frequently freezes in the borehole, the conductivity of the open discontinuities must be reduced. This can be achieved by spraying a thin grout mixture onto the borehole walls with a slotted pipe attached to the end of the grout pump hose. Manufacture a short length of pipe with a sealed upper end, and thin slots cut at numerous positions around the circumference of the pipe. Attach a connector end to the lower end of the pipe to attach a grout tube. Pump a relatively thin grout slurry out through this pipe. Once the grout slurry layer has set, normal grout pumping procedures should work (Oliver, P., personal communication, 1993).

CB-K: Grout Pump Clean up

K1: Cleaning all grout pumps

A complete cleanup of the grout pump is essential to ensure efficient and clean operation, and to reduce the maintenance required. Detailed instructions for the clean up of the grout pump should have been provided by the pump supplier. Add these instructions to this procedure. Work with the crew to ensure that the instructions given in the procedure result in a completely clean pump.

CB-K: Grout Pump Clean up continued

K2: Cleaning the Piston pump

Whenever the pump will be idle for more than 15 minutes:

1) Pump as much of the grout from the hopper as possible.
2) Before the pump runs completely dry, fill the hopper with fresh water.
3) Continue filling the hopper with water and operating the pump until the water flowing from the end of the grout hose is clear.
4) Scrape any accumulations of grout from all exposed surfaces of the pump.

At the end of each shift:

5) Remove the foot valve by unscrewing with a pipe wrench. Wash the ball and ball stop.
6) Remove the shaft casing or riser tube taking care not to damage or bend it. Wash the inside of the tube.
7) Re-assemble the pump and pump fresh water through it.
8) Blow air through the grout hose.

K3: Cleaning the Progressing cavity pump

Whenever the pump will be idle for more than 15 minutes:

1) Pump out all unused grout, but do not operate the pump dry.
2) Wash down the hopper, paddles and housing. Add water to the hopper and continue to pump until the water flowing from the grout hose is clear.
3) Leave the hopper full of water until ready to pump the next batch.

At the end of the shift:

4) Disconnect the grout hose, open the end clamps and remove the rotor/stator and auger. Disassemble the rotor/stator and wash all components.
5) Blow air through the grout hose.
6) Reassemble the pump, and fill the hopper with water.

CB-L: Surface Fixture Installation

L1: Plain plates, Domed plates, Butterfly plates, Straps.

1) Collect the wedges and barrels. If there are different types of wedges and barrels in use on site, ensure that the wedges match the barrels. When the wedges are placed inside the barrel without the cablebolt, the wedges should fit snugly and be of the same taper angle as the barrel, should not stick out from the bottom of the barrel, and should be flush with or slightly protruding from the top.

CB-L: Surface Fixture Installation continued

2) Do not use any rusty or oily wedges or barrels. If there is mud on the wedges or barrels, or the wedge teeth contain grit, clean them completely.
3) Collect the plates and/or straps. Ensure that the plates are of the correct type, surface dimensions, thickness and shape. The straps must have holes of the specified diameter at the correct spacing and be of the specified thickness and surface dimensions.
4) Make sure that the hydraulic jack used to install the wedges and barrels is ready. If the jack has a spring load nose cone, check it every so often. It should be almost impossible to compress the spring with your hand. If the spring is soft, replace it. Clean any rust or dirt from the jaws of the jack.
5) Clean the end of the cablebolt, so that there is no mud, dried cement grout or other substance on the wires. Cut off any exposed tubing.
6A) If using single strand, place the strap or plate over the end of the cablebolt.
6B) For double strand cablebolts, bend over or cut off the end of one of the cablebolt strands. Place the strap or plate over the end of the other cablebolt.
7) Fit the strap over all of the cablebolts that it will be used on to check that it will fit. It is important to do this before installing any of the wedges and barrels so that any problems will be identified before you start. If some of the cablebolts will not fit through the pre-drilled holes in the strap, move onto the next group of cablebolts and inform the engineer of the problem at the end of the shift.
8) Ensure that the plate or strap is roughly perpendicular to the cablebolt. If the cablebolt is angled more than 25° away from perpendicular to the rock surface, use a spherical washer between the plate and the barrel, or use a rounded barrel and domed plate to allow the cablebolt to remain straight. Note where spherical elements have been used on the Cablebolt installation layout sheet. Kinking the cablebolt sharply (> 25°) at the collar of the hole will damage, weaken or break the wire(s).
9) Make sure that the dome on the domed plates will be facing away from the rock face, so that the flat part of the plate rests on the rock and the dome can be compressed.
10) Position the barrel over the cablebolt, against the *plate* or *strap*.
11) Place the wedges over the end of the cablebolt and slide up to the barrel. Make sure that the individual wedges are evenly spaced, and not touching.
12) Using the jack, tension the cablebolt to the load specified, making sure that no part of the jack is bearing against the wedges, with the exception of the setting spring. The jack should only bear against the barrel, otherwise the wedges may split or fracture. Take care that the jack is always supported, especially when working on overhead cablebolts. *Add any instructions for the operation of the particular jack.*
13) If any of the *plates* or *straps* look at all loose, inform the engineer.

Installation Trouble Shooting

Problems will arise during the cablebolt installation process. Suggestions for solutions to some of the problems that might arise during installation are given here. A number of suggestions are provided by Pakalnis et al. (1991).

1) *Borehole blocked so that it is impossible to insert the cablebolt.*

Try to dislodge the obstruction with a grout tube. If it is possible to insert the grout tube to the end of the hole, flush the hole with water. Redrill the hole if it is impossible to clear the obstruction. If hole obstruction is a frequent problem, try to insert and grout the cablebolt as soon as possible after drilling is finished.

2) Grout pump won't run.

Check for any restricted air lines, blockages in the grout hose, closed valves or dirty filters.

Check the air supply pressure and volume.

3) *Borehole collar leaks during grouting.*

Check to ensure that the grout is of the correct consistency. If it is too thin, work with the crew until the correct grout consistency is being mixed and pumped.

Alter the collar plugging method for breather tube installations if the plug continues to leak.

4) *No air flowing from breather tube, and grout pump stalling at the start of hole grouting, indicating that the breather tube is blocked.*

Ensure that the breather tube is placed at least 15 cm below the toe of the hole.

Flush the breather tube with water prior to grouting to remove any blockages.

Use a stronger tube to prevent pinching of the tube or the collapse of the tube under pressure (Cluett, 1991). Increase the diameter of the breather tube.

5) *No air flowing from the breather tube, and grout pumping well, indicating that the rockmass around the borehole is fractured.*

Stop grouting, drain the grout from the tubes and hole, and allow the grout to set in the fractures. Return in 24 hours or later to finish grouting the hole.

3.9.3 Quality Control

Quality control guideline: Cablebolt placement

Quality Control		Consequences of poor quality control	How to achieve good quality control
Good	Poor		
Clean cablebolt	Dirty cablebolt	The cablebolt / grout bond will be reduced by mud, oil or dirt coating the wires of the cablebolt strand.	Store the cablebolts in a clean, dry area until ready for use. Keep the cablebolt cutting / assembling area clean and dry. Support the cablebolts off the ground, or dispense them from a reel or carousel.
Cable hanger well attached and/or of the correct diameter	Cable hanger poorly attached and/or of the wrong diameter	The cablebolt could fall from the hole before the grouting is complete, creating a safety hazard.	Ensure that the hanger is strong enough to hold the cablebolt in the hole, is securely attached and is of the correct diameter for the borehole. Bent wire hangers should be of the correct length, and bent out to the correct angle.
Breather or grout tube near the end of hole	Breather or grout tube not to end of the hole	If the breather or grout tube is too short in uphole installations, then the toe of the hole will be left empty of grout. If either tube is right at the end of the hole, grout flow will be difficult.	Attach the breather or grout tube at least *15 cm* or *6 inches* from the toe end of the cablebolt.
Cablebolt centred in the hole	Cablebolt at the edge of the hole	The cablebolt circumference will not be completely embedded in the grout, likely resulting in reduced capacity.	Use spacers every 1 m along the length of plain strand cablebolts and attach them securely. The cages of modified geometry cablebolts keep the cablebolt away from the edge of the hole.
Hole collar fully plugged	Hole collar not well plugged	Grout will leak out of the hole, leaving voids in the grout column. The leaking grout can burn anyone working below the hole collar. A well designed and installed plug may leak if very wet grout is installed.	Establish a collar plugging method that is adequate to completely seal the collar and can withstand the pressure exerted by the grout column on the plug. The method must also be quick and easy to accomplish.

Quality Control: 3.9.3 continued

Quality control guideline: Grout mixing

Quality Control		Consequences of poor quality control	How to achieve good quality control
Good	Poor		
Dry, fresh cement	Lumpy cement	The overall cement grout strength will be reduced due to partial early hydration. The grout mixer and pump can be damaged by lumps in the grout as well. Lumps lodged in the grout or breather tube or within the pump will prevent complete grouting of the hole.	Ensure that cement bags are well water proofed during transport and storage, by wrapping the palettes in plastic. Store the bags of cement in a dry, shaded, clean place until required for use. Remove any lumps from the cement powder before placing it in the mixer, and discard any bags with a lot of lumps. Do not stack palettes more than 2 high.
Grout of correct *W:C*	Grout too wet	Cablebolt capacity will be reduced from design level. Corrosion resistance of the grout is reduced. Grout will flow out of the borehole in the uphole grout tube installation method, or leak past the collar plug in the breather tube method.	Ensure that the specified quantities of grout and water are placed in the mixer. Any mixing problems should not be rectified by adding more water to the mix, but should be brought to the attention of the underground supervisor and the engineer, so that the grouting system or grout mix design can be modified.
	Grout too dry	Cablebolt capacity may be reduced from design level, if air voids are entrained in the grout column when the grout is too dry to flow properly. The mixing and pumping systems may not function adequately.	
Grout with correct volume of additive	Too much additive	The grout strength may be reduced if too much additive is added, and/or the grout may not flow as designed.	Ensure that the additive is easy to measure out, such as by using a scoop for dry powder or a measuring container for wet liquid. A possible solution to problems with controlling additive volume is to purchase the grout with dry additive added to the mix at the plant before bagging.
	Not enough additive	The grout will not flow as designed, and may hang up in the borehole during pumping or may leave voids in the grout column.	

Quality Control: 3.9.3 continued

Quality control guideline: Grout pumping

Quality Control		Consequences of poor quality control	How to achieve good quality control
Good	Poor		
Grout fully mixed	Grout not fully mixed	If the grout is not completely mixed, it will be harder to pump the grout mix, and the grout strength will vary along the length of the hole.	Ensure that the grout is completely mixed, with no lumps and dry patches. Mix in a batch and not continuously, to ensure consistent water:cement ratio and grout strength. If the grout was not recirculated during mixing, pump the grout through the pump hose onto the floor until a consistent, non-watery grout is flowing, before starting to grout the hole.
Breather or grout tube near the end of the hole	Breather or grout tube at the end of the hole	If the breather or grout tube is pushed right to the end of the hole, grout flow will be impeded. The grout will not flow down the breather tube, or the grout tube might burst.	Attach the breather or grout tube at least *15 cm* or *6 inches* from the toe end of the cablebolt, during the placement of the cablebolt. Cut the end of the tubes at a 45° angle.
Breather / grout tubes fully grouted	Breather / grout tubes not fully grouted	If the tubes are not fully filled with grout, then a void will be left in the column of grout. The larger this void, the lower the cablebolt load carrying capacity	Ensure that the grout returns along the breather tube. When grouting is complete, bend over the ends of the breather and/or grout tubes, and tie securely.
Borehole filled with grout	Borehole not full of grout	Any gaps in the grout column will reduce the capacity of the cablebolt system. These gaps could be caused by grout slumping or "blobbing" down the hole in uphole toe to collar installations, by grout leakage at the collar of any uphole installation, grout leakage into a fractured rockmass in any hole orientation, or by segregation of the grout column (for grouts of water:cement ratio greater than 0 4).	Match the grout flowability to the installation system and the cablebolt geometry to try to ensure that the impedances to full column grouting are minimal (Figure 2.5.9. This can be checked with a pipe pumping test. Make sure that it is possible to mix and pump the grout water cement ratio specified in design and that the correct *W:C* is being installed.

Quality Control: 3.9.3 continued

Quality control guideline: Surface fixture installation

Quality Control		Consequences of poor quality control	How to achieve good quality control
Good	Poor		
Cablebolt and fixtures clean	Cablebolt and fixtures dirty or rusty	The strength of a rusty cable or rusty fixtures will likely be weakened, and mud or rust will reduce the frictional interaction between the wedges and cable.	Use clean, fresh cablebolts and fixtures.
Free end of cablebolt long enough: inside and outside the hole	Free end of cablebolt too short: inside and outside the hole	Inside the hole: If there is no free length of cablebolt inside the hole, only a small % of the tensioning load applied will remain.	Debond a section of the cablebolt near the collar of the hole.
		Outside the hole: A short piece of cablebolt outside the hole will not leave anything for the tensioning jack to grip on to.	Ensure that a long section of cablebolt is protruding from the hole after insertion. Do not cut the cablebolt off until after the surface fixture has been installed.
Cablebolt free length perpendicular to rock face	Non 90 angle between cablebolt and plate	Geometric mismatch between the cablebolt and the rock surface will create a sharp bend in the cablebolt element during tensioning. Some of the strands in the cablebolt may be broken.	Design the cablebolt pattern so that the hole angle is roughly perpendicular to the rock surface. The maximum angular mismatch should be no more than 25 .
Barrel and wedges fit together well	Barrel and wedges mismatched	Mismatched wedges and barrels will not fit well together, and may not retain the load applied during plating, or the steel wedges or barrel may chip or split due to eccentric loading.	Ensure that matched wedges and barrels remain together. If there are several different sizes and types in use, make sure that each kind is very clearly and distinctly marked.
Correct plate	Wrong plate	If the plate is too small, thin, or soft, then the face restraint on the rockmass may not be adequate.	Use the plates designed for the job, and ensure that the plates purchased are stiff and strong enough.
Tension jack good	Tension jack too soft	If the spring in the nose cone is soft, the tension applied by the jack will be lost.	Check the spring stiffness in the nose cone of the jack periodically. Replace soft springs immediately. The teeth in the grips should also be sharp and clean.

3.9.4 Feedback

Feedback: Cablebolt installation observation report

Plan #: Stope #:

Cablebolt ring #s:

Date: Crew Leader:

Grout batch #	*W:C*	Grout additive	Batch mixing time	Any problems with cement quality or mixing equipment? Hole #'s grouted from this batch.

Com-ment #	Ring #	Hole #	Batch #	Deviation from design, problems with equipment, materials or installation procedure?

Changes to the layout should be commented upon on this sheet, and noted on the *Cablebolt installation layout* sheet.

3.10 Automated Cablebolting Systems

The design specifications, installation procedures and quality control guidelines discussed in Sections 3.7 through 3.9 are generally applicable to the installation of cablebolts with automated systems. However, it will be necessary to modify the sheets to reflect the specific requirements and procedures for automated equipment. Some suggestions for modifications to the sheets are given below for automated cablebolt installation equipment.

The automated cablebolting machines grout the borehole first, and then insert the cablebolt into the grout filled hole. It is essential that no part of the operation results in substantial voids being introduced into the grout column. For example, if the cablebolt is secured in the hole by a hanger created by pulling a section of the cablebolt out of the hole, kinking the cablebolt, and then pushing it back into the hole, grout will be lost from the hole (Bawden, Hyett and Cortolezzis, 1992). In addition, as was mentioned in Chapter 2, the grouting equipment supplied with the automatic cablebolting machine must be capable of completely mixing a batch of $W{:}C < 0.35$ grout and of pumping it into the longest borehole that will be used in the cablebolt patterns at the mine site.

The main difference between the automated systems and the regular cablebolting procedure is that one person per shift operates the equipment and is responsible for the entire cablebolting job, from drilling the hole to inserting the cablebolt into the grout filled borehole. The operator works under a protective canopy at all times. The operator has the benefit of drilling the boreholes and installing the cablebolts, thereby gaining continuous experience with the rockmass in the working area. It is still very important that the operator record the drilling and installation feedback information as suggested in Sections 3.8.4 and 3.9.4, so that the engineer can monitor changes to the cablebolt design and rockmass conditions. It is likely that a different crew will install the surface fixtures where they are used.

The manufacturers of automated systems should provide operational instruction in the form of manuals and hands on training sessions. All of the relevant, machine-specific instructions should be added to the sheets in this section.

3.10.1 Automated System Design Specifications

The cablebolt design specifications given to the cablebolting machine operator, should include modified versions of the *Cablebolt Layout Plan and Section* (Section 3.8.1) and *Cablebolt Installation Layout and Notes* (Section 3.9.1).

The *Cablebolt layout plan and section specification* sheets will be directly applicable to most uses of automated cablebolting equipment. However a number of changes to the *Cablebolt Installation Layout and Notes* will have to be made. For example, there is usually no variation in the cablebolt type specified in the design, since individual, specialized pusher heads are required for different types of modified cablebolts.

Specification: Automated cablebolt installation layout (See attached NOTES)

Plan #: Stope #:

Cablebolt ring #:

○ See attached *Cablebolt drilling observation* sheet.

Cablebolt type:

Cablebolt #	As drilled		Design specification		Feedback: As installed				
	Collar dist. from ℄ Hole angle from ℄	Drilled hole length	Cablebolt length	Surface fixture type	Cablebolt length	Surface fixture type	Grout batch #	Grout flow notes	Comment #
1									
2									
3									
4									
5									
6									
7									
8									
9									
10									
....									

Date cablebolts grouted and placed: Operator:

Date surface fixtures installed: Crew Leader:

See attached *Installation layout NOTES* for additional information about this sheet.

In the feedback section of this sheet, please indicate where the installation followed design with a check mark, ✓, or record any change(s) made. Any comments about or problems with the equipment or procedure, as well as feedback on the grouting procedure must be noted on the *Cablebolt installation observation report*.

Automated System Design Specifications: 3.10.1 continued

Specification: Automated system cablebolt installation layout NOTES		
Collar dist. or angle from ℄	The hole collar distance is measured horizontally from the ℄. The position is specified as angle from vertical ℄. (+ CW; - CCW)	
Cablebolt type	PS	Plain 15.2 mm diameter strand.
	TS	Twin 15.2 mm diameter strand.
	NC	25 mm diameter nutcaged 15.2 mm diameter strand.
	BA	25 mm diameter bulbed anchor formed from 15.2 mm diameter strand.
Surface fixture type	Pl	Plate of *x* by *y* surface dimensions and *z* thickness.
	BPl	Butterfly plate of *x* by *y* surface dimensions and *z* thickness.
	DPl	Domed plate of *x* by *y* surface dimensions and *z* thickness.
	Str	Strapping of *x* by *y* surface dimensions, *z* thickness and *s* spacing between holes of diameter *d*.
Grout batch #	A record of the batch mixing details must be kept on the *Cablebolt installation observation report* sheet. The grout batch # must be recorded on the *report* sheet and the *layout specification* sheet.	
Grout flow observation	Record the time between the appearance of watery grout at the collar of the hole and of grout of design consistency, (sec). Note if there is any flow of grout from adjacent boreholes during grouting, or from the borehole at any time after grouting is finished or after the cablebolt has been inserted into the hole.	
Comment #	Any comments about the performance of equipment, materials or procedures should be noted on the *Cablebolt installation observation report* sheet. Cross-reference to the *layout* sheet with a comment #.	

3.10.2 Automated System Procedure and Safety

The cablebolt installation and grouting procedures must include any safety guidelines applicable to the operation of the automated system. The order of work presented in the installation procedures provided in this book for manual installations will have to be altered for the automated equipment, because the operator first drills the borehole, then grouts the borehole and finally inserts the cablebolt(s) into the grout filled hole. Some samples of modified procedure sheets are given in the following text. It is essential that any routine maintenance procedures, as well as commonly encountered problems and their solutions be added.

Procedure: Automated system grout mixing and pumping and cablebolt placement

1 Spray the boom with oil before starting the grouting operation. This will help to prevent the grout from sticking to the machine.

2 If you are working in very fractured ground, run the cablebolt up the hole first, to clear away any loose rock that could obstruct the hole. If the rockmass is very fractured, drill the boreholes for one ring of cablebolts, then install the grout and cablebolts immediately in that ring.

3 Place the required amount of water into the mixer hopper.

4 Start the mixer blades rotating, and slowly add the required number of cement bags. Keep mixing until the grout is of a smooth consistency. Make sure that the quantities of water and cement in the mix are correct. The mixer blades must be kept rotating at all times while the batch is in the hopper.

5 Pump grout through the grout tube until you see design consistency grout flowing from the end of the tube.

6 Stop the grout pump, and insert the grout tube to the toe of the borehole.

7 Turn on the high pressure water jet on the boom to clean the outside surface of the grout tube as it is being withdrawn from the hole.

8 Start the grout pump and automatic withdrawal of the grout tube. If you see the grout tube starting to buckle, it is not being retracted quickly enough.

9 When the grout begins to flow from the collar of the borehole, the grout tube should be about *0.5 to 1 metre* above the collar of the hole. Once the grout tube has been withdrawn completely from the hole, stop the pump. If the grout continues to flow from the borehole after pumping has stopped, the grout is too wet, either due to poor quality control on the mix or due to the presence of flowing water in the borehole. Note the problem on the cablebolt installation observation report. If the problem is severe, the grouted hole cannot be used.

10 Within *15 minutes* of any grouting system break down, wash the pump and flush the tube with fresh water.

11 Insert the cablebolt into the grout filled hole. Keep the head of the boom within about 1 foot of the face, to minimize the free length of cablebolt, thereby reducing the possibility of kinking the cablebolt during insertion.

12 Periodically check the cablebolt reel under the machine. If the cablebolt looks rusty, you will probably encounter difficulties with the flow of the cablebolt off the reel. If you run into problems with the cablebolt feed, change the reel. Place the bound cablebolt pack into the cassette and then cut the steel binding straps.

13 When the cablebolt is about *3 metres* from the toe of the hole, use the *ram* to kink the cablebolt. It is important that the cablebolt is not retracted too much from the hole during this operation, since this will result in grout loss. (*Note that the process of securing the cablebolt in the hole can be quite different from site to site, depending upon the specific operation of each machine*).

14 Stop pushing the cablebolt once it has reached the toe of the hole.

15 Cut the cablebolt off using the cutter head.

Procedure: Automated system clean up	
1	Wash the mixer thoroughly using the high pressure water jet. Flush the water and left over cement out through the bypass valve at the bottom of the mixer.
2	Fill the pump hopper with fresh water and flush water through the grout tube system until clear water is running from the end of the tube. Leave the hopper full of clean water after all of the tubes have been flushed clean.
3	Wash the boom and grout tube reel with the high pressure water jet to remove all cement.

3.10.3 Automated System Quality Control

Many of the quality control guidelines listed for conventional cablebolt installations are applicable to an automated installation. Modify the following sheets as required:

Borehole preparation quality control guidelines	3.8.3
Cablebolt placement quality control guidelines	3.9.3
Grout mixing quality control guidelines	3.9.3
Grout pumping quality control guidelines	3.9.3
Surface fixture installation quality control guidelines	3.9.3

3.10.4 Automated System Feedback

The installation feedback is equally important for conventional and automated cablebolt installation procedures. Modify the following sheets for your site and usage of the cablebolting machine:

Drilling observation report	3.8.4
Cablebolt installation layout and notes	3.9.1
Cablebolt installation observation report	3.9.4

A number of sites using automated cablebolters were visited by the authors. In all cases, an extensive period of adaptation, machine customization and procedural adjustment was necessary before adequate efficiency was achieved.

3.11 Quality Control Monitoring and Testing

The maintenance of good procedures and quality during the cablebolt installation is critical and of great importance to safety and economics. Poor quality control can compromise the load carrying capacity of the cablebolts substantially.

The crews should be monitoring the quality of the installation and reporting any problems that they encounter (Section 3.5). The quality of the installation must also be monitored by the underground supervisors, engineers and technicians during spot check visits to the work area while the crew are installing the cablebolts. Methods for monitoring the quality during installation are given in Section 3.11.2. In addition, the quality of the installations should be checked after the installation is complete, following the guidelines given in Section 3.11.3.

If quality control problems are encountered, steps must be taken immediately to rectify the problems (Section 3.12). If the cablebolt system's performance is strongly compromised by the quality control problems, then additional cablebolts should be installed in the area.

3.11.1 Effect of Quality Control on Cablebolt Capacity

A cablebolt must be well installed in order to attain the load carrying capacity used in design: the clean, undamaged cablebolt strand must be completely surrounded by a full grout column of design water:cement ratio.

The load carrying capacity of cablebolts can be greatly reduced by poor quality control during installation. The capacity reductions that have been quantified in laboratory pull tests on cablebolts are listed in Table 3.11.1. Other factors which have not been quantified, but which can reduce the capacity of a cablebolt are listed in Table 3.11 2. Decreasing stress in the rockmass around the cablebolts can severely reduce their load carrying capacity as well (Chapter 2).

Table 3.11.1: Cablebolt capacity reductions due to poor installation quality control.

Muddy, greasy or heavily rusted cablebolts (Lappalainen and Pulkkinen, 1982; Leclair, 1995)	-50%to -70%
Ungrouted breather tube (11 mm I.D.: *W:C* = 0 45) (Goris, 1990)	-30%
Excess water in the grout mixture: (Reichert et al, 1992)	
W:C increased from 0.35 to 0 40	-15%
W:C increased from 0.40 to 0 55	-45%
Grout column incomplete	Up to -100%

If poor quality control during installation has been reported in an area, and is expected to reduce the cablebolt system capacity to an unacceptably low level, additional cablebolts should be installed.

Table 3.11.2: Impact of quality control problems

Reduction of cablebolt system capacity due to poor quality control		
1)reduction of the interaction between cablebolts in the pattern, 2)reduction of the capacity of an individual cablebolt, or 3) reduction of the grout quality or the bond strength.		
Borehole	Incorrect collar position or borehole orientation	1
	Incorrect borehole length or diameter	2
	Dirty or wet borehole	3
Cablebolt strand	Incorrect type or capacity	2
	Incorrect length	2
	Dirty, very rusty or oily	3
Cablebolt element placement	Cablebolt not to end of borehole	2
	Cablebolt not central in the borehole	2
	Grout tube or breather tube not at specified length	2
Grout mixing and pumping	Incorrect cement type	3
	Incorrect water:cement ratio	3
	Inadequate mixing time or mixing continuously	3
	Breather or grout tubes not full of grout	2
	Hole not filled with grout or grout segregation	2
Surface fixture installation	Incorrect or mismatched wedge and barrel fittings	2
	Incorrect or mismatched surface hardware	2
	Acute angle of the cablebolt with the face	2
	Incorrect installed tension	2
	Plate not in full contact with the rock surface	2
Subsequent effects	Blast damage	3
	Corrosion	2
	Machine damage	2
	Stress decrease	2

3.11.2 Checking Quality Control during Installation

Quality control can be checked during the cablebolt installation by:

1) Observing the installation process.

2) Inspecting the grout quality.

3) Testing the grout water:cement ratio.

4) Testing the grout strength.

Each of these checks should be made frequently, during visits to the working areas. The results of the checks should be discussed with the crews: either to reinforce that they are doing a good job, or to solve any problems that are identified.

Observation of Installation Practice

The drilling and cablebolt crew members are required to check certain aspects of the installation quality control while they are working, and to report on any problems with the installation procedure, materials or equipment.

The underground supervisor, the rock mechanics technician and the engineer should visit the work site often, to monitor the quality of the cablebolt installation, and to identify any problems with the installation procedure. These observations can be recorded on the *Cablebolt quality control check list*, which follows. Modify the sample check list given here to represent the procedures and equipment in use at your site. Any specific information, shown in italics on the sample sheet, about the borehole diameter, the grout mix design or any other item should be added to the sheet. It would be a good exercise to have all mine personnel involved with the cablebolting procedure complete this form from time to time to ensure that they remain aware of the potential quality control problems.

Any problems with the installation procedure that are observed should be discussed with the crew immediately, and should also be noted in the stope file.

When a new crew is formed, or new equipment or hardware is implemented, it is important to check the quality of the installation more frequently than usual. Even when the quality control seems to be excellent, and there are apparently no problems, it is important to continue to check the installation quality control every so often.

Cablebolt Quality Control Check List: Breather Tube Installations — *Page 1 of 2*

Your Name: ______________________ Date: ______________

Crew Location: ______________________________________

Please ✔ for a Yes answer to the following questions. Deviations from design practice or additional comments should be noted in the boxes on the right side of this form.

Drilling

		Comments
☐	Is the specified drill bit being used: Diameter = *65 mm*?	
☐	Are the holes all drilled to the design length: *10 m*?	
☐	Are the holes dry? If not, what quantity of water is flowing (l/s)?	

Material storage, transport and handling

☐	Are the cablebolts clean? If not, please specify the problem:	
☐	○ - muddy, oily or greasy?	
☐	○ - slightly rusty? (rough to the touch)	
☐	○ - very rusty? (slippery to the touch)	
☐	Are the cement bags free of lumps?	
☐	Are the cablebolts free of kinks and damage?	
☐	Are the breather or grout tubes kinked or damaged?	

Cablebolt placement

☐	Are dirty, oily or rusty cablebolts cleaned before placement?	
☐	Are the boreholes clean and dry before cable placement?	
☐	Are *two plain 7 strand* cablebolts being placed in every hole?	
☐	Is the *spring steel hanger* attached securely?	
☐	Are *56 mm* spacers being placed every *1 m* along the cablebolts?	
☐	Are *13 mm ID* breather and *19 mm ID* grout tubes being used?	
☐	Is the breather tube attached ≈*15 mm* below the cablebolt toe end?	
☐	Is there at least *1 m* of grout tube inside the hole?	
☐	Is each cablebolt *11 m* long, with *1 m* sticking out of the hole?	
☐	Are all of the flared sections of the cablebolt inside the hole?	
☐	Is the cablebolt secure in the hole?	
☐	Is the collar of the hole being well sealed with *cotton waste*?	

Cablebolt Quality Control Check List: Breather tube *Page 2 of 2*

Your Name: ______________________ Date: ______________

Crew Location: ____

Grout mixing

		Comments
☐	Consistent, reliable mixer operation? If not, does the mixer:	
☐	○ - sound laboured?	
☐	○ - ever stall?	
☐	How many *25 kg* cement bags being used in the mix?	
☐	Are *40 litres* of water being used in the mix?	
☐	Is the grout being mixed in individual batches?	
☐	Is the grout mixture smooth, well mixed and free of lumps?	
☐	Is the grout being mixed for at least *15 minutes* before pumping?	
☐	Is the grout mixer completely clean and left full of water after use?	

Grout pumping

☐	Is the grout pump hose / grout tube connection adequate?	
☐	Are the grout tubes free from leaks or bursting?	
☐	Does good quality grout always return down the breather tubes?	
☐	Does one grout batch fill *2* holes?	
☐	Are the collar plugs free of leaks?	
☐	Are the breather tubes kinked and tied off as soon as grouting stops?	
☐	Are the grout tubes kinked and tied off as soon as grouting stops?	
☐	Is the grout pump completely clean and left full of water after use?	

Plating and strapping

☐	Are the cablebolts roughly perpendicular to the rock surface?	
☐	Are the wedges and barrels free of rust, dirt and oil?	
☐	Do the wedges and barrels fit well together?	
☐	Are the plates tight after installation of the wedge and barrels?	

Any other comments or problems observed, or suggestions for improvement?

____

____

____

Cablebolt Quality Control Check List: Grout Tube Installations *Page 1 of 2*

Your Name: ____________________ Date: ______________

Crew Location: ______________________________

Please ✔ for a Yes answer to the following questions. Deviations from design practice or additional comments should be noted in the boxes on the right side of this form.

Drilling

		Comments
☐	Is the specified drill bit being used: Diameter = *65 mm*?	
☐	Are the holes all drilled to the design length of *10 m*?	
☐	Are the downhole collars covered to prevent water inflow?	
☐	Are the upholes dry? If not, what quantity of water is flowing (l/s)?	

Material storage, transport and handling

☐	Are the cablebolts clean? If not, please specify the problem:	
☐	○ - muddy, oily or greasy?	
☐	○ - slightly rusty? (rough to the touch)	
☐	○ - very rusty? (slippery to the touch)	
☐	Are the cement bags free of lumps?	
☐	Are the cablebolts free of kinks and damage?	
☐	Are the breather or grout tubes kinked or damaged?	

Cablebolt placement

☐	Are dirty, oily or rusty cablebolts cleaned before placement?	
☐	Are the boreholes clean and dry before cable placement?	
☐	Is a *single plain 7 strand* cablebolt being placed in every hole?	
☐	Is a *spring steel* centralizer used on the end of each cablebolt?	
☐	Is a *19 mm ID* grout tube being used?	
☐	Is each grout tube attached ≈*15 mm* from the cablebolt toe end?	
☐	Is each cablebolt *11 m* long, with *1 m* sticking out of the hole?	
☐	Are wooden wedges placed between the cablebolt and hole collar?	
☐	Is each grout tube round and free of distortion by the wedge?	
☐	Are the cablebolts secure in the hole?	
☐	Are at least *2 m* of grout tube left hanging from each hole collar?	

Cablebolt Quality Control Check List: Grout tube *Page 2 of 2*

Your Name: ______________________ Date: ______________

Crew Location: ____

Grout mixing

		Comments
☐	Consistent, reliable mixer operation? If not, does the mixer:	
☐	○ - sound laboured?	
☐	○ - ever stall?	
☐	How many *25 kg* cement bags being used in the mix?	
☐	Are *35 litres* of water being used in the mix?	
☐	Is the grout being mixed in individual batches?	
☐	Is the grout mixture smooth, well mixed and free of lumps?	
☐	Is the grout being mixed for at least *20 minutes* before pumping?	
☐	Is the grout mixer completely clean and left full of water after use?	

Grout pumping

☐	Is the grout pump hose / grout tube connection adequate?	
☐	Are the grout tubes free from leaks or bursting?	
☐	Does watery grout flow from the holes at first?	
☐	Is pumping continued until thick grout flows from the collar?	
☐	Does one grout batch fill *2* holes?	
☐	Are the borehole collars free of grout leaks after grouting stops?	
☐	Are the grout tubes kinked and tied off as soon as grouting stops?	
☐	Is the grout pump completely clean and left full of water after use?	

Plating and strapping

☐	Are the cablebolts roughly perpendicular to the rock surface?	
☐	Are the wedges and barrels free of rust, dirt and oil?	
☐	Do the wedges and barrels fit well together?	
☐	Are the plates tight after installation of the wedge and barrels?	

Any other comments or problems observed, or suggestions for improvement?

____

____

____

Visual Inspection of Grout Quality

A visual inspection of the grout mixture will give a fairly accurate estimate of the water:cement ratio (*W:C*) of the grout. The appearance of the grout at different water:cement values can be estimated from descriptions and photos. It is a good idea to prepare photos of grout with the crew and any other people who will be checking the quality control, so that they can see grout of different water:cement ratios. It is essential to make your own set of photos for any grout mixes that include additives, since the visual appearance of grout mixtures can be quite different. Add any specific details to the description of the grout mixtures given in Table 3.11 3 that would be useful.

Always wear long water proof gloves when handling grout mixtures.

Table 3.11.3 Grout characteristics

Grout characteristics (after Hyett et al, 1992)		
water:cement ratio	Grout characteristics at end of grout hose	Grout characteristics when handled
< 0 30	Dry, stiff sausage structure.	Sausage fractures when bent, and grout is too dry to stick to your hand. Grout can be rolled into balls.
0.30	Moist sausage structure, which "melts" slightly with time.	Sausage is fully flexible and grout will stick to your hand. Grout is easily rolled into wet, soft balls.
0.35	Wet sausage structure, which "melts" away with time.	Grout sticks readily to your hand, even when upturned.
0.40	Sausage structure is lost immediately. Grout flows viscously under its own weight to form a pancake.	Grout sticks readily to your hand, but can be shaken free.
0.50	Grout flows readily and splashes on impact with the ground.	Grout will drip from hand, with no shaking required.

Visual Characteristics of Grout

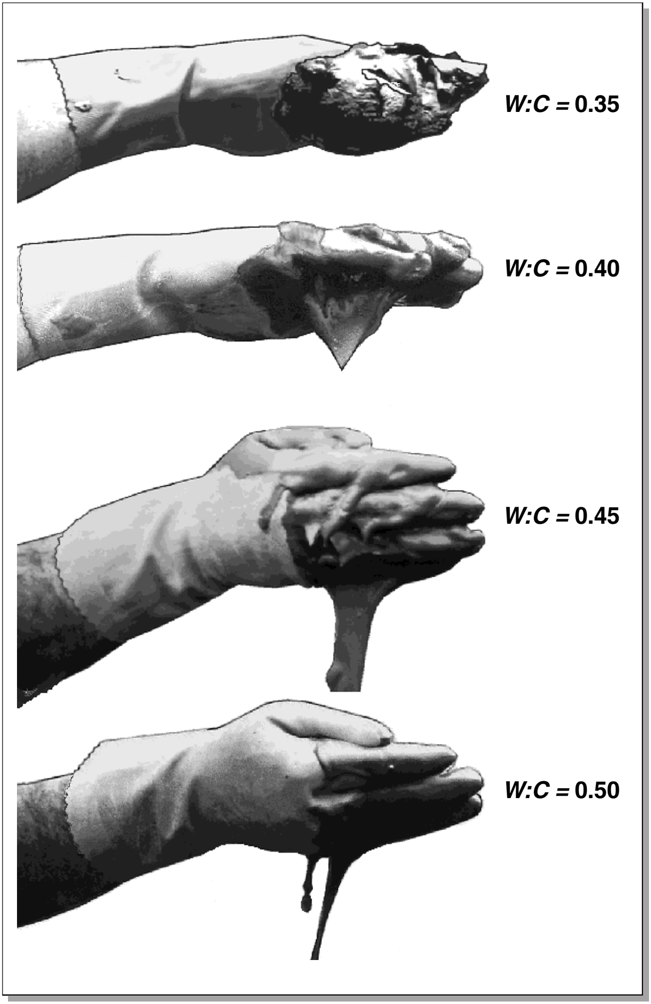


Figure 3.11.1: Visual characteristics of grout - an aid to quality control

Grout Water:Cement Ratio Testing

The water:cement ratio of the grout can be easily measured by taking samples of the grout during visits to the underground working location, using the following procedure (Rheault, J., 1993, personal communication). Be aware, however, of the inherent scatter in wet cement paste density (Figure 2.5.2).

1) Purchase some 1 litre plastic containers with screw top lids. Weigh the containers without their lids. This value is $M_{(cont)}$.

2) Completely fill each plastic container with water measured from a graduated cylinder to determine the volume of the container = $V_{(cont)}$.

3) Fill the container with grout taken from the end of the grout hose, mid way through pumping a batch of grout. Close the lid of the container firmly. *Always wear long water proof gloves when handling grout mixtures.*

4) Weigh the grout filled container without the lid. This value is $M_{(grt+cont)}$.

5) Fill any voids in the container with a volume of water measured from a graduated cylinder. The volume of the extra water added is $V_{(w)}$.

6) Calculate the water:cement ratio, *W:C*, of the grout, by solving the following equations:

Assume that water has a specific gravity, $S.G._W$, of 1.0
(anything else and you shouldn' t be using it for grouting!):
Specific gravity of cement, $S.G._C \approx 3.15$ (Hyett et al., 1992)

$$M_{(grt)} = M_{(grt+cont)} - M_{(cont)} \qquad \{\text{mass of grout in kg}\}$$

$$V_{(grt)} = V_{(cont)} - V_{(w)} \qquad \{\text{volume of grout in litres}\}$$

$$A = \frac{S.G._C \left(M_{(grt)}[kg] - V_{(grt)}[litres]\right)}{S.G._C - 1} \qquad \{\text{mass of cement in kg}\}$$

$$B = M_{(grt)}[kg] - A \qquad \{\text{mass of water in kg}\}$$

$$W{:}C = \frac{B}{A} \qquad \{\text{water: cement ratio}\}$$

Grout Strength Testing

The strength of the grout used in the cablebolting holes can be measured by testing the strength of samples collected at the working face, using the following procedure (Gendron et al, 1992). Noranda uses custom built, foam rubber lined aluminum sample cases for storing and transporting the grout samples. The grout samples are collected in PVC tubes (2" inside diameter and 8" length) with plastic screw threaded end caps. Six of these tubes fit tightly into the case.

1) Spray the inside of the PVC tube with silicone spray, so that the grout sample can be extracted easily from the tube prior to testing.
2) Take random samples of the grout mid way through pumping a batch. *Always wear long water proof gloves when handling grout mixtures.*
3) Transport the samples to the surface. Try not to disturb the samples during transport, and certainly do not turn the sample tubes upside down.
4) Send the cylinders for testing at any time after 7 days of undisturbed hydration. Indicate the date each sample was taken, and when it should be tested.
5) Plot all results on a graph, such as Figure 3.11.1, and note the location of any cablebolts grouted with low strength grout. If the grout is very weak throughout an area, more cablebolts should be installed.
6) Inform the crew of the results. If the tests consistently indicate weak grout, work with the crew to determine the source of the problem.

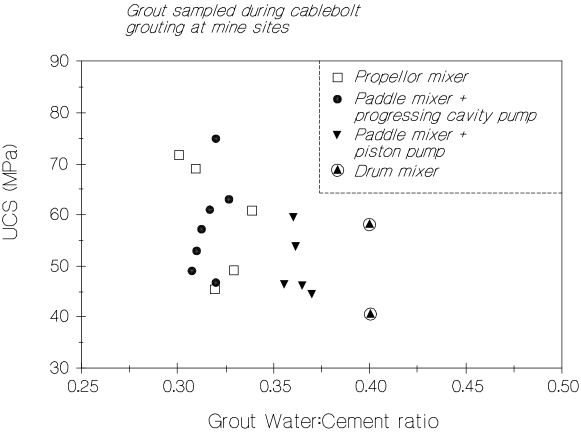


Figure 3.11.2: Results of *UCS* testing of field grout samples from mine sites (after Gendron et al, 1992).

3.11.3 Checking Quality Control after Installation

Post-Installation Visual Inspection of Cablebolts

It is more difficult to determine if there have been installation quality control problems during a post-installation inspection of the work area than from observations made during the installation. However, Table 3.11.4 suggests some of the items that can be checked after installation has been completed.

Table 3.11.4: Visual post-installation quality control checks.

Item to be checked	Method for checking
Grout quality	Look at any grout splashed onto the floor or walls of the work area. If the grout looks more fluid than usual, the water cement ratio was too high.
Completeness of grout column	Check the end of breather and/or grout tubes to see if they are full of grout. If possible, cut off the end of grout or breather tubes when making this check. If the tubes are not full of grout, then you should spend some time observing the crew as they install the cablebolts, since they may not be getting return on the breather tubes, or may be using grout that is too thin with the grout tube installation method. Check any visible borehole collars for completeness of grouting.
Plates and straps	Check the angle of the cablebolt with respect to the rock surface. The cablebolt behind the surface fixture should be roughly perpendicular to the surface. If it is not perpendicular, check the individual wires in the cablebolt strand for damage. Observe the plates and straps. They should be of the correct size and should be flush with the rock surface. You should not be able to move the fixtures with your hand. If the fixtures are loose, then they can be shaken off by blast vibrations, or will be too loose to perform their intended function. Check the orientation of any non-flat plates (e.g. domed or butterfly plates). The rounded part of the plates should be facing towards you (convex) and not in towards the rock face (concave).
Wedges and barrels	Try to move the barrel. It should be impossible to move it. Observe the wedges and barrels. The wedges should be protruding slightly past the end of the barrel, and should show no signs of damage. There should be no grout, oil or dirt on any of the wedge or barrel surfaces.

Post-Installation Cablebolt Pull Testing

Short lengths of cablebolts can be pull tested after installation. In these tests, a load is applied to the cablebolt and the resulting deformation is monitored. A detailed description of pull test procedures is given in Section 2.2.2. Pull tests for comparison of cablebolt bond strength to published test results should be carried out on cablebolts which are installed with 250 mm or 300 mm of grout bond. Such tests can provide site specific capacity estimates. Where stress change and in particular, stress relaxation is suspected, it is useful to perform field pull tests at different phases of the mining sequence (before and after stope excavation, for example). This will assess the risk of bond capacity reduction due to stress decrease (Section 2.6.2).

Pull testing of cablebolts bonded over their full length will only detect major deficiencies in the installation of a cablebolt. If a cablebolt is well installed with good quality grout, for example 0.35 *W:C*, in a rockmass of modulus E = 10 GPA, less than 1 metre of grout bond is required to break a single plain strand cablebolt. The bond length at which the cablebolt will break is termed the *critical bond* or *critical embedment length* (Section 2 6.1). The charts relating bond strength to rockmass modulus, grout quality and stress change that are contained in Section 2.6.2 can be used to estimate the critical bond length for plain strand cablebolts.

If a full length cablebolt pulls out of the grout during a pull test, then the grout is of extremely poor quality, the grout column is full of voids, and/or the rockmass surrounding the cablebolt has undergone a substantial stress decrease since the installation of the cablebolts. If the cablebolt does pull out, then it is essential to determine the cause of the failure, and to install more cablebolts in the area.

In most operational settings, cablebolts with grouted lengths more than 2m should only be pull tested when serious quality control problems are suspected such as incomplete hole filling due to grout loss or when serious stress decreases are suspected. It is not meaningful under normal conditions to perform quality control field pull tests on design length cablebolts which are fully grouted. The anchor length being tested (>2m) by pulling the cable at the face will almost certainly be longer than the critical embedment length (required to break the steel tendon). If these cables pull out at all, then a very serious problem indeed exists.

It is preferable then to periodically install tests cables (not as part of a regular array but in the same area between operational cablebolts) with all but 250mm of strand encapsulated in plastic tubing as illustrated in Figure 2 2.5. Grout the hole normally, leaving a sufficient length of cable extending from the hole for attachment of a testing jack. The pullout loads obtained can be compared (normalized with respect to the length of 250mm to obtain bond strength in kN/m) with the various results shown in Chapter 2.

3.12 Quality Control Improvement

Any problems with the cablebolt installation quality control must be discussed with the crew(s) and supervisors, and solutions should be found and implemented as soon as possible.

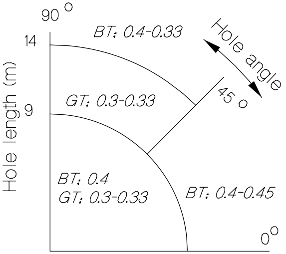
Descriptions of operational grouting problems and their solutions are given by Oliver (1992) and Bourchier et al (1992). These papers provide instructive examples of responses to problems identified in the cablebolt installation process at specific mine sites.

An interesting case history for quality control improvement at Trout Lake Mine, Canada was documented by Cluett (1991). The study was initiated when two large wedge failures occurred in cablebolted cut and fill stopes. When the quality of the cablebolt installations was investigated, it was found that:

- The grout was too thin, and the solid cement in the grout mix was settling, leaving an ungrouted, water filled section at the top of the upholes.
- The thin grout was escaping from the borehole into joints.
- The breather tube (6 mm diameter) in long holes (20 m) was being crushed by the increase in grout pressure in the hole created by trying to pump the grout back down the breather tube.
- The old piston pump in use was incapable of grouting the length of boreholes designed at the specified water:cement ratio.
- The grout was leaking from the collar of the boreholes.

To solve these problems at the mine, they:

- Purchased higher pressure rated breather tubes for the piston pump installations.
- Purchased a progressing cavity pump which can pump a thick grout (0.3 *W:C*) up a long hole (>15 m).
- Examined all of the cablebolt installations. Where poor quality cablebolts were suspected, additional cablebolts were installed.
- Provided additional training for the cablebolting crews.
- Formed a ground control committee consisting of ground support and engineering staff, and health and safety committee members. The members discuss any problems that arise with installation and equipment.



- Cablebolts are now installed soon after the drilling has been completed, so that the period of time during which the rockmass is unsupported is minimal.
- There is now a close liaison between the geologists, mining engineers, mine superintendents, and the mine operators.
- Grouting guidelines, dependent upon hole length and angle were created. The piston and progressing cavity grout pumps are both still in use at the mine, but are only used in situations where it will be possible to completely grout the borehole. The pumping guidelines are reproduced here in Table 3.12.1 and in the figure on the preceding page as an example of what can be done to ensure the correct use of grout pumps at a mine site.
- The number of boreholes grouted per grout batch is always recorded now.

Table 3.12.1: Grouting guidelines for Trout Lake Mine, Canada (after Cluett, 1991).

	Length of hole (m)	Angle from horiz. (deg)	Pump	Breather tube (psi)	Grout tube (psi)	Collar Seal	Grout mix ($W:C$)*
Upholes	> 14	> 45	Minepro	450	250	Cement plug	0.33 to 0 4
	9 to 14	> 45	Minepro	None	100 or 250	None	0.3 to 0 33
	> 9	< 45	Spedel	450	100 or 250	Cement plug	0.4 to 0 45
	< 9	any	Minepro	None	75	None	0.3 to 0 33
			Spedel	75	75	Cement plug	0 4
Flat and down holes	> 9	any	Minepro	None	100 or 250	None	0.3 to 0 33
			Spedel	None	100 or 250	None	0 4
	< 9	any	Minepro	None	75	None	0.3 to 0 33
			Spedel	None	75	None	0 4

* Note that the authors of this handbook recommend a lower bound of 0.35 for water:cement ratio for most applications (Figure 2.5 9). If grout with $W:C = 0.3$ to 0.35 can be pumped and placed with confidence and with little difficulty as in Oliver (1992), then the use of such thick grouts may be warranted.

4 VERIFICATION: Cablebolt Performance Assessment

4.1 Introduction

The performance of cablebolts in the rockmass should be investigated to ensure that the support pattern is performing as designed. In some circumstances, the support pattern may be found to be inadequate for the rockmass and mining conditions, and in others that the support pattern is over designed.

The extent and mechanism of the rockmass failure, as well as the performance of the cablebolts, can be assessed visually and through the use of field instruments in a monitoring program.

- Observations of failed zones in the rockmass and at the ends of exposed support elements can be used to determine the rock failure mechanism, to understand why the cablebolts failed to retain the rockmass, and to provide an estimate of the load taken by the cablebolts. Detailed field notes and a series of photographs should be compiled for each working area.

- Some instruments have been developed in recent years that can augment visual observations, by "viewing" data in areas which are not physically accessible. Borehole cameras allow visual observation of structure or stress induced spalling along the walls of a borehole. Laser distance meters form the basis of equipment which, when positioned within a stope, orepass or other non-accessible mining excavation, will provide a complete 3-dimensional visualization of the current shape and size of the opening.

- Other more conventional instruments such as time domain reflectometry (T.D.R.) devices, extensometers, ground movement monitors, stress cells and strain gauges can be used to monitor the stress and deformation within the rockmass. Specialized instruments have been designed to monitor strain along a cablebolt or the load at the face end of the bolt. When used in combination, these instruments may record data that can provide a more complete understanding of the rockmass behaviour and support performance.

- Computer spreadsheets, graphing tools, and data management software allow for rapid data reduction and presentation. These tools also facilitate data analysis and design feedback.

4.2 Visual Performance Assessment

Visual observations of the state of the rockmass, the final, "as-mined" boundary of the excavation and the ends of the cablebolts exposed by rockmass failure are all important keys to gaining an understanding of the behaviour of the rockmass and the performance of the cablebolts.

When rockmass failure has occurred, try to assess:

- The extent of the rockmass failure (% of surface area and depth of failed zone).
- The appearance of the rockmass. Has there been any spalling or "onion skinning" of the rockmass? Are any open joints visible?
- The approximate size or volume of the blocks that have fallen from the stope boundaries.
- The percentage of cablebolts falling into each of the four distinct failure categories shown in Figure 4.2.1.
- The length of the cablebolt strands exposed by the failure.
- Are any plates remaining? If so, where are these plates located, and are they loose, bent or broken?

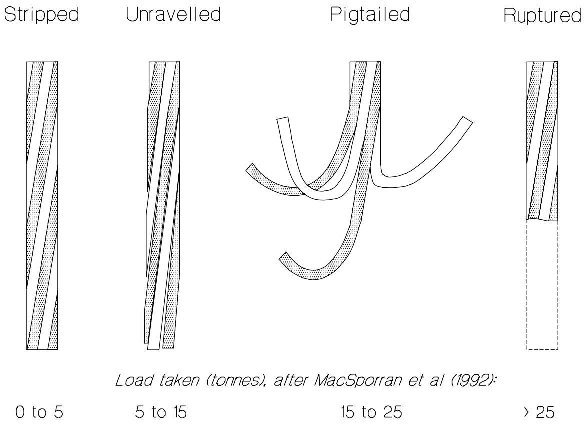


Figure 4.2.1: Visual appearance of failed cablebolts

4.2.1 Remote "Visual" Data Collection

In recent years, instruments have been developed to allow "visual observations" of the rockmass to be made in areas which are not physically accessible. These instruments include the borehole camera and the remote laser distance meter.

Both of these instruments have been tested extensively in field trials in Canada by the Noranda Technology Centre. This work has resulted in the production of readily useable versions of the instruments, which are now commercially available. A number of mine sites around the world have begun to integrate the use of the borehole camera and the remote laser distance meter into their regular monitoring operations.

Borehole Camera

The borehole camera is inserted into a borehole and the head can be rotated to view and photograph either the wall or along the length of the borehole. Cameras are available which will fit into a number of different sized boreholes. In a monitoring program, where regular readings of the instruments will be taken, a borehole for the camera log should be drilled and left open for the duration of the work. The collar of the hole should be blocked to prevent mud or cuttings from falling into the hole.

The borehole camera is being used at the Louvicourt Mine to provide valuable information about the rockmass behaviour during the initial mine design stage (Germain, 1995). Before mining of one of the first stope blocks began, a base survey was made during which the borehole wall was photographed at 0.3 metre intervals. As mining progressed, borehole camera surveys were conducted on a regular basis, allowing observation of the development and opening of fractures and joints within the rockmass.

The borehole camera surveys conducted at Ansil mine (Hutchinson, 1992) provided some of the most useful information collected during the instrumentation program, because they provided a visual picture of the location of open fractures and joints. In addition, horizons within the rockmass where the borehole walls were spalling due to high stress levels were observed with the camera.

Whenever possible, camera logs should also be made of the boreholes in which other instruments will be installed. The camera log will provide information about where pre-existing structures are located along the length of the borehole. This is particularly important for CSIRO stress cell installation (Maloney, pers. comm.), and when cablebolt strain gauges are to be used. The location of movement(s) within the rockmass can be better understood when borehole camera information is available; from the initial survey of the instrumented hole and from adjacent holes which are logged regularly with the camera.

Remote Laser Scanning Device

The remote laser scanning device is mounted on a telescoping arm that can be extended into the stope, as is shown in Figure 4.2.2, or lowered down a borehole. Once in the stope, the laser device is rotated in 3 dimensions to accurately survey the entire stope. The data is recorded directly onto a computer, from where it is easily loaded into a conventional drawing program such as AutoCad™ for data visualization and interpretation. The data that is recorded provides a fairly accurate picture of the boundary of the stope or excavation that is otherwise inaccessible for detailed observations of the rockmass.

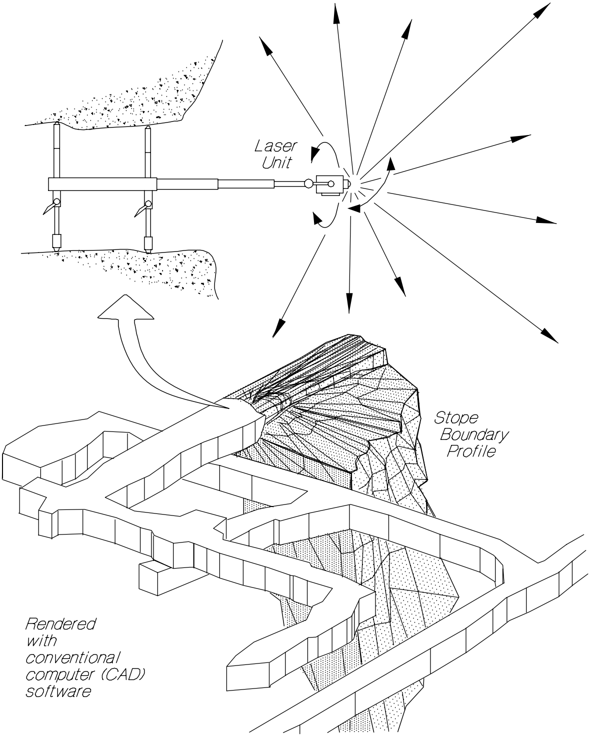


Figure 4.2.2: The remote laser distance scanning device: equipment setup and a stope boundary profile recorded at a mine site (after Miller et al., 1992)

The adaptation and use of the laser distance device at a number of mine sites is well documented by Miller et al. (1992). Since the time of that paper, the device has been tested extensively and improved, and is now commercially available.

The laser surveying device is used by the survey crew to measure the profile of every stope at Hemlo Mine (Anderson and Grebenc, 1995) and at Louvicourt mine (Germain, 1995). The data recorded provides valuable information about the source and volume of dilution due to failure of backfill and waste rock, and indicates where ore has sloughed into the stope and where unblasted ore has been left behind. The results of a laser survey in a stope which produced a large volume of waste rock dilution at Hemlo Mine is shown here. This information was used to design a more effective cablebolt pattern for the adjacent stope which produced much less dilution (Figure 1.2.2).

When using this equipment, be aware of any physical and directional limitations of the hardware: for example, it is not able to "see around corners". It is important to position the laser device far enough into the stope.

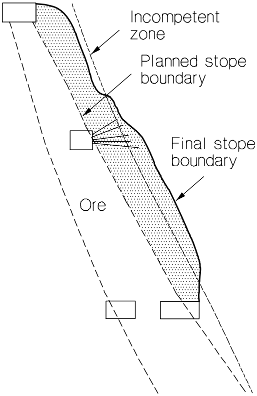


Figure 4.2.3: Dilution measured with a laser distance meter at Hemlo Golden Giant Mine (after Anderson and Grebenc, 1995)

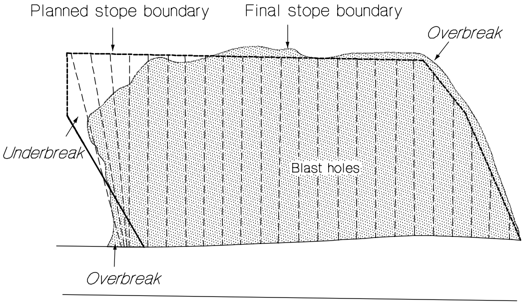


Figure 4.2.4: Final stope boundary measured at Louvicourt Mine (after Germain, 1995)

4.3 Monitoring Performance with Instruments

A well designed instrumentation program includes a variety of instruments and should be directed at answering a specific set of questions. The design of an instrumentation program aimed at assessing the performance of cablebolts in a rockmass might include consideration of some of the following points:

- What is the expected rockmass failure mechanism and which instruments will provide the information to confirm this hypothesis?

- Is the rockmass expected to undergo a stress change during the course of the monitoring program? In instances where the rock slides off the end of the cablebolts, the bond strength was inadequate. Stress change measurements can be used to determine if the failure was due to poor installation quality control, or to a stress decrease, or to a combination of these factors.

- Is the access for installing the instruments adequate? The read out end of the instrument must be safely accessible throughout the life of the program, or be outfitted with a remote read out head that can be protected against damage.

- Are the methods for protecting the equipment adequate? The instruments, wiring and any other equipment must be well protected against damage from falling rock, water, equipment and blast vibrations.

- Is there sufficient redundancy built into the instrumentation program: both in the variety of instruments and the number of each kind of instrument? If some instruments are lost, is it likely that useful information will still be collected by the remaining instruments?

- Are enough robust, lower priced instruments used in the program to give good results? Some instruments are more difficult to install (stress cells) and to interpret (stress cells and strain gauge instrumented cablebolts) or may be expensive. These instruments should only be used when the more basic monitoring of the rockmass behaviour is already assured with the simpler instruments, when the budget allows for more extensive instrumentation, and when the information they can provide is worth the cost.

- Can the purchase or rental of a data logger be justified? A data logger makes the collection and input of the data very efficient and easy, but adds capital costs to the monitoring program.

- How quickly is the information required? Immediate returns may be obtained from the data collected with instruments that take direct readings, for example the displacement recorded by an extensometer. This data can be used immediately for the prediction of stability or impending failure. The full interpretation of a lot of data from a variety of instruments will take more time.

4.3.1 The Instrument Toolbox

Instrumentation for monitoring the behaviour of the rockmass has been used for a long time in many mining operations. A good discussion of the wide variety of instruments, their configuration and their utility is provided by Dunnicliff (1988) and Franklin and Dusseault (1989).

Case histories illustrating monitoring programs have been compiled by the C.I M. (ed. Franklin, 1990) in the Mine Monitoring Manual, and documented by Goris et al. (1990, 1993), Hutchinson (1992), Kaiser and Maloney (1991), Maloney et al. (1992), Stillborg (1993), Windsor and Thompson (1993), and Windsor et al. (1987).

Rockmass Instruments

Different types of instruments record different forms of data. In general however, instruments record the strain or displacement at a point in the rockmass or on the support element. General rockmass instruments should provide information about the:

- change in the dimensions of the surface of the excavation (convergence meters),

- displacement of a point in the rockmass relative to a collar location point (ground movement monitors, T.D.R., and extensometers), and

- strain of the walls of a borehole at a specific point (stress change cells).

A single reading from one of these instruments at a specific time is of little use. Monitoring involves taking a number of readings at different times which will indicate the relative changes in the parameter measured at that particular point. When the history of the data is compared with all of the other points, it may be possible to develop a realistic picture of the behaviour of the rockmass and the support elements in the area of the study.

As with the cablebolt materials and equipment, there are many manufacturers and suppliers of rockmass and support instrumentation. There are likely to be differences in the configuration of each type of instrument and in the ease of installation and data logging. It is advisable to investigate the range of instruments available from suppliers, before selecting the equipment for use at the mine site. The instruments should be robust, accurate, simple and easy to calibrate (Douglas and Arthur, 1983).

The use of these instruments will not be discussed in detail in this chapter, but the interested reader is directed to the documents listed above for additional information.

Cablebolt Instruments

The suite of cablebolt instruments currently available on the market is much more limited than the general rockmass instruments. A number of new instruments will likely be developed within the next few years, particularly in light of the increasing use of modified geometry cablebolts in mines, for which there are no instruments currently available.

Cablebolt instruments currently available provide information about the average strain along a section of plain strand cablebolt (cablebolt spiral strain gauge). These data can be used to determine:

- whether the cablebolts are being loaded close to their breaking strength,
- if the cablebolts are providing the bond strength used in designed, or
- if the cablebolt pattern or length should be changed.

The spiral strain gauge can be of any length and will monitor the *average* strain in the cablebolt over that length. However, no specific information about the true distribution of the load applied to the cablebolt is given by the gauges.

The spiral strain gauge consists of a set of plastic sheathed wires that are wrapped around the cablebolt, following the "groove" between adjacent, individual wires, as shown in Figure 4.3.1. The wires are anchored in rubber plugs that are glued to the cablebolt. As the cablebolt deforms, the wires stretch within their plastic sheathes, and the resistance of the wire changes. The change in the wire resistance can then be related to an average load along the cablebolt. There will be some initial creep of the gauges, for up to 3 months after installation of the gauge (Choquet, 1993), that should be accounted for in the data reduction.

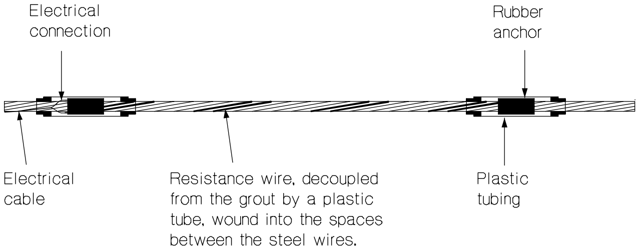


Figure 4.3.1: Configuration of a spiral strain gauge installed on a cablebolt (after Choquet and Miller, 1988)

The application of the correct, small amount of tension to the resistance wires and the complete waterproofing of the gauges are both very important steps during the installation of the gauges on the cablebolt. If these two procedures are not well done, the life of the gauges will be limited. Where available, it is advisable to purchase the gauges pre-installed at the desired locations on the cablebolt.

Due to the presence of plastic sheathed wires within the "grooves" between the individual cablebolt wires, the interaction between the cablebolt and the surrounding grout will be different than usual. It is thought that the bond strength and capacity of the cablebolt will be reduced to some extent. In addition should the grout column move past the instrumented cablebolt for any distance, damage to the wires is expected to occur. The degree of damage will depend upon the grout quality, rockmass confinement and any stress change.

A tension gauge can be located at any point along the length of the cablebolt. The influence of the local geology on the subsequent rockmass movement should be considered when designing the layout of the gauges. If possible, a borehole camera should be used to log the position and characteristics of any structures in the borehole. Knowledge of the pre-existing structural characteristics of the rockmass can be of great use when interpreting the data recorded by the gauges. Using this information, the gauges can also be positioned on the cablebolt at the location of structures that are expected to dilate and load the support.

The spiral strain gauge should be used in conjunction with at least extensometers and borehole cameras if possible. These other instruments will provide information about the rockmass failure mechanism and the resultant source and magnitude of deformations within the rockmass which are loading the cablebolt. The interpretation of the performance of the cablebolts is very difficult without rockmass deformation information.

The spiral strain gauge is described in further detail in papers by Windsor et al. (1987), Choquet and Miller (1988), and Choquet (1993). Other cablebolt and grout instruments have been developed and investigated by the Commonwealth Scientific and Industrial Research Organization (CSIRO) in Australia. These instruments are not commercially available at this time, but the interested reader is referred to the paper by Windsor (1992).

Several field investigations of cablebolt performance have included strain gauge instrumented cablebolts (Choquet, 1993; Goris et al, 1990; Hutchinson, 1992; Thibodeau, 1994; and Windsor and Worotnicki, 1986). A review of these case histories will provide the interested reader with additional information about the use and interpretation of these gauges.

4.3.2 Design of the Instrumentation Program

The selection of instruments for a monitoring program at a particular site will be governed by budget, past experience, relative cost and applicability of the instruments, orebody geometry, underground access and the program objectives.

Objective of the Instrumentation Program

The objective of the instrumentation program should be defined before starting to design the program. The objective of the instrumentation program undertaken at Ansil mine (Hutchinson, 1992; Hutchinson and Grabinsky, 1992) was to develop a better understanding of the interaction between the cablebolts and the rockmass, in order to improve the cablebolt design.

Ideally it should be possible to develop a complete understanding of the interaction between the rockmass and the cablebolt reinforcement by comparing the data recorded by different instruments. If this is the aim of the instrumentation program, then individual groups of the gauges and anchors of the various instruments should be positioned close to one another in the rockmass. At Ansil mine the instruments were installed in a diamond pattern in each stope (Figure 4 3.2), which was designed to provide complete coverage of the rockmass at the centre of the span of each stope, both in terms of spatial coverage and the type of data collected. The diamond pattern consisted of an extensometer, borehole camera hole and stress cells in the centre, within the array of cablebolts, some of which were instrumented with spiral gauges. The monitoring program is discussed in more detail in Hutchinson and Falmagne (1991) and Hutchinson (1992).

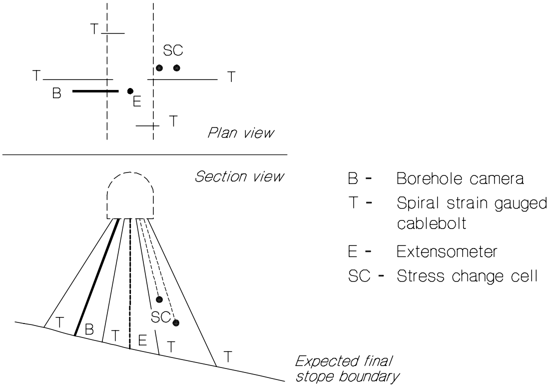


Figure 4.3.2: Basic diamond-shaped instrument pattern used at Ansil mine

Instrument Cost and Value of the Data

The relative value of the data recorded by each instrument should be assessed, based on previous experience where possible. The purchase cost of an instrument is not the only factor in this evaluation. In fact, Dunnicliff (1988) cautions against the purchase of the cheapest instrument unless it will provide the sensitivity, accuracy, and longevity required. On the other hand, if it is expensive to purchase or time consuming to install the instrument or to interpret the data, then the instrument should be used with caution.

In the early stages of an instrumentation program at a mine site, numerous inexpensive instruments should be used, if they will provide the information required. More expensive instruments could then be used when they are required to answer a specific question arising from the previous instrumentation program.

A ranking of instruments, with respect to the purchase cost, the ease of installation, and the time required for collecting the data, data reduction and data interpretation, can be a very useful tool during the design of the instrumentation program. A ranking based on experience at Ansil mine is shown in Table 4.3.1, where a data logger collected the extensometer, stress cell and strain gauge data. A similar ranking should be developed at each site as experience is gained with different instruments.

The lowest ratings in Table 4.3.1 are for the borehole camera and the extensometers. If the instrumentation program budget is small, or time for analysis is limited, then there should be a heavy emphasis on the borehole camera and extensometer, which are easily installed, read and interpreted. Where instrumented cablebolts are used, more time has to be spent interpreting and presenting the data. Stress change cells require a significant amount of time; for installation and then reducing, presenting and interpreting the data. The cost of this time should be considered when budgeting the instrumentation program.

The purchase of a data logging device should be evaluated. While the initial purchase cost may be high, a data logger records data reliably and as frequently as required. Some of the purchase cost of a data logger is offset by the substantial reduction in the time spent by personnel reading and inputting data. The frequency of data collection can be set at any level with a data logger. The data logger also continues to collect data at times when personnel cannot access the instrumentation site, such as during production blasts. The data logger selected for the site must be robust enough to survive the adverse conditions (dust, moisture and vibrations) found in most underground mines.

Careful field observations, including photographs and detailed notes, can provide extremely useful information about the deformation and failure of the rockmass and about the performance of the cablebolts.

Table 4.3.1: Ranking of the instruments used at Ansil mine (after Hutchinson, 1992)

	Borehole camera	*Extenso-meter*	*Stress Cell*	*Spiral gauge*
Unit Cost	1*	2	2	2
Drilling Cost	2	1	3	1
Installation difficulty**	0	1 to 2	3	1
Time for collecting data	2	1***	1***	1***
Data reduction difficulty	0	1	3	2
Data presentation difficulty	2	1	3	1
Interpretation difficulty	1	1	3	2
Risk of inaccuracy or misinterpretation	0	1	2	3
Ranking****	8	9 to 10	20	13

* These costs are for rental of the borehole camera and data logger, and do not consider the purchase price.

** This ranking assumes that there is easy access to the collar of the camera holes. The ranking for an extensometer without remote readout head is 1 and with remote readout head is 2.

*** Data collected with a data logger.

****Increasing ranking indicates greater difficulty or higher cost.

Geometry of the Orebody and Access

The overall geometry of the orebody should be considered in the design of the instrumentation program:

- If the boundary of the orebody, and the structure and geology of the rockmass are fairly regular, then the data recorded by a particular instrument is likely to be representative of the behaviour of the adjacent rockmass, and as such can be compared with data collected by adjacent instruments which monitor some other aspect of the behaviour.
- Where the geometry, geology or structure of the rockmass is very irregular and three-dimensional, the data recorded at a point in space is not easily related to that recorded by a nearby instrument. In this case, more instruments are required over a smaller area to develop an understanding of the local rockmass behaviour. A picture of the regional rockmass behaviour will be even more difficult to obtain.

The design of the instrumentation pattern will also depend upon the underground access available. Hangingwall, footwall and sill drifts provide opportunities to install instruments in a number of different patterns.

Redundancy

The instrumentation program must also include some "redundant" instruments. All monitoring programs, no matter how well designed, installed or protected will suffer the loss of some gauges or instruments over the life of the program. Therefore the number of instruments considered adequate to collect the required data should be increased by at least 10 to 20%.

During the life of the Ansil monitoring program, 28% of the cablebolt spiral strain gauges and 14% of the extensometer anchors were lost after providing little information, due to malfunctioning equipment, operational problems in which wires were irreparably cut, and due to an unexpected, sudden failure of the rockmass in one area (Hutchinson and Falmagne, 1991).

Remote Readout

The instruments should be safely and easily accessible throughout the life of the monitoring program, or else be outfitted with remote readout heads.

Remote readout heads may be required to convert the instrumentation output into a form that can be recorded by a data logger. This is the case with some mechanical extensometers for which physical displacement is converted into electrical signals through the use of potentiometers or voltage transducers. The heads should be used with caution however, because they can be expensive, inaccurate, subject to excessive scatter, difficult to install, or may not be designed to survive the harsh environmental conditions often encountered in mines.

Protection

The instruments, wires and data logger must be protected as much as possible from damage. Moving equipment can cut or pull down wires and damage the instruments. Blasting can vibrate instruments excessively, putting them out of alignment or calibration, or "fly-rock" can damage the instruments or data logger.

The instruments should be installed from a remote and protected area if possible, such as from a remote drift or an embayment in the drift wall or from a "cut-out" in the rock surface. The instruments and wires should be protected from damage by very visible signing and physical barricades such as a fence or a plate over the head of the instrument. Wires should be protected from damage and vibration: at Ansil mine the wires were placed inside old steel water pipes which were bolted to the drift back. Slots were cut along the length of the pipes so that the wires could be inserted easily. The point at which the wires entered the pipes was protected by short lengths of old air hose slit down one side so that it could be slipped over the wire (Hutchinson and Falmagne, 1991). Wires could also be covered with a protective layer of shotcrete.

4.3.3 Installation of the Instruments

The correct installation of the instruments is very important. Problems encountered during installation or poor procedures are often the source of subsequent failures of the equipment. Some points to consider when embarking on the installation process are:

- Most instruments should be supplied with an installation manual or procedures. Read the manual first to gain an understanding of the procedures involved.

- Work through the installation procedures on surface, in a trial run before going underground. Check that the procedures make sense, and assemble all of the tools and equipment required. When calibration of the instruments is required, take the readings at surface if possible.

- Take a pre-installation reading of all electrical readouts.

- Measure the length of all holes that are to receive instruments.

- Keep a detailed record of all observations made during the installation of the instruments, including any problems encountered. Photos of the rockmass, the instrument site and the procedure used may be helpful later to evaluate any problems or subsequent questions. Record a post-installation "bench mark" reading for each instrument.

4.3.4 Data Recording

Data should be measured and recorded frequently just after the installation of the instrument when there are no disturbing influences such as nearby blasting, to determine if there is any "creep" or "noise" of the data readings.

Readings from all of the instruments should be taken just prior to and soon after any events that are expected to result in changes in the stress or displacement in the rockmass and support elements.

The data should be plotted in its raw form as soon as possible after the reading is taken to observe general trends in the data. Take readings as frequently as required to define trends (Figure 4.3.3). It is never possible to take too many readings: the data points in a long run of similar results can be discarded, but missing data points that should have been recorded during an important event cannot be retrieved after the fact.

Any readings that appear to be anomalous should be taken again as soon as possible to determine if a mistake has been made or to confirm the value recorded.

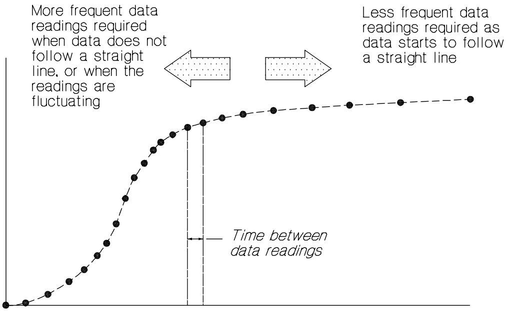


Figure 4.3.3: Frequency of readings required to fully define the data set

4.3.5 Data Reduction and Plotting

The data should be reduced and plotted as soon as possible after the readings are taken. Some suggestions for data reduction and plotting are:

- Convert the data to the units of interest (mm, MPa, tonnes). The conversion factors supplied with the equipment should be checked by calibration tests conducted at the mine site. It is convenient to record the data in a spreadsheet, where the conversion calculations and plotting parameters are set up.

- Normalize any calculated data with respect to critical parameters that are unknown or in which there is little confidence in the true value. For example the modulus, *E*, in stress change cell data calculations is not likely to be well known, but is a direct multiplier of the data. Note this assumption and the expected value of the factor in the spreadsheet and on the plot.

- Plot the data on whatever axes are the most convenient and easy to interpret. The scale of the axes should be established at the start of the monitoring program, so that the data is always viewed in a similar manner. For example, average loads recorded by spiral strain gauges on cablebolts should be plotted on axes scaled to the rupture load of the cablebolt. The data points are usually plotted with respect to time on the x-axis. Figure 4.3.4 shows some other ways to plot data that might be easier to interpret in certain situations.

- Clearly mark any important data on the plots, such as the position of extensometer points, gauges or cells with respect to the expected stope boundary, the length of spiral strain gauges on the cablebolts etc. Use a specific symbol to indicate the time of important events, such as blasts, on the plots.

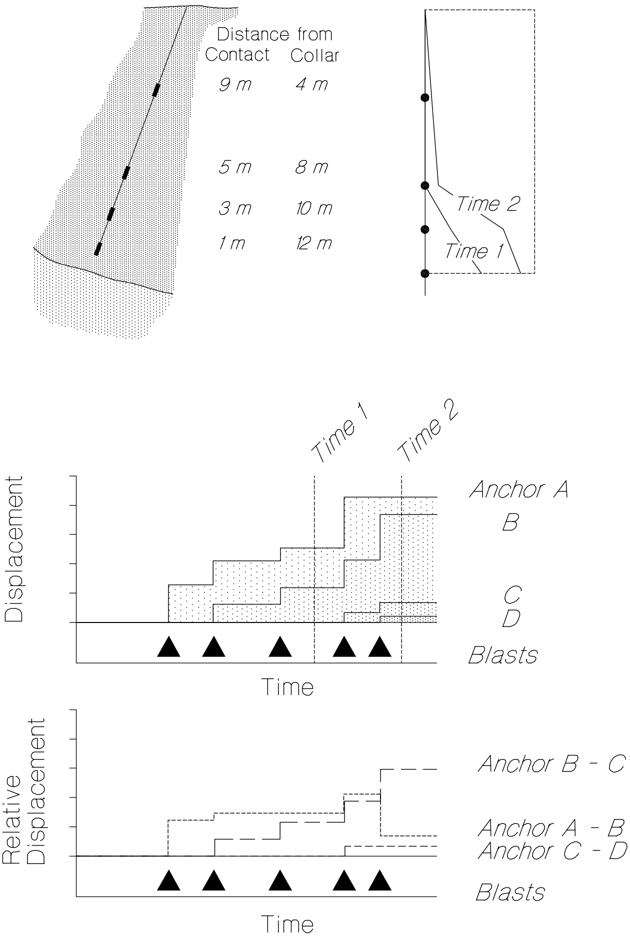


Figure 4.3.4: Different methods of plotting the data can assist in understanding and interpreting the results

4.3.6 Data Visualization and Interpretation

The interpretation of the data collected from instruments such as extensometers, borehole camera logs, stress change cells and cablebolt spiral gauges is an iterative process, that evolves as more data is collected, as the rockmass failure mode starts to become clearly understood, and as mining progresses. This process is aided by effective and timely plotting of the data and visualization of the rockmass and cablebolt performance.

In general, the changes in extensometer data with time indicate the magnitude and rate of movement of the rockmass, and the approximate source of the displacements. Borehole camera logs provide immediate visual evidence of joint opening, displacement and shear, and borehole spalling in high stress conditions. The cablebolt spiral strain gauges and the stress change cells indicate the approximate magnitude of the cablebolt load and the approximate stress change in the rockmass respectively.

An example of a simple, relatively rapid interpretation of instrumentation data is given in Figure 4 3.5, which shows several possible rockmass deformations which could be recorded by extensometers or strain gauged cablebolts.

The interpretation of the results of the instrumentation program for an evaluation of the interaction between the cablebolts and the rockmass is more difficult and time consuming. The data recorded by the stress change cells will indicate the magnitude and orientation of stress changes. Once joints near the stress cells open up, the data recorded by the cells will become fixed in the direction normal to the joints and the interpretation of the data will eventually become invalid as joints and fractures propagate. The borehole camera should be used to log the hole(s), both while the rock is still intact, and frequently thereafter as mining causes the rockmass to deform and the joints to dilate. The extensometer and instrumented cablebolts will record meaningful non-zero data once the individual rockmass blocks begin to move.

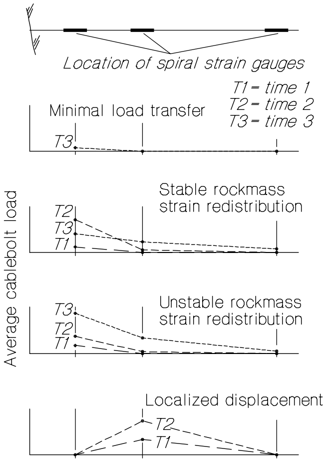


Figure 4.3.5: Evaluation of rockmass failure modes from the data distributed along a hole

Data Visualization

The most difficult component of the analysis of the instrumentation data is visualizing and understanding the large volumes of diverse data collected. The geometry of the orebody and the location of the instruments at a particular site may make the visualisation fairly simple. However, the compilation of the data recorded at Ansil mine proved to be a difficult task, due to the three-dimensional shape of the hangingwall, and the large number of instruments (Hutchinson, 1992). The best plotting method that was developed is shown in Figure 4.3.6. The magnitude of displacement (extensometers) or cablebolt load (spiral gauges) is indicated by the size of spheres centred on the position of the gauges. Figure 4 3.7 shows the most illustrative way to plot stress change data. These diagrams were produced for all events of significance at Ansil mine.

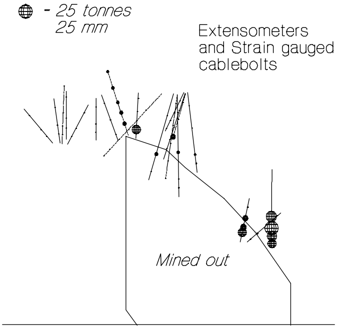


Figure 4.3.6: Data recorded by the instruments in the hangingwall of the upper horizon stopes at Ansil mine

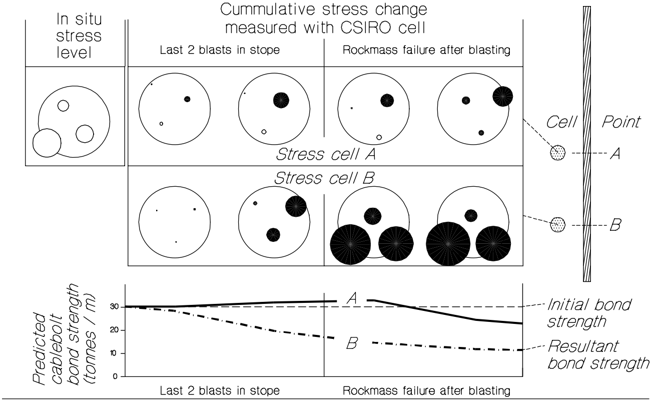


Figure 4.3.7: Prediction of cablebolt bond strength (Section 2.6) in a rockmass undergoing stress change as measured by CSIRO HI cells. The magnitude of the stress change is represented by the diameter of the circle, and the orientation of the stress change is plotted as trend/plunge on the stereonet. (after Hutchinson and Diederichs, 1995)

Data Interpretation

The interpretation of the results of the instrumentation program should start with some idea of the rockmass failure mechanism. Three possible modes of failure are shown in Figure 4.3.8. For each of these failure mechanisms, the instruments will record different data, thereby either confirming the original hypothesis about the rockmass failure or leading to some new understanding of the rockmass behaviour.

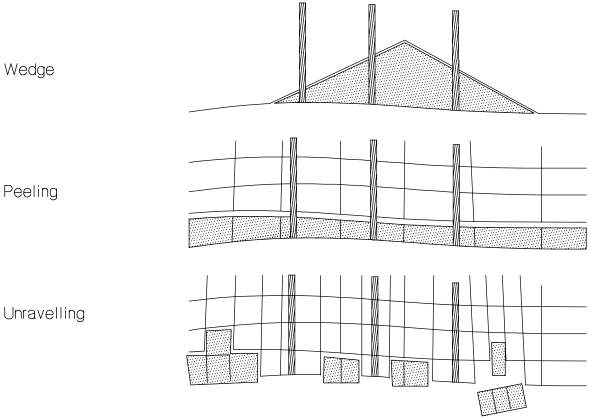


Figure 4.3.8: Potential rockmass failure modes to be considered in the interpretation of data collected from an instrumentation program

Examples of the data that could be recorded by a suite of instruments in a rockmass failing by wedge failure, by peeling failure or by unravelling failure are displayed in Figures 4.3.9 and 4.3.10. In these drawings, the tangential stress change, $\delta \sigma_t$, is oriented parallel to the excavation boundary, while $\delta \sigma_r$, the radial stress change, is perpendicular to the excavation boundary. A black, filled zone indicates a decrease in compressive stress, while a white zone indicates an increase in compressive stress. Further details regarding the influence of the formation of an excavation and possible subsequent rockmass failure on the stress field around the opening are given by Kaiser et al (1992) and Diederichs et al (1993).

Extensometer data alone will provide some information about the rockmass failure mechanism (Figure 4.3.11).

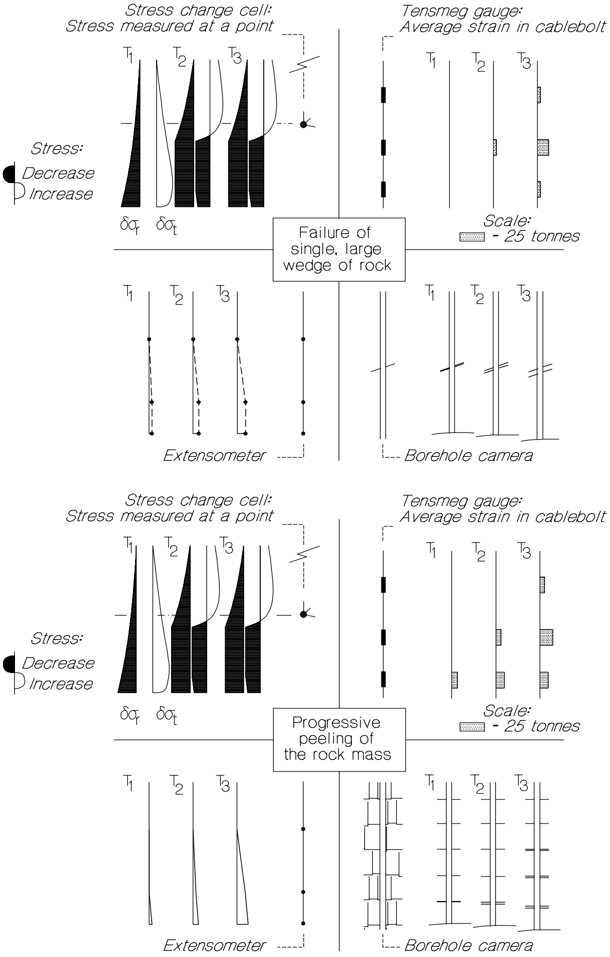


Figure 4.3.9: Hypothetical data recorded by rockmass and cablebolt instruments for different rockmass failure modes: wedge and peeling failure

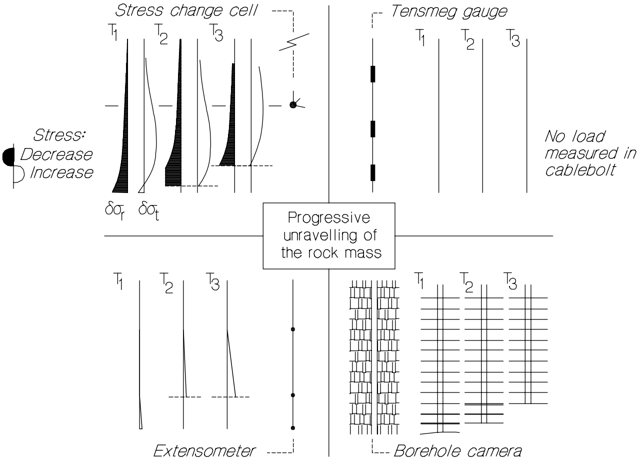


Figure 4.3.10: Hypothetical data recorded by rockmass and cablebolt instruments for different rockmass failure modes: unravelling failure

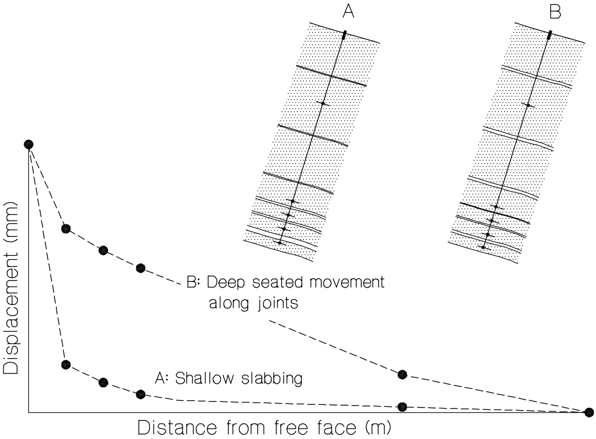


Figure 4.3.11: Interpretation of data recorded by multi-point extensometers (after Hansmire, 1978)

4.4 Instrumentation and Failure Analysis

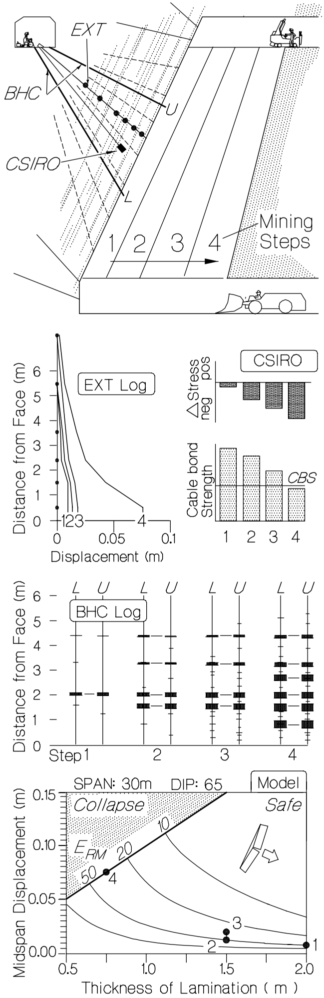


Figure 4.4.1: Instrumentation Example

Consider the example stope in Figure 4.4.1 (top). The inclined, 30m high hangingwall is in a foliated but initially unparted schist. Cablebolts are installed from a auxiliary drive as shown. A six point extensometer (*EXT*), a stress change cell (*CSIRO*) and two borehole camera holes (*BHC*) are laid out to monitor the hangingwall. Two *BHC* logs are required to assess the persistence of large scale delamination partings. The logs for the various instruments for 4 mining stages (1,2,3,4) are shown (middle).

Information regarding face displacement (*EXT* data) and lamination thickness (*BHC* data) are use to calibrate a *Model* Voussoir beam (bottom). This analysis is described in Section 2.18.12. It can be seen that the initial (back calculated) rockmass modulus is approximately 50 GPa. The modulus drops at stage 3 due to blast damage and an increase in small partings, although the large scale delamination does not change. New lamination partings occur at stage 4. It can be seen from the Voussoir model that the hangingwall is reaching critical displacement and failure is imminent.

The *CSIRO* cells give information about stress change, Δ*stress*, across the cables. From these readings the change in cablebolt bond strength is calculated (Section 2.6.1). By stage 4 the bond strength has dropped below the critical value (*CBS*) and the rock begins to slide off the cables (see Figure 2.6.3). Modified geometry cablebolts (Section 2.9) are therefore recommended for future stopes to prevent beam failure.

4.5 Experience: The Best Design Tool

It seems fitting to conclude a discussion of the verification toolbox with a mention of perhaps the least expensive and most reliable design, verification and back analysis tool a mine engineer can have - *his or her own experience*. Since stopes and mine drifts are typically laid out in often repetitive patterns, experience gained in every metre of tunnel and with every tonne of ore extracted is directly applicable to future design (Parker, 1973).

The first step in this process, the maintenance a tunnel and stope geomechanics database, is critical. Information on successes as well as failures can be processed in numerous ways to optimize design. For example, a simple plot of intersection span with respect to percentage of failures in different rockmasses can serve to prevent the under-design and over-design of future support systems, thereby increasing safety and reducing unnecessary costs.

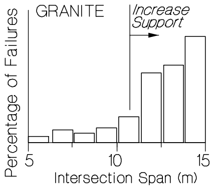


Figure 4.5.1: Data example

A computer database is a convenient method of maintaining this data (Yazici and Kaiser, 1992). The information can be kept in a spreadsheet if desired. For every excavation (drift, intersection, stope) a list of rockmass and environment information, geometric information and performance data is required for future analysis. Relationships between rock type, stress level, structural control, local mine stiffness and failure potential (Kaiser et al , 1992), excavation span and orientation, mining sequence and support type can be analyzed with respect to recorded excavation performance (stability, service life, dilution) to determine critical design parameters in the future. The following is a suggested data list:

"*Cause*"
Rockmass: (Section 2.14)
    type, strength, structure (joint sets)
    quality (*Q, RMR, Q, N'*)
    water, moisture
    failure potential, local mine stiffness (Kaiser et al., 1992)
Stress: (Section 2.13) overstressed/relaxed
    maximum induced stress, pillar stress, depth, stress change (+/-)
Excavation: shape, span, hydraulic radius, type (intersection, drift, stope)
Support: type, age, *W:C*
Environment: calm/heavy blasting/seismic, extraction ratio

"*Effect*"
Excavation: stable/failed/damaged
Failure: Type - slab/wedge/ravelling/cave/burst
    failure tonnage, dilution (monitor as in Section 4.2.1)
Support: adequate/damaged/corroded/broken/stripped(cables)

4.6 Application of Performance Assessment Results to Cablebolt Design

The results of the performance assessment or verification process can lead to several conclusions:

1) the rockmass is well supported, but the cablebolts are over designed and therefore more expensive than necessary,

2) the cablebolts are well designed and well installed, or

3) the rockmass is not well supported because the cablebolt design and/or installation procedure is inadequate, and should be re-evaluated.

Some questions that should be answered in the case of the third result are:

- Can the installation procedure or equipment be improved, thereby improving the capacity of the installed cablebolts?

- Was the capacity and the pattern of cablebolts sufficient to retain the rockmass? What is the potential rockmass failure mode? Is the best cablebolt type being used for the failure mode and degree of stress change observed? Are surface restraint elements required?

- Can anything in the mining environment be realistically changed to improve the cablebolt performance? Would the installation of the cablebolts at a different stage in the mining sequence reduce the effects of destressing at the cablebolt locations? Are additional drifts required to provide more complete access for cablebolt installation?

Monitoring programs can be very time consuming and frustrating. The installation of the instruments, and then the collection, reduction and interpretation of the data can lead to ambiguous results which only provide a limited improvement in understanding of the rockmass failure mode or the cablebolt performance. On the other hand, when monitoring programs do work as designed, and when the instruments record meaningful data throughout the mining of adjacent excavations, the results can be extremely useful, leading to improved design of the cablebolt patterns and a potentially huge savings in money spent on cablebolt support.

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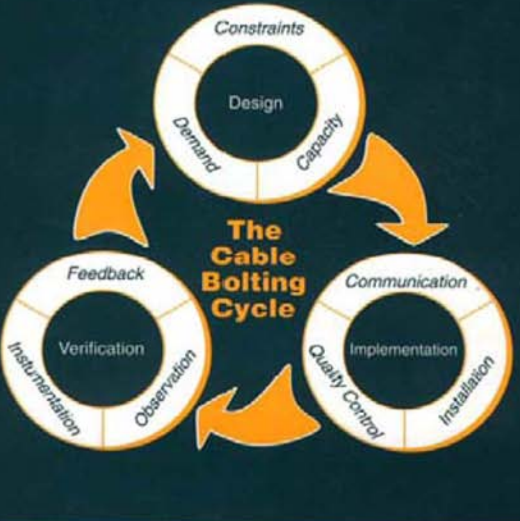
Cablebolts are flexible tendons composed of multi-wire strand which are normally installed and grouted in drill-holes at regular spacings to provide reinforcement and support of excavations. This comprehensive handbook covers virtually all aspects of cablebolting, for support of underground excavations in rock, from theory to implementation with an emphasis on (but not restricted to) applications in the mining industry. Recent innovations in cablebolting are presented and the current body of international research is summarized in the context of cablebolt type selection and support system design. Installation and quality control procedures are outlined along with suggestions for crew training and management. Practical techniques for support performance assessment and design verification round out the cablebolting cycle. This book is an essential guide for the rock mechanics engineer or mining ground control specialist and is an excellent foundation reference for researchers and developers in the field.



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