A NEW ROCK BOLT DESIGN CRITERION AND KNOWLEDGE-BASED EXPERT SYSTEM FOR STRATIFIED ROOF

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Dissertation submitted to the Faculty of the
Virginia Polytechnic Institute and State University
in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

in

MINING ENGINEERING

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July 1, 1999

Blacksburg, Virginia

Keywords: Bolt, Roof Support, Ground Control, FLAC model, Expert System, Knowledge

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ABSTRACT

Since its development in the 1920s, bolting has become the most dominant support method in underground construction. However, because of the geological environment, the design process for roof bolt systems is an art rather than a science. To quantify the selection of bolting systems a MSBT (minimum solid beam thickness) approach was developed. The ultimate goal of this bolt design paradigm was achieved by optimizing bolt length, bolt density, and bolt pretension during installation. The impact of the number of strata layers within bolting range and pretension applied to bolts upon the stability of an opening was investigated using FLAC model. Four statistical models for predicting optimum bolt supports using a minimum solid beam thickness were established, and based on these results, a design criterion was proposed.

To meet support needs in various geological and geotechnical settings, a variety of bolt types have been developed. The installation of such bolt-based support systems is often complex and specialized, and thus imposes a challenge for engineers to identify the specific cause and to take appropriate remedial measures once problems arise. To solve these problems, a knowledge-based expert system (KBES) has been developed. The knowledge base includes the data accumulated from years of laboratory and field investigations conducted by the Mine Safety and Health Administration of the US Department of Labor. A user-friendly Windows-based program was implemented using KAPPA environment. After identifying the problem, the KBES searches its knowledge base and reasons out the most likely, secondary, and other potential causes, then provides solutions according to users' input.

The results of this research are validated and demonstrated using case studies.

ACKNOWLEDGMENTS

I would like to express my deepest gratitude to my advisor, Dr. Christopher Haycocks, for his invaluable guidance and direction and kind encouragement throughout my graduate study.

Also, I extend sincere thanks to the other members of my advisory committee: Dr. Michael Karmis, Dr. Gerald Luttrell, Dr. Gregory Adel, and Dr. Gavin Faulkner, and to Dr. Stanley Suboleski and Dr. John Wilson, for their constructive suggestions and criticism.

A special thanks is given to Bob Canter for his great contribution to the final correction of grammatical and spelling errors.

Forever, I am indebted to my family members — my father, mother, and my brothers and sisters, especially my beloved wife, son, and daughter. Without their unselfish love and generous support, I would never reach this point.

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CHAPTER 1
INTRODUCTION

1. STATEMENT OF PROBLEM

For centuries, the support methods for underground structures, such as props and cribs, were external and passive. Since the first use of primitive slot-and-wedge rock bolts in 1927 and the proposed use of rock bolting as a systematic method for weak roof support by Weigel in 1943, rock bolting has become the most important support system in mining engineering. Rock bolting is more economic than other methods because it saves material and manpower consumption. Most important of all, rock bolting is more effective and efficient because it is an active support method, utilizing the rock to support itself by applying internal reinforcing stresses. Furthermore, rock bolting can be satisfactorily used to meet a variety of geological conditions and various support requirements.

In terms of the number of fatal accidents per million-tonnage associated with roof failure and the amount of underground coal production in the United States under bolted roof, it is apparent that bolting technology has advanced tremendously over the past 50 years. Yet despite these successful applications and great research efforts, the mechanisms of bolting technology are not fully understood. A rational basis for all bolting system designs has not yet been achieved. In fact, the process of bolting system design remains an art rather than a science since most decisions are made based on previous experiences. To ensure the success of roof bolting applications, support systems are thus usually overdesigned. The ultimate goal of roof bolting design is to provide support most efficiently with minimum cost, and this goal can only be achieved by optimizing bolt length, bolt density, and the tension applied to bolts on installation.

To meet specific support requirements and unique geological settings, numerous bolt types have been developed. However, when problems with bolt support systems are encountered, it is very difficult and requires considerable expertise to determine whether the problems are caused by geological conditions, poor installation practices, or malfunctioning supports. Quick and correct diagnosis of the causes will enable the engineer to take appropriate remedial measures to prevent the same problem from repeating itself, thus minimizing future potential ground control problems.

2. RESEARCH SCOPE AND METHODOLOGY

In the United States, most coal seams are deposited horizontally or with a very small dip angle. The stability of entries is related to bedding plane spacing in the immediate roof strata and the frictional forces between the layers. The extra force produced by bolting, resulting from either the vertical movement of immediate roof or pretension applied to bolts on installation, helps limit horizontal movement along the bedding plane and reinforce the beam building effect.

According to pressure arching theory, a de-pressured zone directly above the opening will be created due to the stress redistribution after an opening is excavated. Under gravitational forces, the strata within this de-pressured zone sag with differential magnitude. Bed separation can occur in the lower part of the zone. The major function of support systems is to carry the weight of the strata in the de-pressured zone, thus maintaining the stability. In laminated immediate roof, bolting creates a beam building effect. The bolted beam is supposed to be strong enough to carry its own weight, as well as the strata weight of the de-pressured zone. The material properties of the immediate roof and the overburden thickness determine the minimum thickness of bolted beam.

Therefore, this research focuses on:

1. The exploration of the impact of bedding plane spacing and tension applied to bolts on installation upon the stability of an opening;
2. The development of numerical models for predicting the minimum solid beam thickness, which in turn help optimize the bolt length; and

3. The implementation of a knowledge-based expert system for trouble-shooting roof support systems.

The objectives of this research are twofold: to provide guidelines for optimal design of bolting support systems and the methodology for determining the minimum bolt length, and to provide a tool to simplify the process of trouble-shooting a particular bolting system.

To accomplish the goals of this research, numerical models are created using FLAC, a program designed for geotechnical and mining simulation. Experimental results are mostly presented using figures generated by EXCEL and ORIGIN. Statistical models are built using EXCEL and SAS. KAPPA-PC is chosen as the development environment for implementing the knowledge-based expert system.

CHAPTER 2
LITERATURE REVIEW

1. EVOLVEMENT OF ROOF BOLTING

Roof failure has long been the primary concern in all kinds of underground mines. A large portion of fatal accidents are associated with roof failure. Numerous efforts were made to develop better support systems and to improve rock stability. However, for centuries, all support systems were passive and external. They were floor-to-roof supports, such as timber props or cribs. These types of supports require a great amount of timber and constant maintenance; worst of all, they are not very effective in controlling roof stability. In 1927, a metal mine in the U.S. began using a new support technology: very primitive steel slot-and-wedge rock bolts (Bolstad et al., 1983). This was the first time that internal reinforcing stresses were applied to roof strata, making the support system active. At that time, the idea perhaps came from the simple fact a bolt might bind rocks as a nail bound two pieces of boards. This was a revolutionary technology of underground ground control. In 1943, Weigel, in the *Engineering and Mining Journal*, proposed the basic concepts of roof bolting as a systematic method to support weak roofs. Some of his ideas about roof bolting are still the foundations of modern bolting theories and application guidelines. These are (Weigel, 1943):

- Support weakened rock below the natural arch line;
- Bolt weak, thin strata together to create thicker, stronger strata; and
- Bolt early in the mining cycle.

In an attempt to reduce the number of accidents caused by roof falls, the U.S. Bureau of Mines (USBM) advocated the use of roof bolting technology in 1947. Realizing its effectiveness, more than 200 mines throughout the U.S. employed this new roof support method in less than two years. By 1952 annual roof bolt consumption had reached 25 million bolts. By 1968, 55 million bolts were used annually by 912 coal mines alone, and 60% of coal production took place under bolted roofs. In the 1970’s, a rapid increase in the use of roof bolts was triggered by the 1969 U.S. Coal Mine Health and Safety Act. This Act required that roofs and ribs of all active underground roadway, travelways, and working places be supported or otherwise adequately controlled to protect persons from fall of roof or ribs. As a result, most entries in underground coal mines were supported with roof bolts. After roof bolting was accepted and widely used in the coal mining industry, there was a considerable reduction in accidents and at the same time a large increase in production. In 1984, the USBM estimated that about 120 million roof bolts were used and over 90% of underground coal production took place under bolted roof (Bieniawski, 1987). Figure 2.1.1 shows the increasing use of roof bolts in coal mining industry.

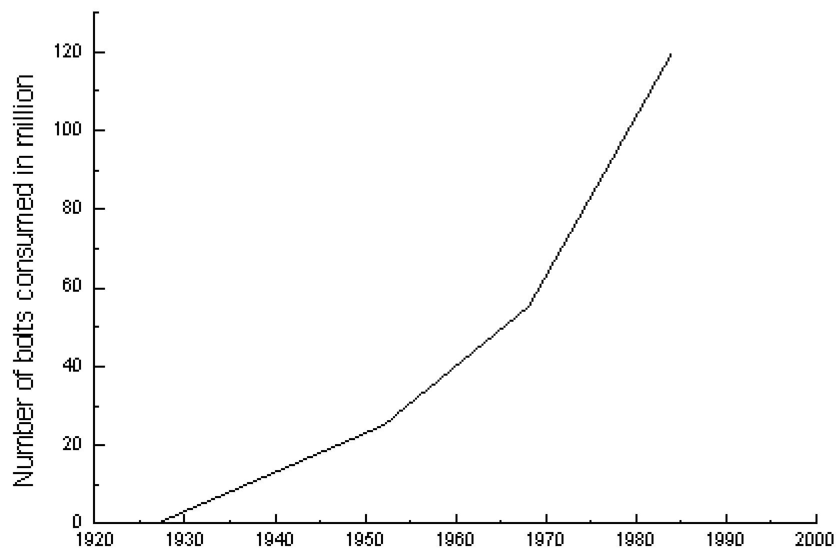


Figure 2.1.1 Increasing trend of rock bolting

Roof bolting gained so much popularity so quickly due not only to advocacy by the U.S. government, but mainly to its effectiveness of ground control and cost reduction. In addition, some advantages of roof bolting over other supporting systems also significantly enhanced its application. These advantages include:

- Reducing storage and material handling requirements;
- Decreasing the size of the opening that is needed to achieve the same given clearance;

- Preventing any appreciable roof deformations by quick installation after opening is excavated;
- Improving ventilation by lowering the resistance to the air via elimination of obstructions, such as cribs, posts, and girders;
- Providing greater freedom for trackless vehicles without risk of dislodging supports;
- Providing natural supports for hanging pipes, tubes, and electrical cables;

Today, rock bolting not only is widely used in underground coal mines to support entries primarily and secondarily, but also finds applications in surface mining, hard-rock mining, tunneling, civil engineering, and almost everywhere ground stability is involved.

2. BOLTING THEORIES:

In general, rock bolting is very effective in a variety of geological and geotechnical conditions. The main function of roof bolting is to bind together stratified or broken rocks such as sedimentary rocks containing bedding planes, rocks consisting of natural joints and fractures, or rocks with artificial fractures and cracks caused by the use of explosives (Peng 1984). The theories used to explain the bolting mechanisms vary from place to place and sometimes are elusive. However, it is broadly believed that bolt-binding effects are accomplished by one or a combination of the following three basic mechanisms: suspension, beam building, and keying.

2.1 SUSPENSION

Whenever an underground opening is made, the strata directly overhead tend to sag. If not properly and adequately supported in time, the laminated immediate roof could separate from the main roof and fall out. Roof bolts, in such situations, anchor the immediate roof to the self-supporting main roof by the tension applied to the bolts. In some instances, it appears that the immediate roof is suspended from the main roof by the bolts, as shown in Figure 2.2.1, or weak strata are suspended from stable strata, as shown

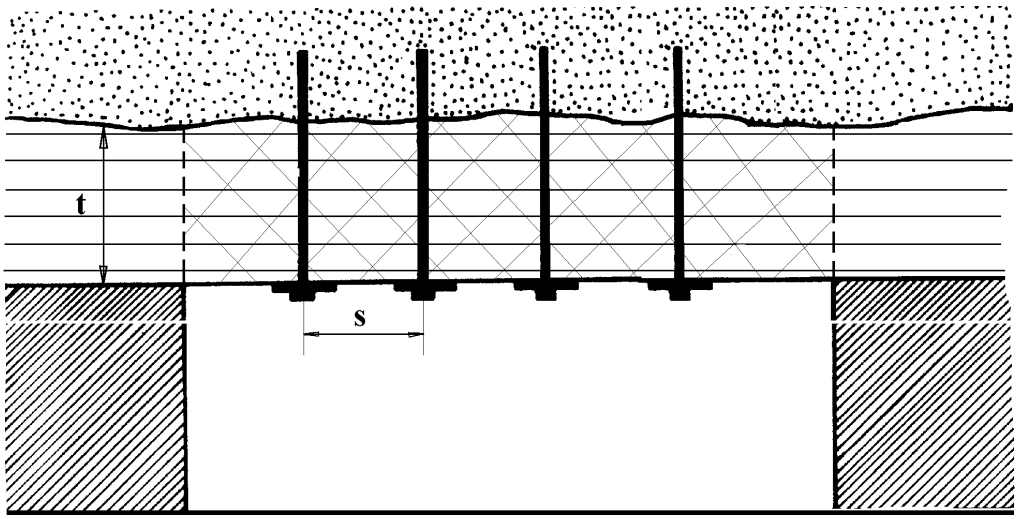


Figure 2.2.1 Suspension effect of roof bolting

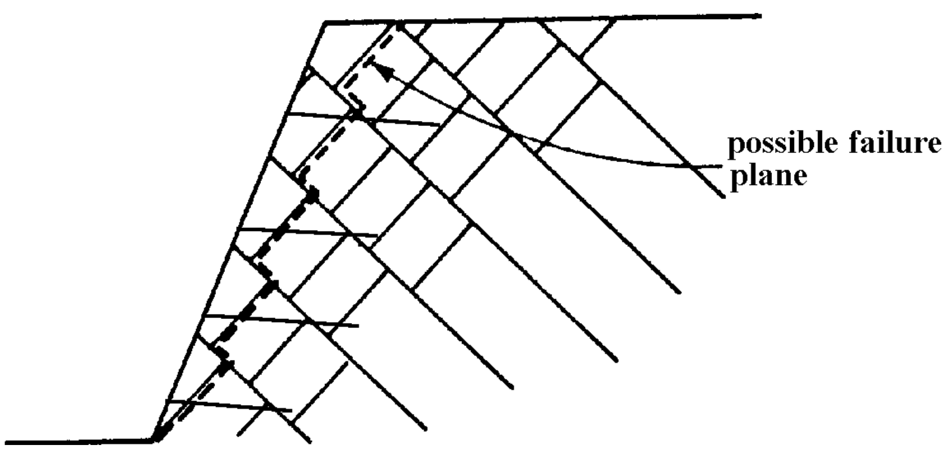


Figure 2.2.2 Partial suspension effect in rock slope reinforcement

in Figure 2.2.2. The bolts have to carry the dead weight of the strata between bolt heads and anchors. For the first case, the load carried by each bolt can be calculated as (Peng, 1984):

$$P = \frac{wtBL}{(n_1 + 1)(n_2 + 1)} \qquad \text{Equation 2.2.1}$$

where

w = Unit weight of the immediate roof;

t = Thickness of the immediate roof;

B = Roof span (i.e. entry width);

L = Length of immediate roof;

n_1 = Number of rows of bolts in length L;

n_2 = Number of bolts per row.

This equation holds only if the immediate roof would completely separate from the main roof such that it is suspended entirely by the bolts, and the portion of weight of the immediate roof supported by the abutments on both sides of the opening is ignored. Therefore, this equation estimates the upper limit of load a bolt could bear while achieving the suspension effect.

In the second case mentioned above, the estimation of the load each bolt must carry is more complex. It involves identifying a possible failure plane and then both the bolts tensile and shear stresses. As shown in Figure 2.2.3, the failure plane has an angle of α and the unstable block has a total weight of *P*. The block will remain stable if the shear force along the possible failure plane does not exceed the frictional force. To ensure stability, bolts must provide sufficient axial force to increase shear force. Suppose that a safety factor *SF* is required, then (Biron and Arioglu, 1983):

$$SF = \frac{R_s + R_1 + R_2}{T_\alpha} \qquad \text{Equation 2.2 2}$$

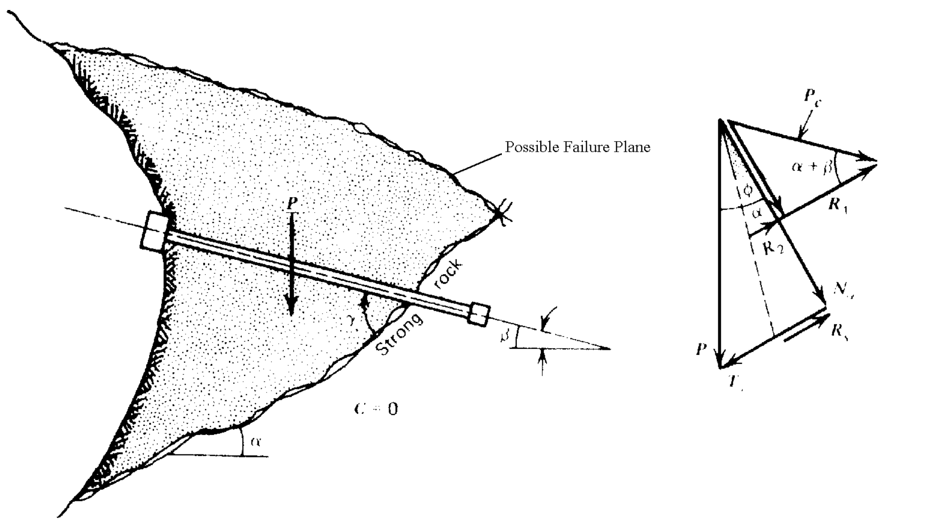


Figure 2.2.3 Carrying capacity of rock bolt for suspension (Biron and Arioglu, 1982)

where

R_s = Friction force, $R_s = P\cos\alpha\tan\beta$;

R_1 = $P_c\cos(\alpha + \beta)$;

R_2 = $P_c\sin(\alpha + \beta)\tan\phi$;

T_α = $P\sin\alpha$;

P = Dead weight of the block;

α = Angle of the possible failure plane;

β = Angle of the bolt;

ϕ = Friction angle along the possible failure plane;

P_c = Axial force given to the bolt.

Equation 2.2.2 then can be written as:

$$SF = \frac{P\cos\alpha\tan\phi + P_c[\cos(\alpha+\beta) + \sin(\alpha+\beta)\tan\phi]}{P\sin\alpha} \qquad \text{Equation 2.2.3}$$

The required axial force for the bolt to maintain stability with a safety factor, *SF*, can be computed as:

$$P_c = \frac{P(SF \sin\alpha - \cos\alpha \tan\phi)}{\cos(\alpha+\beta) + \sin(\alpha+\beta)\tan\phi} \qquad \text{Equation 2.2.4}$$

In order to create a suspension effect and stabilize the immediate roof or slope, the bolts should anchor in a competent stratum at least 9 inches beyond the possible bedding interface.

2.2 BEAM BUILDING

In most cases, the strong and self-supporting main roof is beyond the reasonable distance that ordinary roof bolts can reach to anchor for suspension. However, roof bolts can be applied in such situation with great success. In fact, sagging and separation of roof laminae cause both vertical movement and horizontal movement along the bedding interfaces. Bolts through these layers can prevent or greatly reduce horizontal movement, and the tension applied to the bolts manually on installation or induced by rock vertical displacement clamps the layers together, making all the layers have to move with the same magnitude of vertical displacement. On the other hand, frictional forces, which are proportional to the bolt tension, are induced along the bedding interface, also making horizontal movement difficult, as shown in Figure 2.2.4. This bolting pattern is very similar to clamping a number of thin, weak layers into a thicker, strong one, forming a fixed-end composite beam. Theoretically, assuming that all the thin layers are of same material, the maximum bending strain at the clamped ends of the composite beam is (Peng, 1984):

$$\varepsilon_{\max} = \frac{wL^2}{2Et} \qquad \text{Equation 2.2.5}$$

where

$E =$ Young’s modulus;

$L =$ Length of the immediate roof;

$t =$ Thickness of the composite beam;

$w =$ Unit weight of the immediate roof.

This equation shows that the thicker the beam, the smaller the maximum strain induced at the clamped ends. In other words, the clamping action produces a beam building effect.

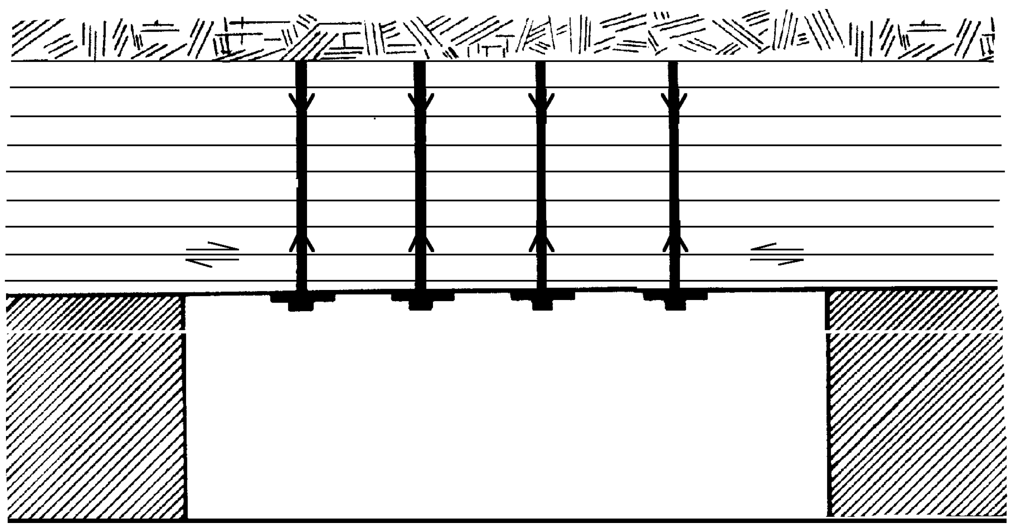


Figure 2.2.4 Beam building effect of roof bolting

Panek’s research (1956) indicated that beam building effects increase with decreasing bolt spacing, increasing bolt tension, increasing number of bolted laminae, and decreasing roof span. In most situations, where the immediate roof consists of laminated strata, both suspension and beam building effects coexist.

Xiu (1990) found that beam building effect has a dual contribution toward the stability of the immediate roof when he studied the mechanisms of rock bolting in gateroads of retreating longwalls in China. Beam building apparently increases the bending strength of the composite beam; it also increases the bending stiffness as well.

For a beam consisting of n identical layers without bolts, as shown in Figure 2.2.5.a, the bending strength, B_1, can be computed as:

$$B_1 = n\frac{bh^2}{6} \qquad \text{Equation 2.2.6}$$

where

$n =$ Number of layers;

$b =$ Length of the beam;

$h =$ Thickness of a layer.

The bending stiffness, T_1, can be expressed as:

$$T_1 = n\frac{Ebh^3}{12} \qquad \text{Equation 2.2.7}$$

where

$E =$ Young's modulus;

other variables defined previously.

For a composite beam consisting of n identical layers with bolts binding them together firmly, as shown in Figure 2.2.5.b, the bending strength, B_2, can be computed as:

$$B_2 = \frac{b(nh)^2}{6} \qquad \text{Equation 2.2.8}$$

The bending stiffness, T_2, can be expressed as:

$$T_2 = \frac{Eb(nh)^3}{12} \qquad \text{Equation 2.2.9}$$

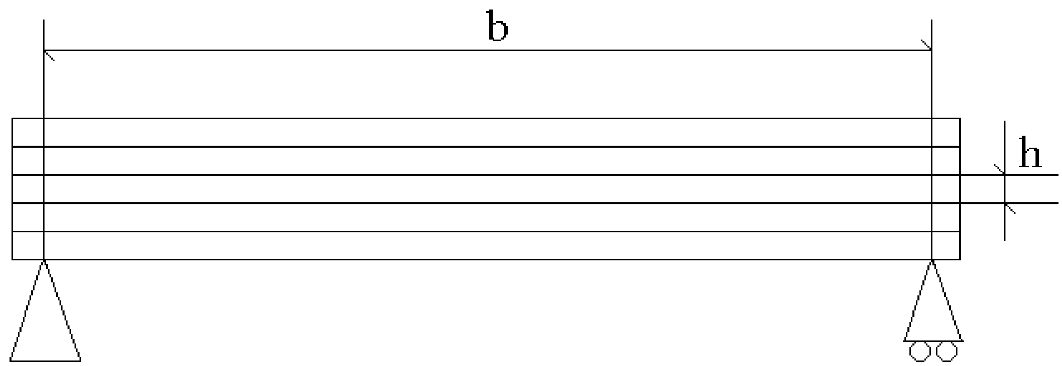


Figure 2.2.5.a Beam bending strength and stiffness without bolting

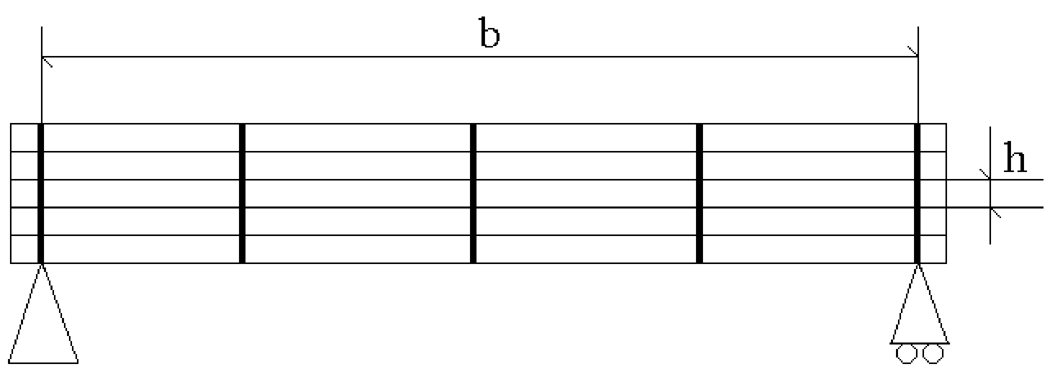


Figure 2.2.5.b Beam bending strength and stiffness with bolting

The bending strength of the bolted beam increases by n times compared to that of the unbolted beam, while the bending stiffness increases by n^2 times. The improvement of bending strength is always good for roof stability. However, under certain conditions, increasing bending stiffness may cause extra load from the overlying strata acting on the beam. The beam may not fail in tension because of the increased bending strength, but may fail by shearing at the two ends once the accumulated shear forces exceed the shear strength of the composite beam, as shown in Figure 2.2.6. It is observed that this kind of failure has the following features:

- The bolted composite beam falls out;
- Failure planes at the two ends of the beam are nearly vertical;
- The upper failure plane is exactly at the bolted horizon where pre-tension of the bolts creates a tensile stress area around the anchor of each bolt; and
- Sometimes using longer bolts just increases the height of roof fall.

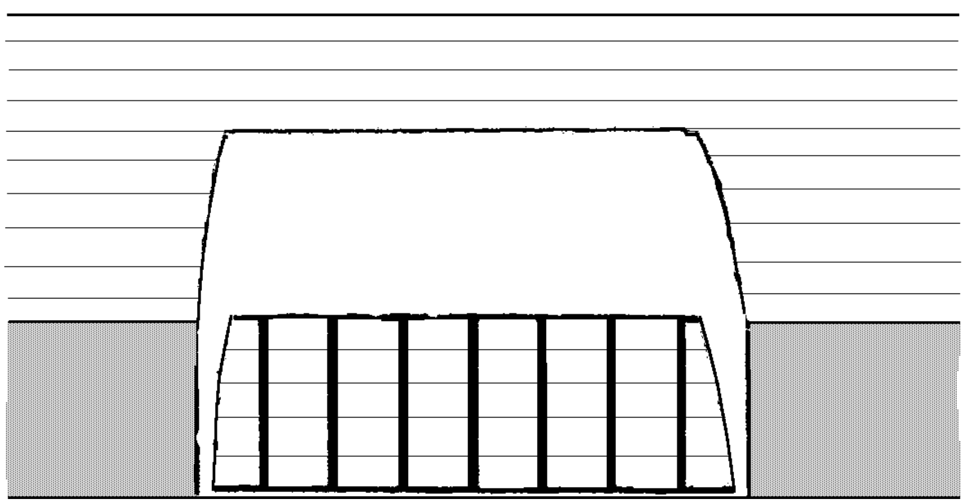


Figure 2.2.6 Shear failure of beam building

2.3 KEYING

When the roof strata are highly fractured and blocky, or the immediate roof contains one or several sets of joints with different orientations to the roofline, roof bolting provides significant frictional forces along fractures, cracks, and weak planes. Sliding and/or separation along the interface is thus prevented or reduced, as shown in Figure 2.2.7. If the bolts are installed inclined to the roofline and perpendicular to the fracture plane as shown in Figure 2.2.8.a, the minimum axial stress which the bolt must provide for stability is (Peng, 1984):

$$\sigma_b = \frac{\sigma_p (\sin\alpha \cos\alpha - \cos^2\alpha \tan\phi)}{\tan\phi} \qquad \text{Equation 2.2.10}$$

where

$\sigma_p =$ Horizontal stress;

$\alpha =$ Angle between the normal to the fracture plane and the horizontal plane;

$\phi =$ Friction angle of the fracture plane.

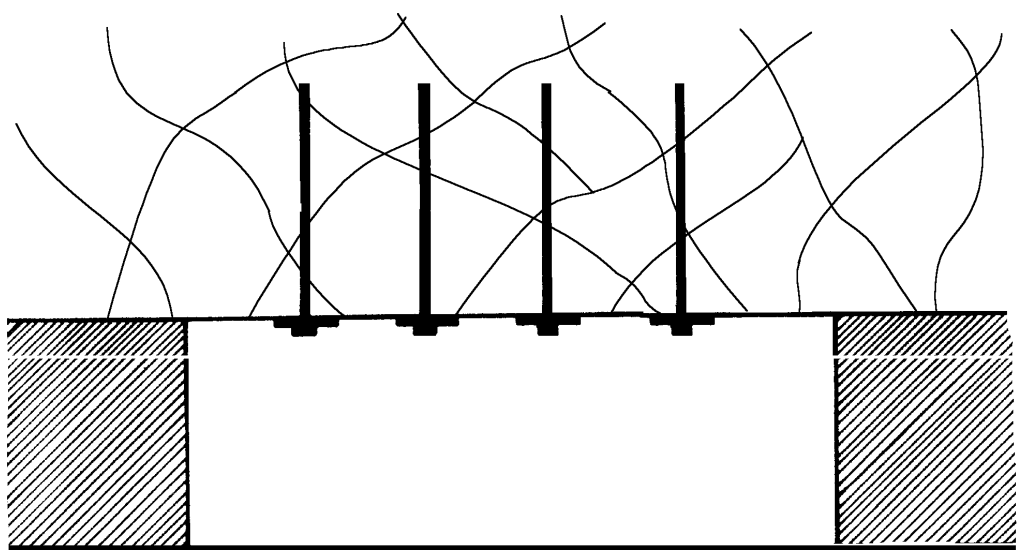


Figure 2.2.7 Keying effect of roof bolting

The minimum axial stress needed to maintain stability, when the bolt is installed perpendicular to the roof line as shown in Figure 2.2.8.b, can be estimated as:

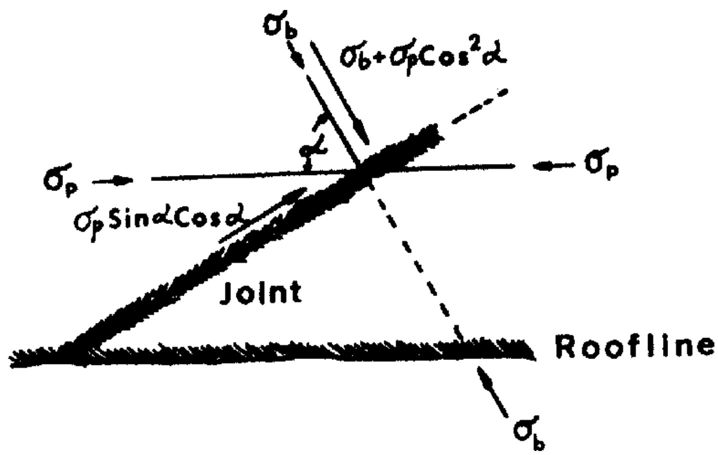


Figure 2.2.8.a Bolt installed inclined to the roofline (Peng, 1984)

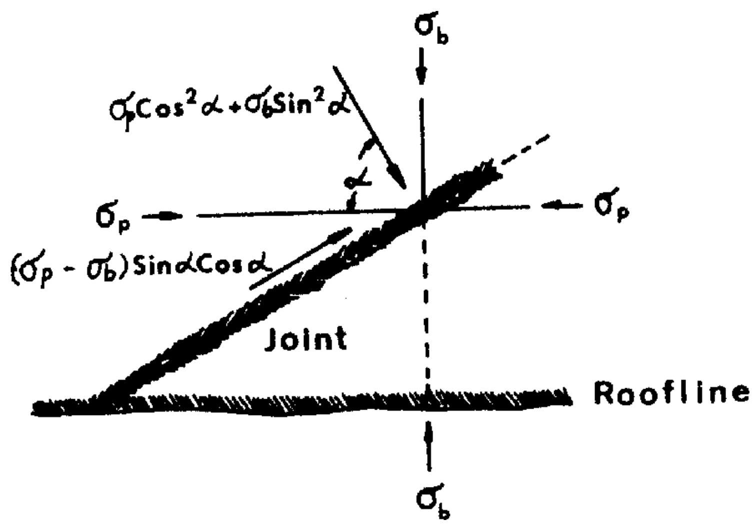


Figure 2.2.8.b Bolt installed perpendicular to the roofline (Peng, 1984)

$$\sigma_b = \frac{\sigma_p(\sin\alpha - \tan\phi\cos\alpha)}{(\cos\alpha + \tan\phi\sin\alpha)\tan\alpha} \qquad \text{Equation 2.2.11}$$

The equation demonstrates that the minimum axial stress needed is proportional to the horizontal stress in both cases. The smaller the horizontal stresses, the more effective the keying effect. Also σ_b becomes zero if α is equal to ϕ, indicating that stability can be maintained without using any bolt.

The keying effect mainly depends on active bolt tension or, under favorable circumstances, passive tension induced by rock mass movement. It has been shown (Gerrard, 1983) that bolt tension produces stresses in the stratified roof, which are compressive both in the direction of the bolt and orthogonal to the bolt. Superposition of the compressive areas around each bolt forms a continuous compressive zone in which tensile stresses are offset and the shear strength are improved, as shown in Figure 2.2.9.

3. TYPES OF ROCK BOLT AND VARIATIONS

3.1 CONVENTIONAL BOLT TYPES

Bolts reinforce the rock by binding the stratified or broken rock layers or blocks together. The binding effect is achieved through the friction forces created by the physical interlocking along the anchor and rock interface. Based on the basic anchor types, bolts can be categorized into: 1) point-anchored, and 2) full-length-grouted. The upper end of a point-anchored bolt is anchored by either a mechanical device or a short resin column. A bearing plate set between the bolt head and roofline serves as the other anchor. Usually a certain amount of tension is applied to the bolt at the time of installation. A full-length-grouted bolt is grouted to the borehole by fast-setting, high-strength resin or inorganic cement throughout the full length of the hole. No pretension is applied to the bolt.

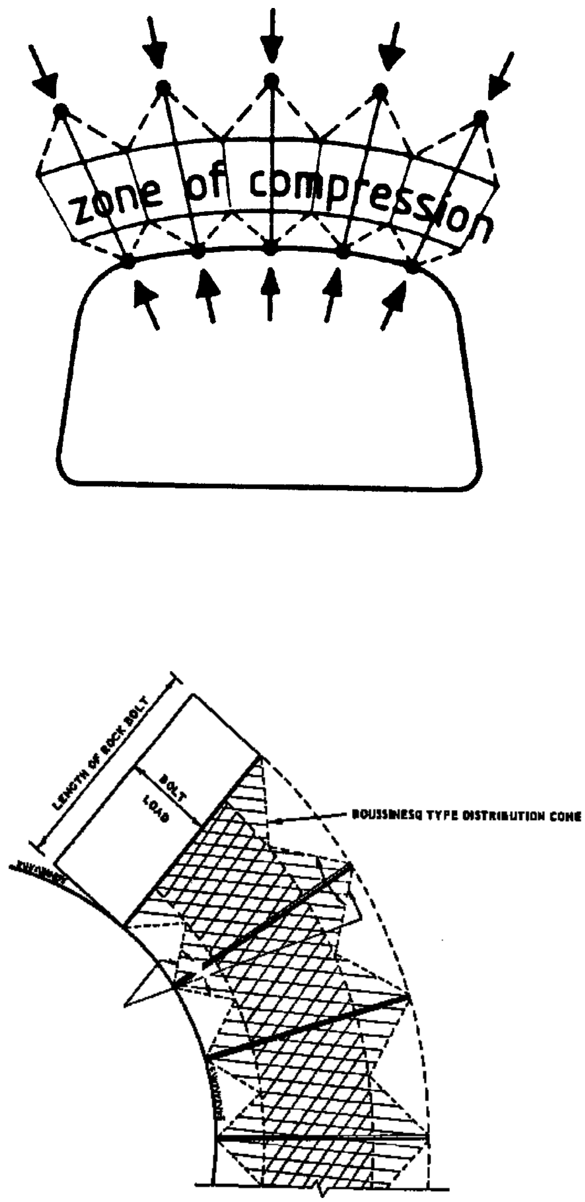


Figure 2.2.9 Compression zone created by keying (Gerrard, 1983)

Another way to categorize bolts is based on whether tension is applied to the bolt at the time of installation. Usually, a point-anchored bolt is tensioned, and it is also called a tensioned bolt, while the full-length-grouted bolt is referred to as an untensioned bolt because no tension is applied. Figure 2.3.1 shows five major types of roof bolts. In practice, many types of roof bolts are developed by combining these two basic bolt types to meet the specific support requirements and geological or geotechnical conditions. Table 2.3.1 shows the bolt types commonly used in coal mines, non-coal mines, and surface mines in the U.S..

It is estimated that, in underground mines, the most popular bolt is a mechanical bolt, because it is economical, easy to install, and effective for the entry lifetime, which are expected to last for a few months or years. About 60% of the bolts used in the U.S. are this type. Resin-grouted bolts account for about 30%. The remaining 10% are other methods of roof bolting, such as roof trusses, cable bolts, and friction stabilizers (split sets).

A survey conducted by Peng et al., in 1994 showed that at least seven different types of bolts are used to support tailgates. They are full-length-grouted, combination, point-anchored, tensioned with T channel, truss, rebar with truss, and double locks. Full-length-grouted bolts are the most popular (about 46% of the total mines surveyed). Point-anchored, combination, and tension with T channel bolts are also widely used in longwall tailgate.

3.2 SPECIAL BOLTS

In practice, in some specialized geologic and tectonic environment, high abutment pressure induced by mining activities often prevent the conventional bolting systems from functioning effectively. Special bolt types such as cable bolts, trusses, and split sets are designed to counteract such conditions.

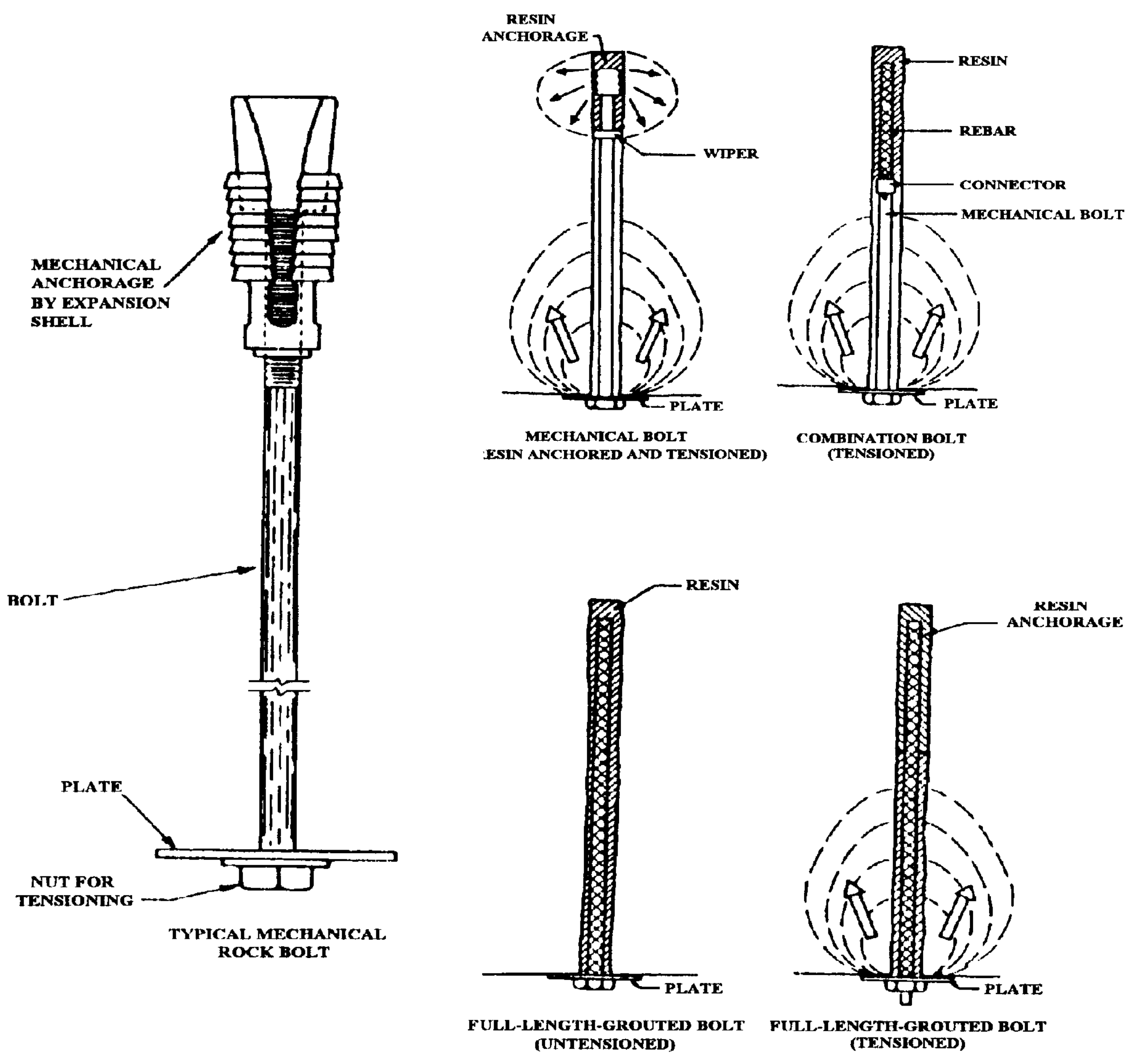


Figure 2.3.1 Types of bolts commonly used underground (Bieniawski, 1987)

Table 2.3.1 Types of roof bolts (Peng, 1984)

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

3.2.1 Cable bolts

In the early years, discarded hoists or slusher ropes, which were still in good condition, were reused after degreasing as bolts instead of rigid steel bar. While they were not expected to work better than steel bars, they were less costly, and flexibility allowed the cable to be packed into a coil, greatly facilitating material handling. An additional benefit was that bolt length was no longer limited by the opening geometry. Cable flexibility also allowed horizontal movement, thus reducing the tendency to shear. Cable bolts were introduced to the U.S. mining industry in 1970 as a method of reinforcing the ground before mining, as shown in Figure 2.3.2 a. The presence of cables had the effect of reinforcing the rock mass against the subsequent blast shock and stress redistribution. In longwalls, they are now widely used as secondary support, supplementing or replacing traditional secondary support systems like wood cribs and posts, hydraulic jacks, or spot roof bolts (Tadolini and Kock, 1993). Today the high-strength cables are about $\frac{5}{8}$ inch in diameter and consist of seven strands, as shown in Figure 2.3.2.b.

Since cable bolts are not limited by length, they can be used to reach the main roof, which is beyond the ordinary range of traditional bolts, to generate suspension effect. A cable bolting system is capable of strengthening and reinforcing roof strata to transfer high pressure into the main roof and onto supporting structures away from the ribline, enhancing pillar performance. This system also reduces entry convergence. Cable bolts are widely used to stabilize slopes. Figure 2.3.3 shows a typical application of this kind.

3.2.2 Roof Trusses

Trusses were introduced in the 1960s as an alternative method of supporting unstable roof when conventional roof bolts alone were not effective, such as when fallout is frequent between bolts or cutter roof is encountered. Basically, a truss consists of two inclined point-anchored bolts, a connection bar, a turnbuckle to give proper tension to the bar, and an adjusting wedge box, as shown in Figure 2.3.4. The tensioned connection bar

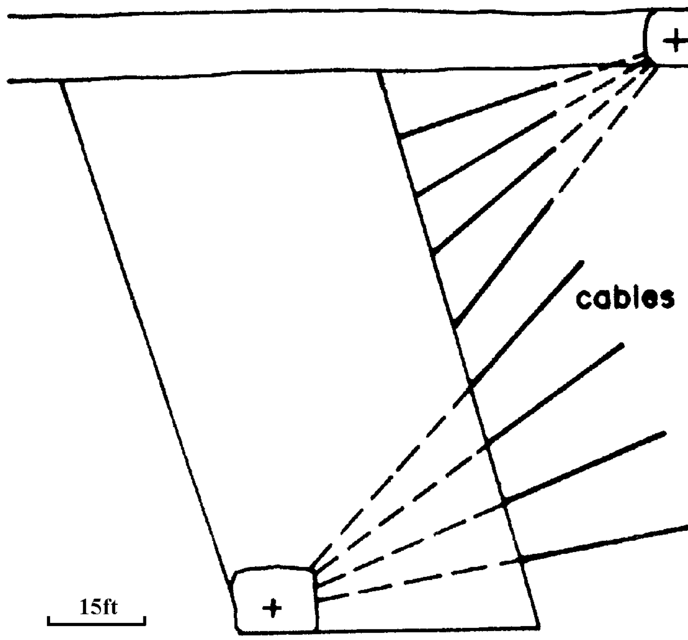


Figure 2.3.2.a Cables using to reinforce ground before mining (Fuller, 1983)

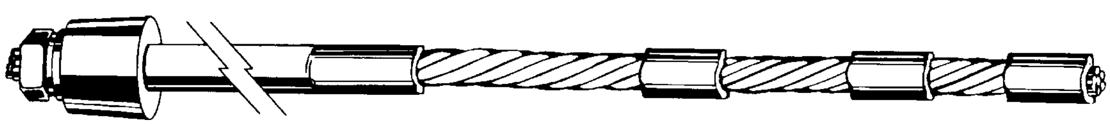


Figure 2.3.2.b A typical high-strength cable bolt

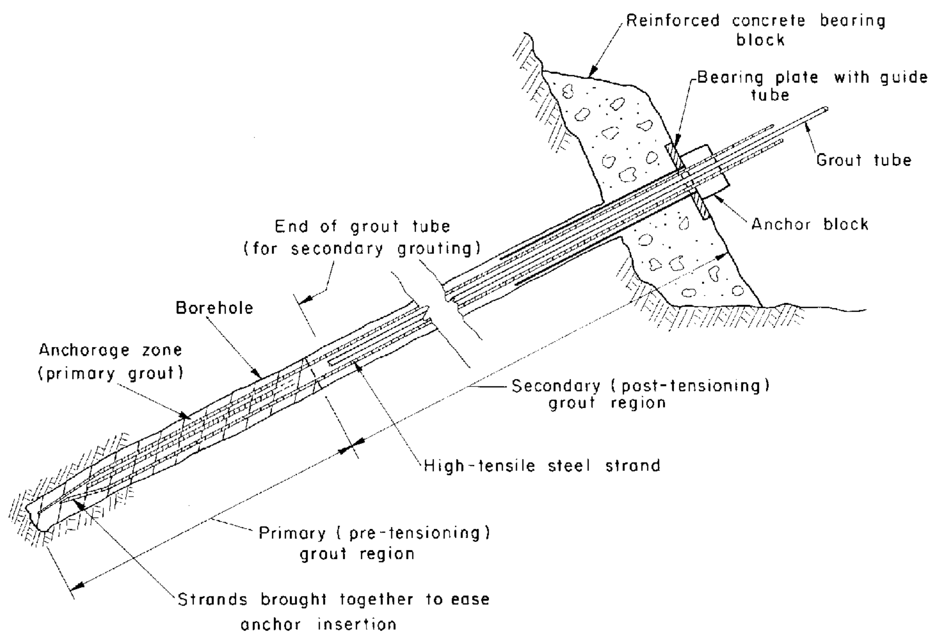


Figure 2.3.3 Typical cable bolt installation for slope stabilization (CANMET, 1977)

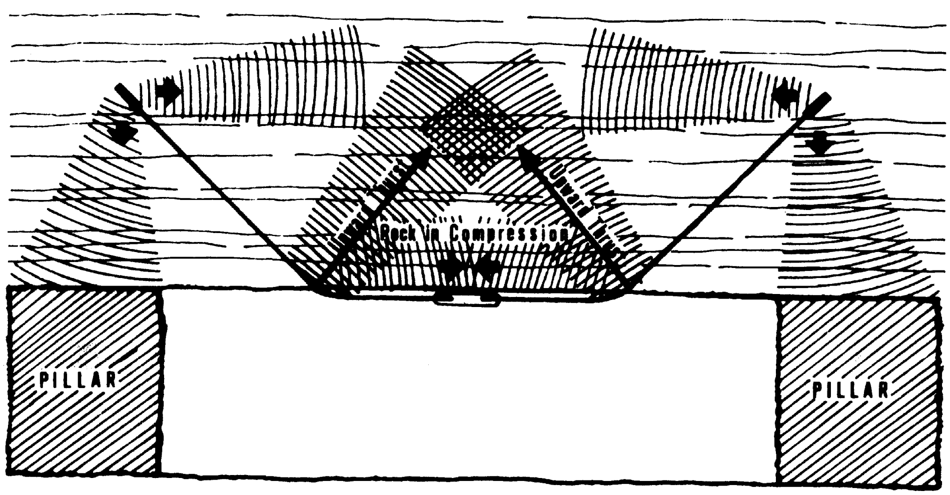


Figure 2.3.4 Compression zone created by trusses (Khair, 1983)

keeps the roof in compression and also can yield downward as the roof displaces. When used as slings, trusses provide a suspension effect to the roof, transferring load away from the entry out to the center of the pillars. In three- and four-way intersections, where tensile stresses are very high, trusses can produce compressive forces to offset high tensile stresses at mid-intersection roof. In areas where roof falls have previously occurred, trusses are a useful way to stabilize newly exposed roof arches.

Under certain conditions where mining-induced high horizontal shear stress can affect the rigid inclined truss anchor, an alternative cable truss was introduced. The flexibility of cable allows installation without regard to the level or degree of bearing surface of the immediate roof. The inclined anchor can be installed at the roof/rib corner, transferring the load further into the pillars, thus reducing the chance of sloughage.

3.2.3 Split Sets

A split set rock reinforcement system (friction rock stabilizer) is widely used in the U.S. metal mines. It consists of a thin-wall steel tube of $1\frac{1}{2}$ inches in diameter, which is forced into a borehole with a diameter of $1\frac{3}{8}$ inches. The spring action of the compressed steel tube induces a frictional force along the length of the tube and anchors the tube into the rock. Installation is very quick and is quite effective if the split set is not installed close to a face and the stresses imposed on the tube are not very large. Approximately 3.5 million split sets are used in metal mines per year.

4. PREVIOUS RESEARCH IN BEAM BUILDING

The success of ground control by rock bolting depends on at least one of the three basic mechanisms: suspension, beam building, and keying. In some coal mines, the competent self-supporting main roof is too far above the roof line to provide suspension anchor (Gerdeen et al., 1977). In such situations, beam building is the major mechanism for explaining the success of roof bolting applications. In an attempt to understand how beams are built, how to select bolting parameters, such as bolting pattern, bolt length, bolt density, for achieving optimal beam building effect, a lot of research, from field test, laboratory experiment, to numerical modeling, have been conducted and great progress has been made.

In the early roof bolt research conducted by the U.S. Bureau of Mines (USBM), the analytical studies and laboratory investigations with physical models were aimed at beam building, in which the strata were clamped together by tensioned bolts to form a laminated beam with enhanced bending strength. In showing how mechanical point anchor bolts support the immediate roof, Panek (1956) reported that for beam building to take place the bolts must be in tension. The tension in the bolt would produce a normal force between layers such that the frictional forces can carry the horizontal shear stress. The reduction of the bending strain due to beam building effect is:

$$\varepsilon_{\Delta f} = \varepsilon_f - \varepsilon_{fu} \qquad \text{Equation 2.4.1}$$

where ε_f and ε_{fu} are the maximum strains in the bolted and unbolted roof, respectively.

The reinforcement factor, *RF*, due to the friction is defined as:

$$RF = \frac{1}{1 + \frac{\varepsilon_f}{\varepsilon_{fu}}} \qquad \text{Equation 2.4.2}$$

This is the design equation for the degree of reinforcement produced by roof-bolting laminae of equal thickness. After being visualized as in Figure 2.4.1, the equation can be used as a bolt design tool. Furthermore, this equation indicates that beam building effect increases with decreasing bolting spacing, increasing bolt tension, increasing the number of bolted laminae, and decreasing roof span.

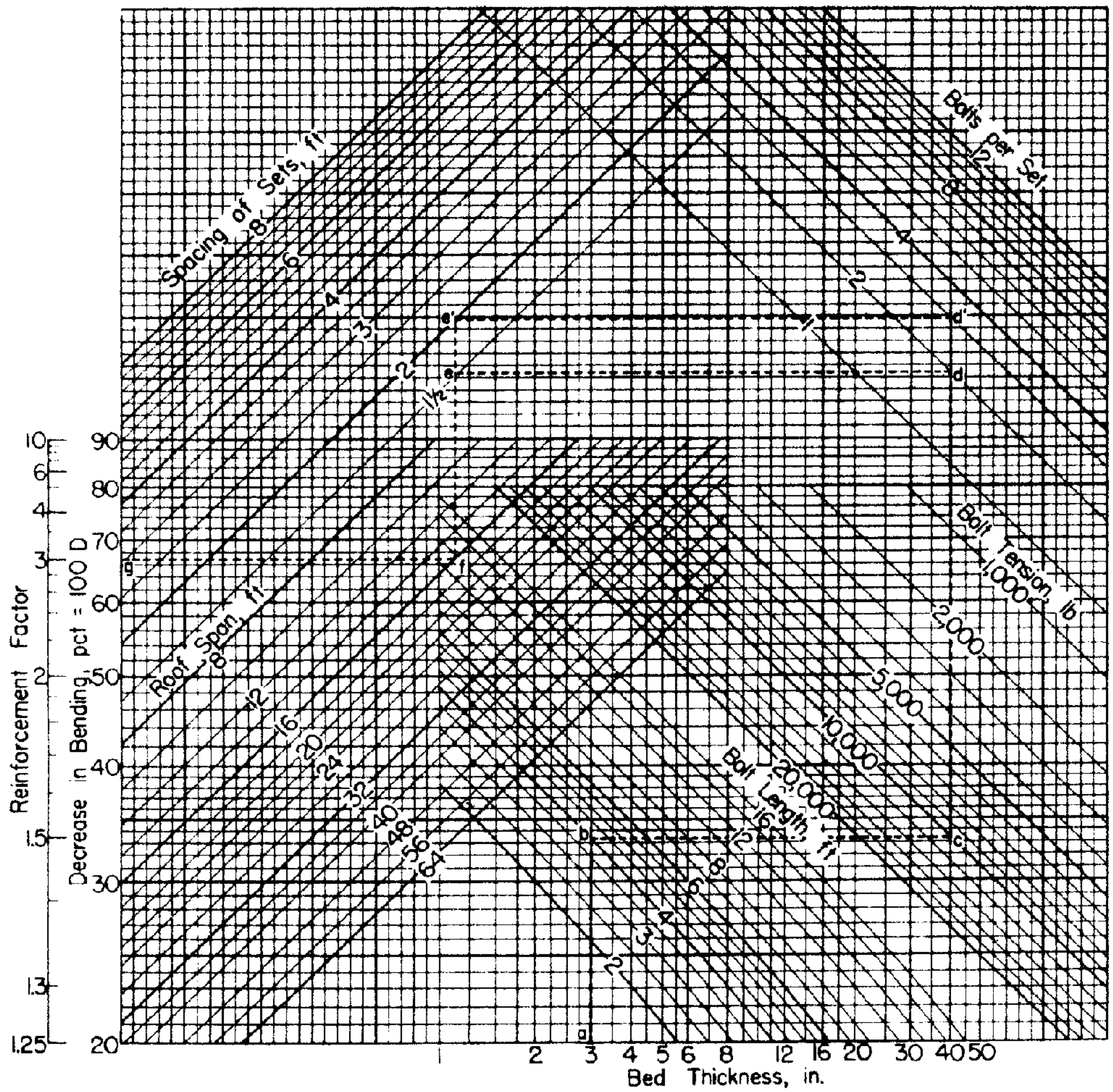


Figure 2.4.1 Chart for evaluating beam building of roof bolting (Panek, 1956)

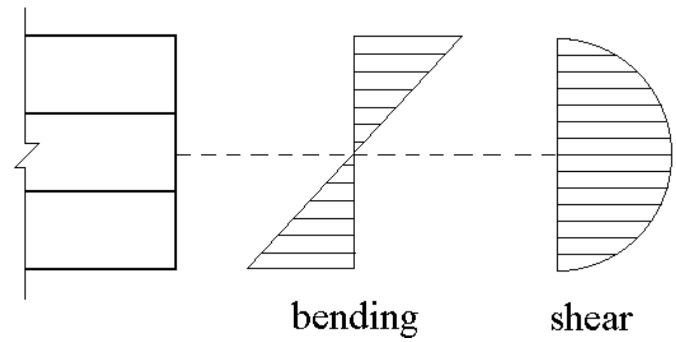
A beam as a structural member must carry two types of stresses: bending or normal stress and vertical shear stress, as shown in Figure 2.4.2. The vertical shear stress produces horizontal shear stress that must be transferred from one layer to the other if the laminated beam is to act as a monolithic system. Tensioned mechanical bolts increase the normal forces between the layers, thus increasing the friction effect. Fully grouted, untensioned bolts transmit this shear from layer to layer by the shear stiffness of the bolt-grout-rock system (Snyder, 1983). Experimental study conducted by Krohn (1978) indicated that the fully grouted, untensioned bolts were more effective in terms of the reduction of the mid-span deflection. Further study of the strains at the both ends and the mid-span showed that each layer tried to act independently of each other with some overall bending taking place near the center of the beam. In fact, as Jeffrey and Daemen (1983) pointed out, it is unlikely that the additional force provided to the layer interface by bolting would ensure complete bonding and interaction of the layers on either side of the interface. Some slip and normal displacement would occur. Bolts installed untensioned generate resisting forces only in response to these small displacement. Incomplete interaction between two beams resting one on top of the other and connected by shear connectors was studied. A complicated equation defining the deflection along the composite beam respect to the distance to one end was derived:

$$\begin{aligned} y = \frac{w}{\sum EI}[&\frac{ZC_2}{C_1^3}(\cosh(\sqrt{C_1}x) - \tanh(\sqrt{C_1}\frac{L}{2})\sinh(\sqrt{C_1}x) - 1) \\ &- \frac{L^2}{2}(\frac{x^3}{6L} - \frac{x^4}{12L^2}) + \frac{ZC_2L^2}{2C_1}(\frac{x^3}{6L} - \frac{x^4}{12L^2} - \frac{x^2}{C_1L^2}) \\ &- \frac{Lx}{24}(-L^2 + \frac{ZC_2L^2}{C_1} - 2Z\frac{C_2}{C_1^2})] \end{aligned} \qquad \text{Equation 2.4.3}$$

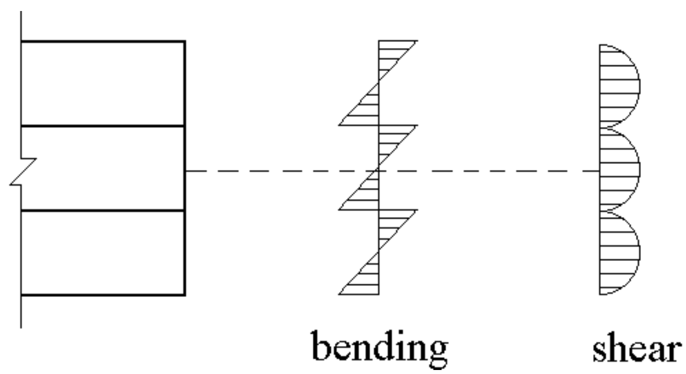
where

$$C_1 = \frac{k}{s}\frac{\overline{EI}}{\overline{EA}\sum EI};$$

$$C_2 = \frac{k}{s}\frac{Z}{\sum EI};$$



Monolithic beam



Each layer acting independently

Figure 2.4.2 Stress distribution in beams (After Snyder, 1983)

k = Shear stiffness of shear connector;

s = Spacing of shear connectors;

$\sum EI = E_1 I_1 + E_2 I_2$;

$\overline{EI} = \sum EI + \overline{EA} Z^2$;

$\overline{EA} = (E_1 A_1 E_2 A_2)/(E_1 A_1 + E_2 A_2)$;

E_i = Young's modulus of beam i;

I_i = Moment of inertia of beam i about its neutral axis;

A_i = Cross section area of beam i;

Z = Distance between neutral axes of the beams;

w = Uniformly distributed load acting upon the upper beam;

L = Length of the beams;

X = Position along length of the beam.

Jeffrey and Daemen's research showed that, in laminated beams with complete interaction, the bending takes place about the neutral axis of the composite. By contrast, the bending in laminated beams with complete interaction takes place partly about the composite neutral axis and partly about the individual centroids of the layer. And as the shear resistance of the interface approached zero, each layer will act independently and bend entirely about its own neutral axis. The location of maximum interlaminar shear stress does not occur at the ends of beams with clamped ends; instead it occurs at a location that is away from the end toward the inflection point of the beam. Shear failure is most likely to initiate at this location. In coal mines, it is frequently observed that dominant steep shear fractures and cutters develop between one to three feet inside the ribline. Jeffrey and Daemen also noticed that shear and normal failures interact with each other and the interaction becomes complicated and difficult to analyze as the number of layers involved increases.

Xiu (1990) noticed this interaction between the shear and normal failure. This interaction is caused by the dual nature of beam building effect. On one hand, the bending strength of the beam bound together by bolts increases linearly with respect to the number of layers bound, enhancing the load carrying capacity of the composite beam. On the other hand, bending stiffness of the beam increases exponentially, with a power of two, to the number of layers bound. This causes extra load from the overburden to act on the beam, consequently causing shear stress to accumulate in the vicinity of the two ends. Therefore, for the success of a bolting application, it is essential to seek a balance between this dual nature of the beam building effect according to the specific geological settings and lithological properties.

Fairhurst and Singh (1973) presented an analytical theory for the analysis of the beam building effect using a two-dimensional plate buckling criterion. The effectiveness is a measure of the moment of inertia of the bolted beam, which depends on such variables as friction between layers, bolt density, and shear stiffness of bolt-grout-rock interface. But later it was found (Snyder, 1982) this theory is not valid for soft rock.

Stimpson (1983) conducted a series of experiments investigating the influence of roof bolt location on the reinforcement by fully grouted bolts. Eight patterns of roof bolting were used in beam tests, as shown in Figure 2.4.3. The results of reduction in beam deflection in terms of bolting pattern and number of bolts are presented in Figure 2.4.4 and Figure 2.4.5 respectively. It is noticed that pattern 5 with 6 bolts is the most effective bolting pattern among the eight, though the effectiveness of patterns 4, 6, and 7 does not significantly differ from pattern 5 and each other. The extent of deflection reduction increases as the number of bolts increases, but it becomes insignificant after the number of bolts exceed 5 or 6. Based on this research, a non-uniformity of bolt spacing with a smaller spacing towards the ribsides is recommended for maximum reinforcing effect.

Researches conducted by Stankus and Guo (1996) indicated that in bedded and laminated strata, when beam building is the primary support mechanism, point anchor and fully tensioned resin-assisted roof bolts were very effective, especially when they were installed at high tensions as soon as after excavation. Bolts were installed in a

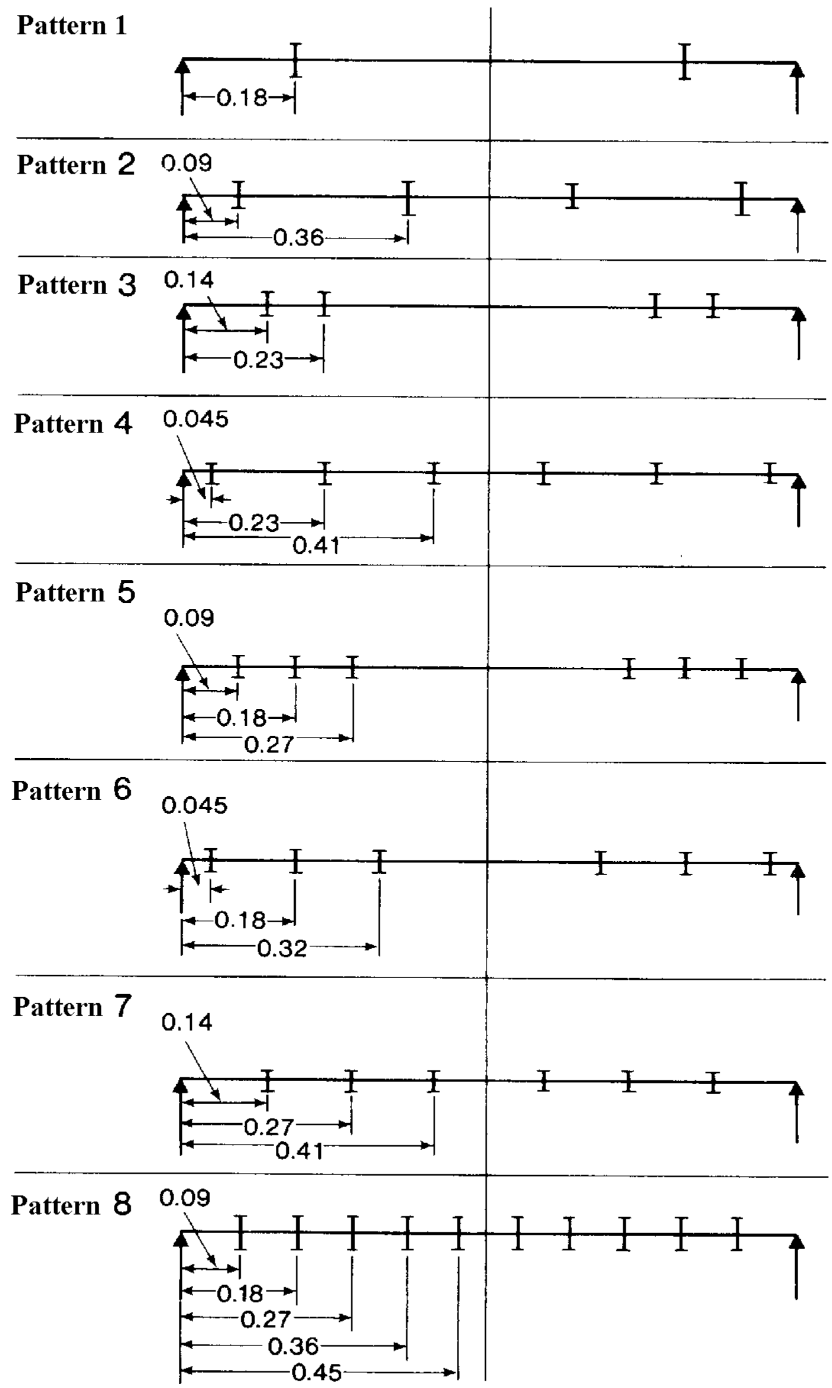


Figure 2.4.3 Eight patterns of bolting used in beam tests (Stimpson, 1983)

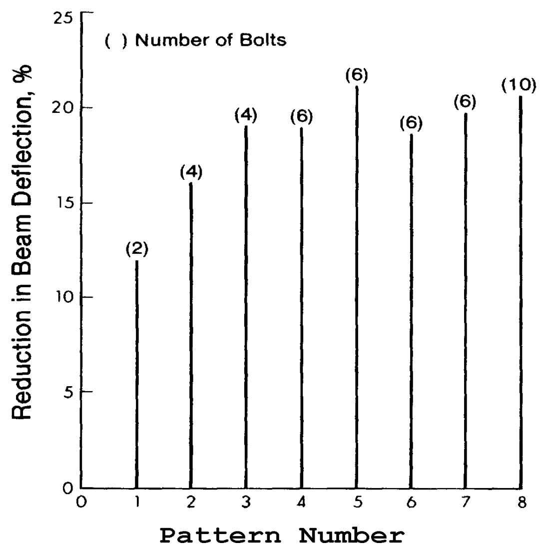


Figure 2.4.4 Reduction of beam deflection for various bolting patterns (Stimpson, 1983)

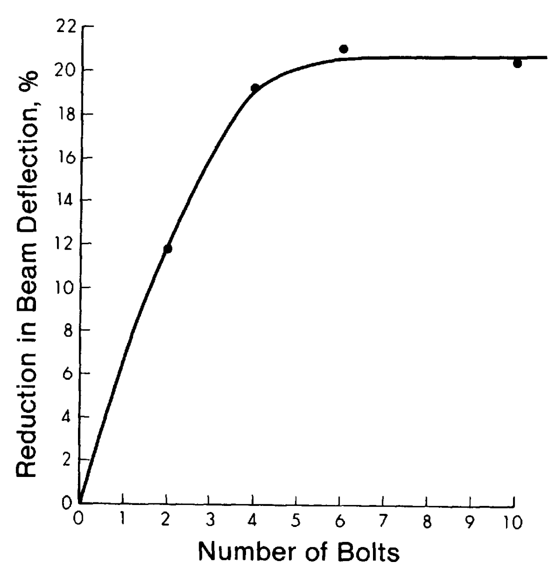


Figure 2.4.5 Relationship between reduction in beam deflection and number of bolts (Stimpson, 1983)

longwall headgate entry with three different bolt lengths: 11 feet, 8 feet, and 5 feet, and three different tensions: 15,000 lbs, 20,000 lbs, and 25,000 lbs. It was observed that:

- The longer the bolt, the larger the beam deflection. Alternatively, the shorter the bolt, the smaller the deflection and the stronger the beam;
- For bolts with the same length, the larger the installed tension, the smaller the beam deflection. That is, a stronger beam can be built with the same bolt by utilizing a larger installed tension;
- Bolts with higher pre-tension induce a smaller deflection within the bolting range, which in turn results in a smaller deflection higher up in the roof above the bolting range; and
- The longer the bolt, the larger the load that will be induced in the bolt as the result of in-situ or mining related stresses.

To determine bolt length, pre-tension, and spacing to achieve the optimum beaming effect (OBE), a new methodology was thus developed, as shown in Figure 2.4.6. This method involves the determination of the amount of allowable bed separation. For computer modeling, it is set to be zero, but in reality, stability still can be maintained with some bed separation.

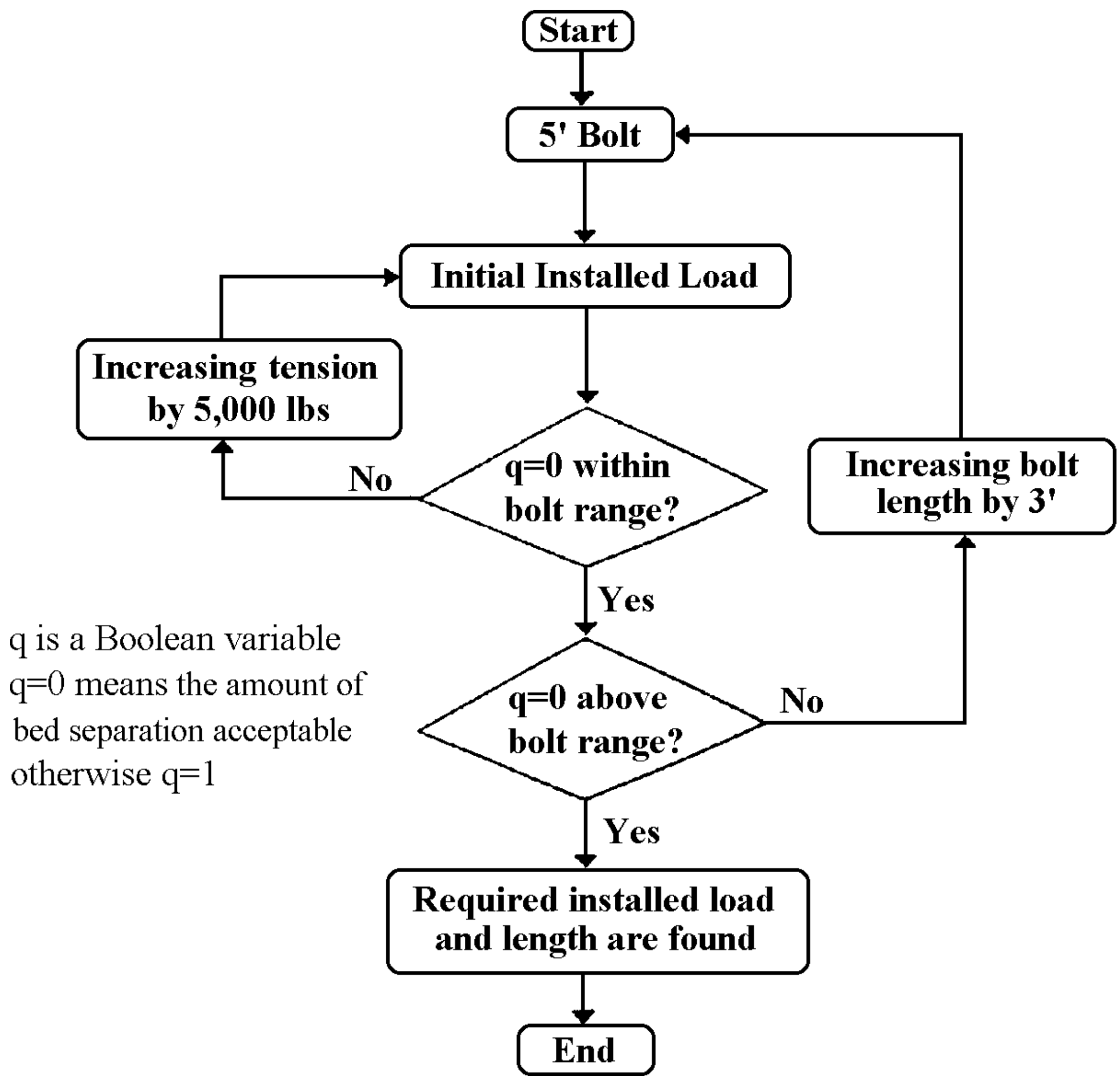


Figure 2.4.6 Flow chart for determining OBE (Stankus, 1997)

CHAPTER 3
DESIGN OF BOLTING SYSTEMS

1. INTRODUCTION

As in the design of other support systems, the design of a rock bolting system depends on geotechnical discontinuities in the intact rock, material properties, the magnitude and distribution of the in-situ and induced stresses, support requirements such as acceptable roof deformation and lifetime of the opening, and the size and shape of the openings. For a complete and appropriate rock bolting system design, the following parameters must be determined properly:

- Bolt type;
- Bolt length;
- Pattern and spacing of bolts;
- Bolt diameter and anchor capacity; and
- Whether pretension should be applied or not. If pretension is desirable, what is the appropriate magnitude of the pretension?

2. SELECTION OF BOLT TYPE

There are many types of commercial bolts available. They are designed and manufactured to meet almost all kinds of geological settings and support requirements. The selection of an appropriate bolt type to meet specific requirements at a lowest cost and maximum effectiveness could be a very confused task. Carefully examining the geological conditions and thoroughly understanding the performance of different types of bolts under different settings are essential for a successful bolt type selection. Guidelines for choosing bolt type are presented below (Smith, 1993):

1. Mechanical bolts are used in:
    - Harder rock conditions where the rock properties will not adversely affect the gripping force of the anchor;

- Temporary reinforcement systems;
- Where bolt tension can be checked regularly;
- Rock that will not experience high shear forces;
- Rock that is not very highly fractured; and
- Areas away from blast sites where bolt tension may be lost.

2. Grouted bolts are generally used in:
    - The above conditions where mechanical bolts are not recommended;
    - Permanent reinforcement systems;
    - Boreholes without continuous water run-off problems or with continuous water run-off that would not interfere with installation; and
    - Rock without wide fractures and voids in which significant amounts of grout will be lost.
3. Untensioned bolts are recommended in rock that is highly fractured and deformable as long as adequate bolt installation is feasible. Generally, bolts in more competent strata often require a shorter grout column than do bolts in less competent strata.
4. Tensioned grouted bolts are recommended for use where additional frictional forces, in combination with a grouted column, may enhance roof stability.

Compared to mechanical bolts, fully grouted resin bolts have the following advantages (Peng and Tang, 1984):

- Anchorage is virtually guaranteed and independent of strata type;
- The bolt is permanently effective throughout its full bonding length;
- The fully grouted bolt prevents both vertical and horizontal strata movements;
- The hole length is not critical since the bonding can be adjusted according to the type of strata;
- The fully grouted bolts seal wet holes and exclude air, thus reducing corrosion of the bolt assembly and weathering of rock;
- Damage to the bolthead, bearing plate, or rock at the collar of the hole does not cause the resin bolts to become ineffective;

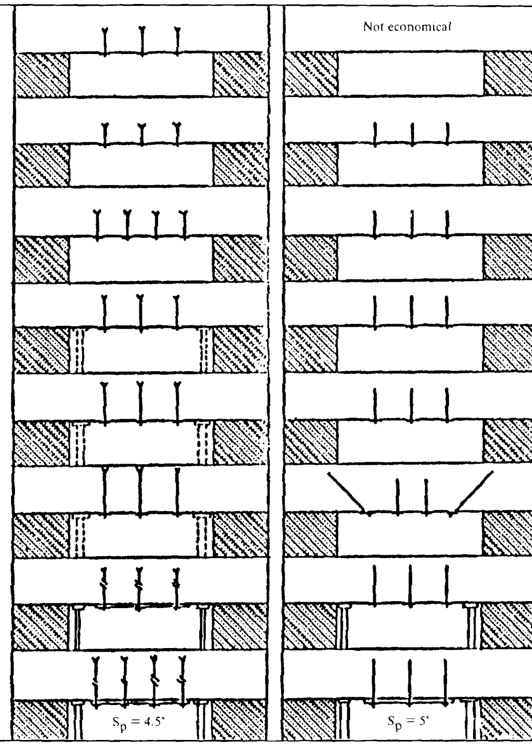
- The resin grouted bolt can absorb blast vibrations without bleed-off of the bolt load;
- The fully grouted resin bolt shows excellent performance with regard to anchorage creep; and
- Time and labor for tensioning the bolts are not needed.

Therefore, a general rule for mechanical bolts is that they cannot be reliably installed in critical areas as permanent support, or in places where significant roof deformation is likely to occur, or in extremely soft rock such as shales and clays that exhibit highly plastic behavior. However, for temporary reinforcement system, mechanical bolts should be considered first. The major factors that affect mechanical bolt performance are bleed-off of anchorage, deterioration of strata under the bearing plate through weathering, wear and damage of threads on bolts and damaged bearing plates, and the effect of blasting vibration on the anchorage. Once mechanical bolts are chosen, the following guidelines should be followed to ensure best performance (Peng and Tang, 1984):

- Bolt the roof as soon after excavation as feasible to prevent strata separation;
- Determine the optimum expansion shell and the optimum anchorage horizon through on-site pullout tests;
- Employ substantial structural components, i.e. high strength bolts and plates with low deformation characteristics, to reinforce the strata effectively;
- Ensure that all bolts work together by installing them with equal tension.

Another practical way of selecting bolt types is based on rock mass classification. The RMR system is one of the most widely accepted rock mass classification systems. Unal (1983) developed a selection chart for coal mines, as shown in Figure 3.2.1. This procedure requires prior knowledge of the geological conditions at the site and particularly the following parameters: strength of intact rock material, spacing of discontinuities, orientation of discontinuities, conditions of discontinuities such as roughness, separation, weathering, infilling, and continuity, groundwater conditions, and the in-situ stresses. Due to these limitations, this procedure is only useful in the planning and preliminary design stages.

Roof rock class	Rock mass rating (RMR)	Rock load height HT(ft)	Support specifications: Mechanical bolts	Support specifications: Resin bolts	Alternate support patterns: Mechanical bolts/posts	Alternate support patterns: Resin bolts/posts	Specifications for posts
I Very good							
	90	2.0	L: 2.5' S: 5' x 5' G: 40 ø: 5/8" C: 6.2 tons			Not economical	
	80	4.0	L: 2.5' S: 5' x 4.5' G: 60 (40) ø: 3/4"(7/8") C: 11 tons	L: 2.5' S: 5' x 5' G: 60 ø: 3/4" C: 12 tons			
II Good							
	70	6.0	L: 3.0' S: 4' x 4' G: 60 ø: 3/4" C: 10 tons	L: 3.0' S: 5' x 5' G: 60 ø: 3/4" C: 18 tons			
	60	8.0	L: 4.0' S: 5' x 5' G: 60 ø: 5/8" C: 9 tons	L: 4.0' L: 5' x 5' G: 60 ø: 1" C: 23.7 tons			$\phi_p = 5.5"$ $S_p = 10'$
III Fair							
	50	10.0	L: 5.0' S: 5' x 5' G: 40 ø: 3/4" C: 8 tons	L: 4.0' S: 5' x 4' G: 60 ø: 1" C: 23.7 tons			$\phi_p = 6.5"$ $S_p = 10'$
	40	12.0	L: 6.0' S: 5' x 5' G: 40 ø: 3/4" C: 7 tons	L: 4.0' S: 4' x 4' G: 60 ø: 1" C: 23.7 tons			$\phi_p = 6.5"$ $S_p = 7.5'$
IV Poor							
	30	14.0	L: 7.0' S: 5' x 5' G: 40 ø: 5/8" C: 6 tons	L: 5' S: 5' x 5' G: 60 ø: 3/4" C: 12 tons			$\phi_p = 5.5"$ $S_p = 5'$
	20	16.0	L: 8.0' S: 4' x 4.5' G: 40 ø: 5/8" C: 5 tons	L: 5' S: 5' x 5' C: 60 ø: 3/4" C: 12 tons	$S_p = 4.5'$	$S_p = 5'$	$\phi_p = 6.0"$



L = bolt length G = grade of steel C = bolt capacity S_p = post spacing
S = bolt spacing ø = bolt diameter $ø_p$ = post diameter

Figure 3.2.1 Roof bolt selection chart for coal mine entries of 20 feet-wide (Unal, 1983)

3. BOLT LENGTH

Generally, bolt length is based on the total thickness of unstable strata, especially when bolts are used to control slope stability. But in underground situations, the maximum length of a rigid bolt is limited by the roof-to-floor height of the opening. However, some bolts such as $\frac{5}{8}$ in. bolts can be bent before inserting into the boreholes and then easily straightened immediately afterward. Rebars for full-length grouted bolts are notched such that they can be bent and straightened, but the bolt strength is reduced. Cable bolts with great flexibility completely overcome this limitation. If bolts are to act mainly in suspension, it should be ensured that the bolts are long enough to be firmly anchored in a competent rock mass. If the situation does not allow suspension, bolt length should be long enough to create beam building effects or keying effects. Lang and Bischoff (1982) developed a relationship between bolt length and roof span, as shown in Figure 3.3.1, which can be used as a guideline to determine bolt length.

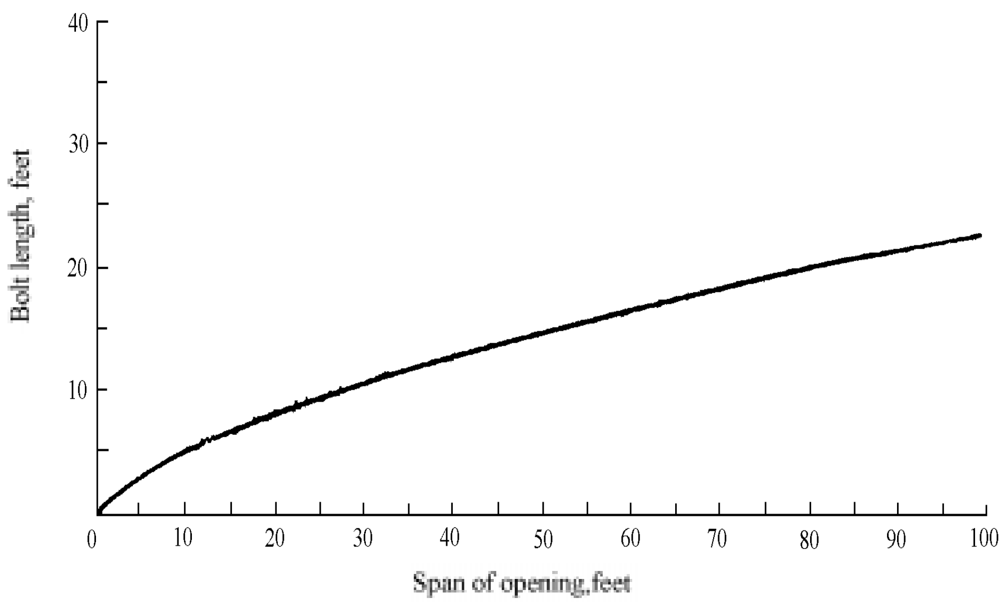


Figure 3.3.1 Relationship between bolt length and roof span (Lang and Bischoff, 1982)

Biron and Arioglu (1982) simplified the relationship between roof bolt length and the roof span to be linear. For strong roofs, bolt length is about one-third of the roof span, while for weak roofs bolt length is about one-half of the roof span. For very strong roofs, where the bolting is used only to prevent spalling, the minimum recommended bolt length is 3 feet 4 inches.

Stankus' OBE concept provides a methodology for determining a minimum beam thickness, namely the minimum bolt length, to achieve the optimum beaming building effect. However, it requires a special program to create a two-dimensional finite element model and then conduct experiments with varying bolt length and pretension until the OBE is reached. On the other hand, the precision of this modeling result depends largely on how accurately the rock material properties and the composition of the roof lithology are modeled. Thus, the application of this promising method is limited.

While bolt length plays a vital role in the success of roof bolting application, the procedures of determining it are elusive. Hoek and Brown (1980), the U.S. Army Corps of Engineers (1980), the U.S. Bureau of Mines (Lang and Bischoff, 1982), the Canadian Ontario Hydro (1978), and the British Construction Industry Research and Information Association (Douglas and Arthur, 1983) provide some general empirical rules to check for bolt lengths. These rules are:

The minimum bolt length should be the greatest of the following three:

- Twice the bolt spacing;
- Three times the width of critical and potentially unstable rock blocks defined by the average discontinuity spacing in the rock mass; and
- For span less than 20 feet, bolt length of one half the span. For spans between 20 to 60 feet, linearly interpolate between 10 to 16 feet lengths, respectively. For excavations higher than 60 feet, sidewall bolts are one-fifth of wall height.

For coal mines, Title 30 of the Code of Federal Regulations (1977) require that openings should not exceed 20 feet in width where roof-bolting is the sole means of support, nor should they exceed 30 feet when roof-bolts and other support, such as wood posts, are used. Section 75 of CFR 30 stipulates that in no case should the length of roof bolts be less than 2.5 feet plus 1 foot if anchored in the stronger strata to suspend the immediate roof. When it is unclear whether to design support for a suspension or beam building or keying, it is generally better to design for all situations and then use the more conservative approach. In many coal mines, 4- to 6-feet-long bolts are sufficient.

4. BOLTING PATTERN AND SPACING

In the U.S., Federal Regulation CFR 30 mandates that coal mines follow a systematic pattern of bolting, e.g. 4×4 ft., 5×5 ft., regardless of bolt length. Figure 3.4.1 shows several roof bolt patterns for stratified roof. Usually, bolts are installed vertically, sometimes at an incline. In pattern *a*, bolts are used to create beam building effect, binding several thin layers into a thicker one. In pattern *b*, longer bolts are used and anchored into a stronger main roof to achieve suspension effects. The two bolts in the center of the entry are longer than the ribside bolts. Sometimes staggering bolt length by using longer bolts in the center is more effective, especially when weak clay seams are present. Inclined bolts, either to the inside (pattern *d*) or to the outside (pattern *e*) of the opening, could be useful when installed in combination with regular bolts. In pattern *e*, those inclined bolts are anchored in the roof above the pillars. This helps transfer load to the center of the pillars, reducing the possibility of roof failure along the ribside. For inclined bolts, a special bearing plate must be used such that the plane of the bearing plate is always perpendicular to the bolt.

Fairhurst and Singh (1974) pointed out that following a prescribed systematic bolting pattern would not work for every situation. Sometimes a bolting system pattern works too well by leaving the supported rock intact, resulting in an entire section of supported roof falling as a unit. Thomas (1951) and Jorstad (1967) observed that this type of failure was associated with the artificial fracturing passing through the ends of the longest bolts. Research also showed that fractures are more likely to propagate further as bolt spacing is

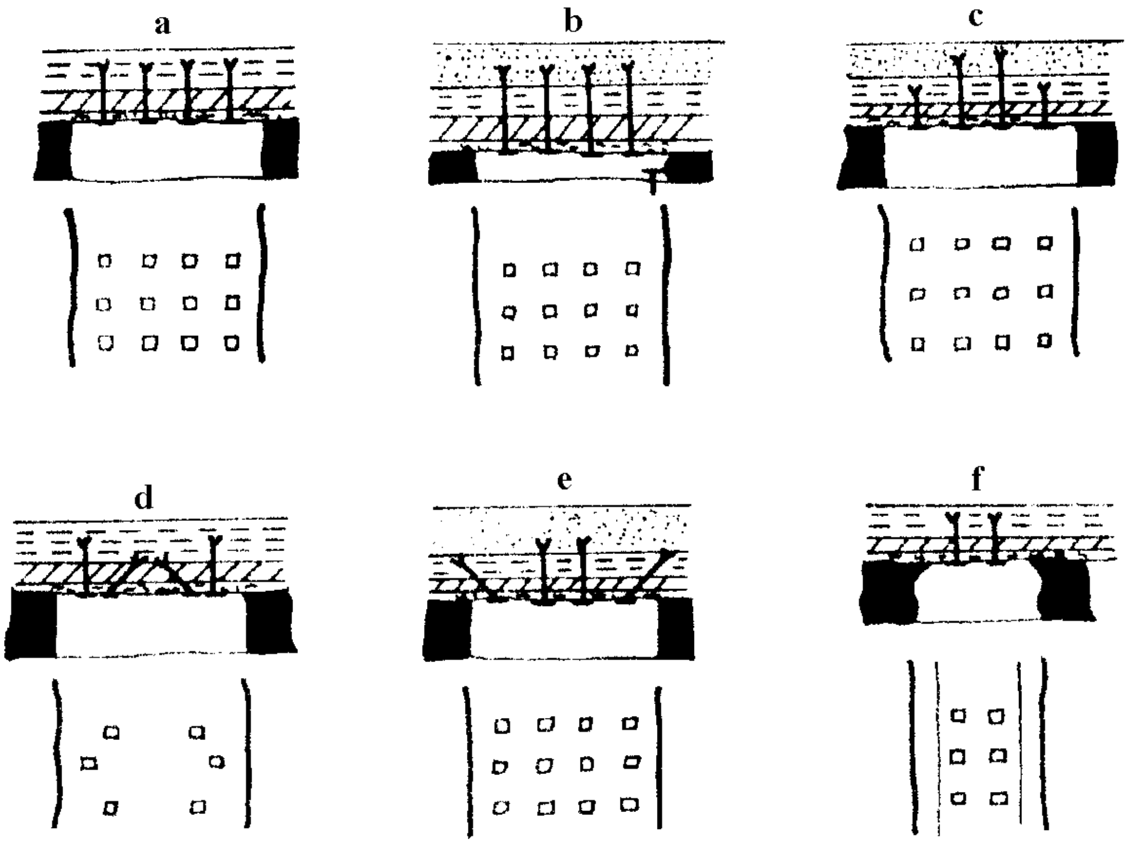


Figure 3.4.1 Typical roof bolting patterns (Panek and McCormick, 1973)

reduced. Generally, the creation of artificial fractures in the bolted roof is insignificant when compared to the additional reinforcement provided by the bolting system.

Because bolt pattern and strata interact, factors affecting bolt spacing include strata thickness, location of weakness plane, roof condition, bolt tension, and bolt characteristics such as yield strength, length, and diameter. According to the photoelastic investigation of Coates and Cochrane (1970), the bolt spacing can be estimated as:

$$b = \frac{2}{3} l \qquad \text{Equation 3.4.1}$$

$$\text{or } b = \frac{2}{9} L \qquad \text{Equation 3.4.2}$$

where

$b =$ Bolt spacing;

$l =$ Bolt length;

$L =$ Roof span (i.e. entry width).

A more complex relationship between bolt spacing and bolt length, which also takes rock density and bolt yield strength into account, is as follow:

$$b = \sqrt{\frac{R_{\max}}{l_{\max} \gamma}} \qquad \text{Equation 3.4.3}$$

where

$R_{max} =$ Bolt yield strength (i.e. maximum carrying capacity of bolt);

$\gamma =$ Rock density;

$l_{max} =$ Bolt maximum length.

It is suggested that if bolt pretension is less than $0.5\sigma_{max}$ of the bolt strength the spacing should be taken as half of the value computed by Equation 3.4.3.

A general rule to check for the bolt spacing is:

- The maximum bolt spacing should be the least of the following three:

1. One half the bolt length;
2. One and one-half the width of critical and potentially unstable rock blocks;
3. 6 feet.

- And the minimum bolt spacing should not be less than 3 feet.

5. BOLT DIAMETER AND ANCHOR CAPACITY

Bolt diameter depends on the yield strength and bolt length and spacing, which determine the load that the bolt is supposed to carry. It can be estimated by the following equation (Biron and Arioglu, 1983):

$$d = 2\sqrt{\frac{SFR}{\pi \sigma_a}} \qquad \text{Equation 3.5.1}$$

where

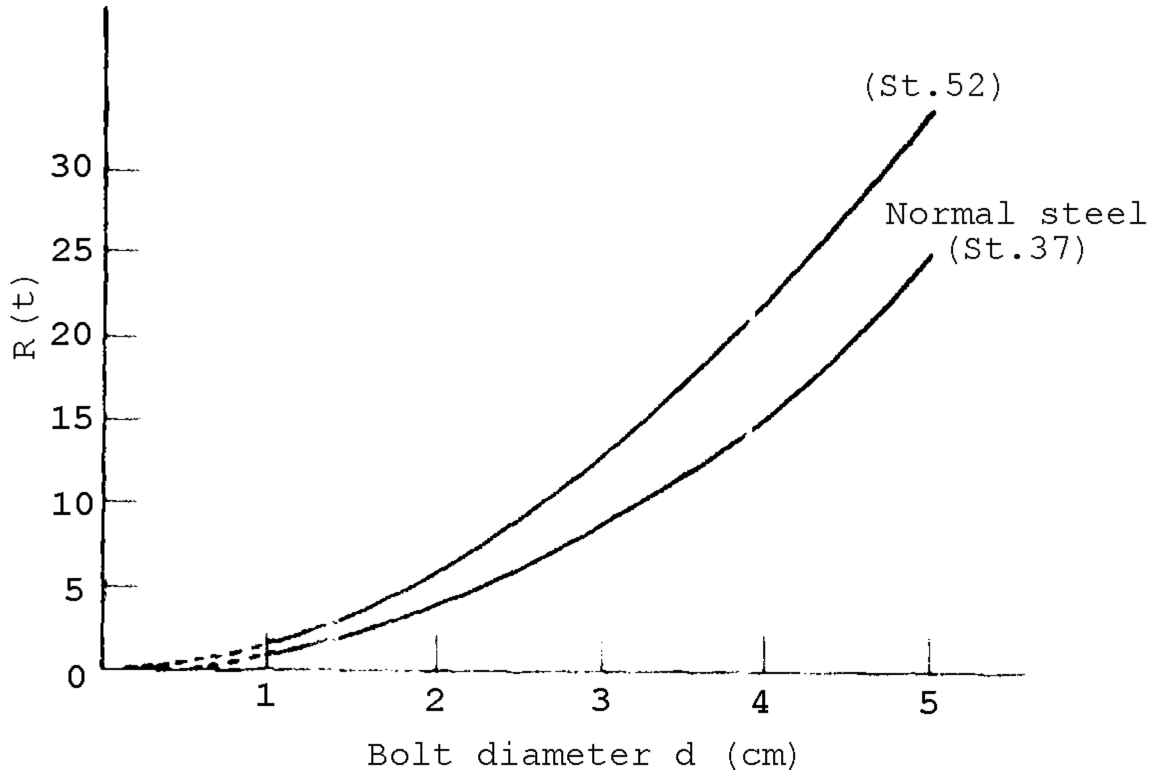
SF = Safety factor, usually 2-4;

R = Allowable axial force in bolt;

σ_a = Yield strength of steel.

Figure 3.5.1 shows the relationship between bolt diameter and carrying capacity for steels St. 37 and St. 52 with a safety factor of 2. In practice, the most commonly used bolts in underground mines have diameters of $\frac{3}{8}$, $\frac{5}{8}$, $\frac{7}{8}$, 1, or $1\frac{1}{8}$ inches.

Bolt carrying capacity is determined not only by the bolt size and strength but also by the anchorage capacity. For mechanical bolts, anchorage capacity depends on how firmly the expansion shells grip against the hole wall. For a shell body of fixed configuration, the maximum gripping force without slippage or anchorage failure is determined by the



1 t = 2205 lbs 1cm = 0.39 inches

Figure 3.5.1 Relationship between bolt diameter and carrying capacity (Biron and Ariogli, 1983)

rock type and the integrity of the rock around the anchorage area. Generally, stronger rocks or rocks of higher integrity provide better anchorage for mechanical bolts. Underground in-situ pull test of roof bolts is the usual way of determining the maximum carrying capacity without anchorage slipping or failure. A good anchorage is defined as one with minimum movement and an anchorage capacity exceeding the bolt yield strength; a fair anchorage is one whose capacity is equal to or slightly exceeds the bolt yield strength; and a poor anchorage is one that moves excessively with loads below the bolt yield strength (Peng, 1984). Figure 3.5.2 shows the anchorage characteristics of mechanical and grouted bolts. It is noticed that the rigidity of resin grouted bolts is higher than that of mechanical bolts as seen from the initial load-deformation curves that have a steeper rise for resin bolts, and thus negligible anchor slippage. For the same type of rock, resin grouted bolts have a higher anchorage capacity. Other research shows that a resin-grouted bolt is at least twice as strong as a mechanical bolt of the same size. For

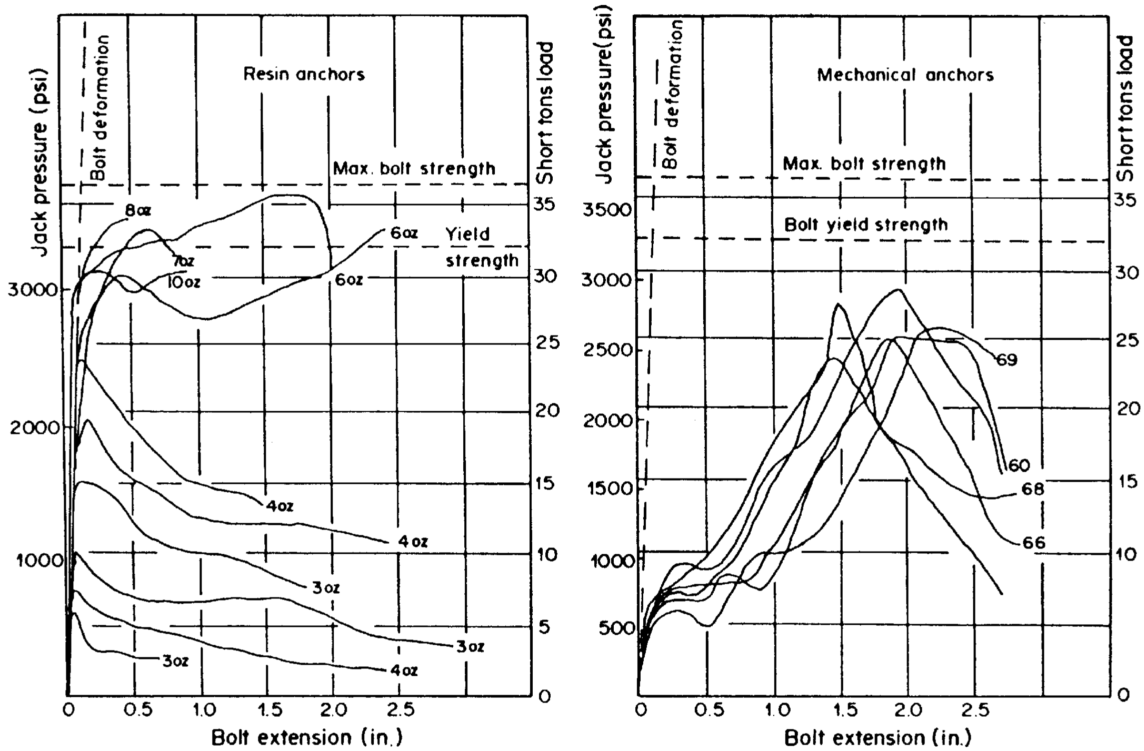


Figure 3.5.2 Anchorage characteristics for rock bolts

(Franklin and Woodfield, 1971)

resin grouted bolts, the clearance between the bolt and the hole is very important. Increasing the hole size or clearance decreases the rigidity and anchorage capacity. An annulus of $\frac{1}{8}$ to $\frac{1}{4}$ inches was found to be the most appropriate.

6. PRETENSION

For point-anchored bolts, the tension applied at installation plays a crucial role for creating suspension, beam building, and keying effects. Based on composite beam theory and an assumption of competent strata with full-load transfer, Lang and Bischoff (1982) derived an equation, as shown below, to estimate the minimum bolt tension to ensure roof stability:

$$T = \alpha \frac{\gamma AR}{k\mu}\left[1 - \frac{c}{\gamma R} - \frac{h\mu}{\gamma R}\right]\frac{1-e^{-\frac{\mu k D}{R}}}{1-e^{-\frac{\mu k L}{R}}} \qquad \text{Equation 3.6.1}$$

where

T = Minimum bolt tension;

α = Ratio depending on time delay of installation of bolts after excavation (0.5 for active reinforcement and 1.0 for passive reinforcement);

γ = Unit weight of the rock;

μ = $\tan\phi$, where ϕ is friction angle of the rock mass;

c = Apparent cohesion of the rock mass;

k = $\frac{1-\sin\phi}{1+\sin\phi}$;

h = Average horizontal stress;

L = Length of bolt;

A = Reinforced area ($s \times s$, where s = bolt spacing);

R = Shear radius of rock column (s/4)

D = Height of stressed rock above opening ($L+s$).

Except for friction angle and cohesion, this equation ignores the geological and rock conditions. It can only be applied when the rock mass is moderately competent. For weak rocks, this equation underestimates the required tension.

Although application of pretension is very important for mechanical bolts and resin anchored bolts to create frictional forces between layers and to diminish interlaminar separation, it is not always true that higher pretension will ensure better stability. Previous research shows that pretension might disturb the anchor zone and produce fractures, leading to roof failure with the bolted formation failing as a unit. It is also believed that higher pretension sometimes generates a stronger beam. However, high pretension does not allow the roof to deform within acceptable extent for stress relief. Extra load is thus transferred to the abutments on both sides of the opening or shear stresses accumulate inside the bolted roof near the ribsides. This stress redistribution may finally lead to pillar failure or cutter roof fall. A general rule for determining the maximum pretension is that pretension should not exceed 60% of the bolt yield strength or 60% of the anchorage capacity.

CHAPTER 4
NUMERICAL MODELING

1. INTRODUCTION

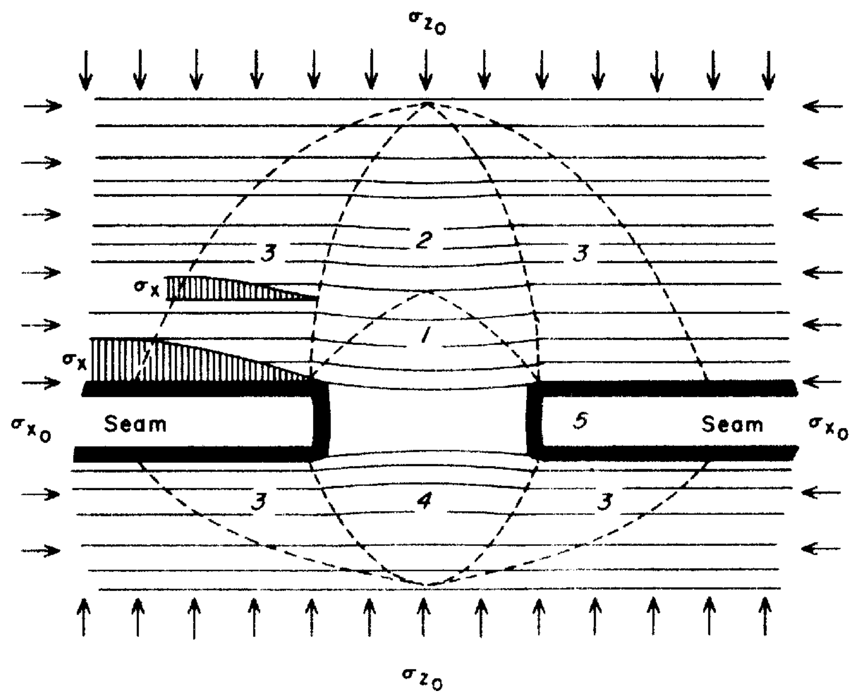
Since the introduction of roof bolting technology into the mining industry as a systematic support method, a great deal of effort has been made to improve its efficiency and reduce its cost. In terms of the number of roof falls and related fatal accidents, it is apparent that roof bolting technology has been advanced tremendously. To meet specific support requirements and various geological conditions, several roof bolting design paradigms were developed. Each paradigm includes a considerable amount of descriptive methodologies and requires the designer to possess experience to make decisions. Even the quantitative methodology produces varying results, depending on geological settings and material properties. Due to the difficulty of precisely modeling those variables numerically, the accuracy of the quantitative results is in question. As a result, the design is usually conservative to ensure the success of roof bolting applications; this in turn wastes manpower during the drilling and installation, as well as the bolt and grout material. There is one thing in common among these paradigms, that is, the ultimate goal is to provide support most sufficiently and effectively with minimum cost. And this goal is achieved by optimizing the bolt length, the number of bolts per row, and the tension applied to bolts on installation.

2. STRESS DISTRIBUTION AROUND AN OPENING

Before an opening is excavated, the underground stress distribution is uniform and the magnitude of vertical stress increases proportionally to the depth. But once an opening is made, the portion of the strata directly above the opening loses its original support and the stress equilibrium is disturbed. As a result, the load of the immediate roof is transferred towards the both sides of the opening, which is called abutments. The roof starts to sag under the gravitational force. If the immediate roof strata are competent, the sag will stop before the roof collapses and the stresses around the opening will eventually reach a new equilibrium. However, in many coal mines, the immediate roofs of entries are not competent enough to sustain the changes of the stress distribution and the interaction induced by mining. These may finally collapse into the opening if they are not sufficiently supported by some means. The stress disturbance creates five influential zones around a rectangular opening (Peng, 1984), as shown in Figure 4.2.1. In Zone 1 and Zone 2, the strata are released from the superincumbent pressure and downward vertical displacements take place due to the gravitational force within the Zones. In Zone 1, interlaminar separation occurs and the magnitude diminishes gradually upwards due to the clamping action of abutment pressure and frictional resistance between the layers. There is no bed separation in Zone 2, but displacement is still noticeable. In Zone 3, both vertical and horizontal stresses build up, forming an arch-shaped high stress zone. Floor heave occurs in Zone 4. In Zone 5, the rib sides expand towards the opening. Both movements in Zones 4 and 5 cause vertical movement of the abutment and strata above, releasing the built-up stress in the arch area significantly. Thus a certain amount of floor heave and rib expansion is beneficial to maintaining the opening's stability.

To maintain the stability of an underground opening, it is essential to keep Zone 1 stable. Roof bolts in this zone force all the bolted layers to sag with the same magnitude; the layers within the bolting range thus act like a solid beam. Ideally, the beam must be strong enough to carry all the weight of strata in Zones 1 and 2 plus the extra load transferred to the zones by mining activities nearby. Building such a beam is actually the ultimate goal of roof bolting where beam building effect is the prevalent mechanism.

It is apparent that the properties such as Young's modulus, Poisson's ratio, frictional angle, lithological conditions such as number of layers within the bolt ranges, and material properties of bedding planes are essential factors affecting the beam strength. In addition, the bolt's tensile strength and shear strength, bolt length and diameter, anchorage capacity, and tension applied to bolt on installation all significantly contribute to the beam strength.



Zone 1 — Bed separation due to differential sag or buckling

Zone 2 — Layers sag without separation. Gravity acts to develop sags in Zone 1 and 2

Zone 3 — Horizontal and vertical pressure build to their undisturbed values

Zone 4 — Floor heave occurs without bed separation

Zone 5 — Seam expands towards excavation because of release of horizontal stress at the ribs

Figure 4.2.1 Different zones developed after opening excavation (After Peng, 1984)

3. NUMERICAL MODELING

The vertical displacement at the mid-span of the opening roof is an indicator of the stability of the opening once the rock material properties are given. The magnitude of displacement is related to the layer spacing in the immediate roof and the frictional forces between the layers. The extra frictional forces created by bolting help limit horizontal movement along the interface, in turn reinforcing the beam building effect. To further investigate the relationship and how these variables affect the magnitude of vertical displacement, a numerical model can be used.

3.1 TOOL SELECTION

Among various available numerical modeling tools, FLAC (Fast Lagrangian Analysis of Continua) was chosen. It is a two-dimensional explicit finite difference program originally designed for modeling geotechnical and mining situations. This program is capable of simulating the behavior of structures built of soil, rock, and other materials that undergo plastic movement after their yield limits are exceeded. Materials are represented by elements. It uses interface elements to represent distinctive planes along which slip and separation can occur and structural elements to model underground structural supports like roof bolts, cables, and shotcrete lining. FLAC also contains a powerful built-in programming language FISH, which can be used to customize the model for specific requirements. These features are desirable in modeling an underground opening and its stability analysis.

FLAC Windows version requires Windows 95/NT and a minimum of 4 MB RAM. Its DOS version can be operated directly on 486 DX or later microcomputers running DOS with at least 4MB RAM.

3.2 ASSUMPTIONS

To simplify the numerical model, the following assumptions are made:

- The roof and floor of the opening are all comprised of the same rock and the layers are of the same thickness. The interfaces between layers are of the same mechanical and material properties. This assumption greatly reduces the complexity of the problem. FLAC is capable of modeling complicated geological condition. And since the model simulates the problem in more detailed way, the accuracy is subsequently higher. But the sacrifice is that the model will take up memory quickly, increasing computation time dramatically by tens and perhaps hundreds of times. On the other hand, uneven layer spacing reduces the model's flexibility; slight changes of bolt parameters may result in considerable FISH code modification.
- Horizontal stresses on both sides of the opening are ignored. In reality, in-situ horizontal stress is ubiquitous underground. It is estimated by the following expression:

$$\sigma_h = \frac{\nu}{1-\nu}\sigma_v \qquad \text{Equation 4.2.1}$$

where

$\nu =$ Poisson's ratio;

$\sigma_v =$ In-situ vertical stress

However, this estimation is not accurate because it assumes that gravity is applied to an elastic mass of material in which lateral movement is prevented. In coal mines, in-situ horizontal stress can be very difficult to determine. In many cases where the in-situ horizontal stress is non geotectonic, its role in roof stability is not significant compared to the other factors. Since there can be a wide deviation of in-situ horizontal stress, using an estimated value does not represent realistic situations. Instead it may create unexpected results.

- The opening is excavated at a the depth of 800 feet. That is, the floor of the opening is 800 feet below the surface. The overburden thickness strongly influences the stability of an opening since it determines the magnitude of vertical stress around the opening. Usually, the in-situ vertical stress is computed as:

$$\sigma_v = g\rho z \qquad \text{Equation 4.3.2}$$

where

g = Gravitational acceleration;

ρ = Average mass density of the overburden strata;

z = Overburden thickness.

- The opening is 20 feet wide by 6 feet height. Bolts are installed immediately after the opening is excavated. The bolts are 5 feet long and 4 feet apart from each other and from the rib side.
- The layer's spacing is such that the number of layer is integral within the bolting range.

3.3 CRITERION OF FAILURE

The simplest and best-known criterion of failure for rocks is the Mohr-Coulomb criterion. In FLAC, Mohr-Coulomb plasticity model is one of the built-in constitutive material models. It is used for materials that yield when subjected to shear loading but the yield strength depends on the major and minor principal stresses only; the intermediate principal stress has no effect on yield. For the Mohr-Coulomb model, the following material properties is required:

- Rock mass density;
- Bulk modulus;
- Shear modulus;
- Friction angle;
- Cohesion;
- Dilation angle; and

- Tensile strength.

Instead of using Young's elasticity modulus and Poisson's ratio directly, FLAC uses bulk modulus and shear modulus. Bulk modulus and shear modulus can be calculated using the following expressions:

$$K = \frac{E}{3(1-2\nu)} \qquad \text{Equation 4.3.3}$$

$$G = \frac{E}{2(1+\nu)} \qquad \text{Equation 4.3.4}$$

where

K = Bulk modulus;

G = Shear modulus;

E = Young's modulus;

ν = Poisson's ratio.

3.4 INTERFACE REPRESENTATION

Interfaces here mean joints, faults, or bedding planes in rock masses. In FLAC, interfaces are characterized by Coulomb sliding and/or tensile separation. They have properties of friction, cohesion, dilation, normal and shear stiffness, and tensile strength. Figure 4.3.1 shows how an interface is conceptually represented in FLAC. For bedding planes to be modeled, slip along the plane and bed separation are the major desired features. Tensile strength and cohesion of the bedding plane are of no significance.

The values of the friction angle, normal and shear stiffness govern the behavior of the interface. Friction angles are easy to obtain from laboratory testing, but stiffnesses are not. It is usually a reasonable estimation to set both normal and shear stiffness to 10 times the equivalent stiffness of the neighboring materials. They can also be approximated by back-calculation from information on the deformability and bedding

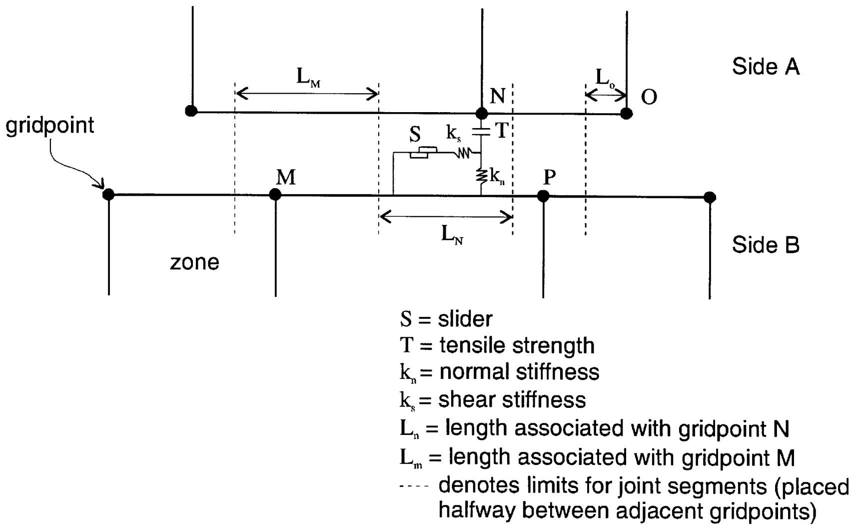


Figure 4.3.1 Conceptual representation of interface of bedded strata

plane structure in the laminated rock mass and the deformability of the intact rock. Normal stiffness, K_n, can be computed as:

$$K_n = \frac{EE_r}{s(E_r - E)} \qquad \text{Equation 4.3.5}$$

where

E = Rock mass Young's modulus;

E_r = intact rock Young's modulus;

s = Bedding spacing.

Shear stiffness, K_s, can be approximated by:

$$K_s = \frac{GG_r}{s(G_r - G)} \qquad \text{Equation 4.3.6}$$

where

G = Rock mass shear modulus;

G_r = Intact rock shear modulus.

3.5 BOLT REPRESENTATION

There are several structural elements in FLAC for simulating structural supports. One of them is called cable element, which is a one-dimensional axial element. It can be point-anchored or grouted to the surrounding material so that the cable element develops forces along its length as the surrounding media deform. A cable element can yield in tension or compression, but it cannot sustain a bending moment. Also, a cable element can be initially pretensioned. Therefore, cable element is the best structural element for modeling rock bolts.

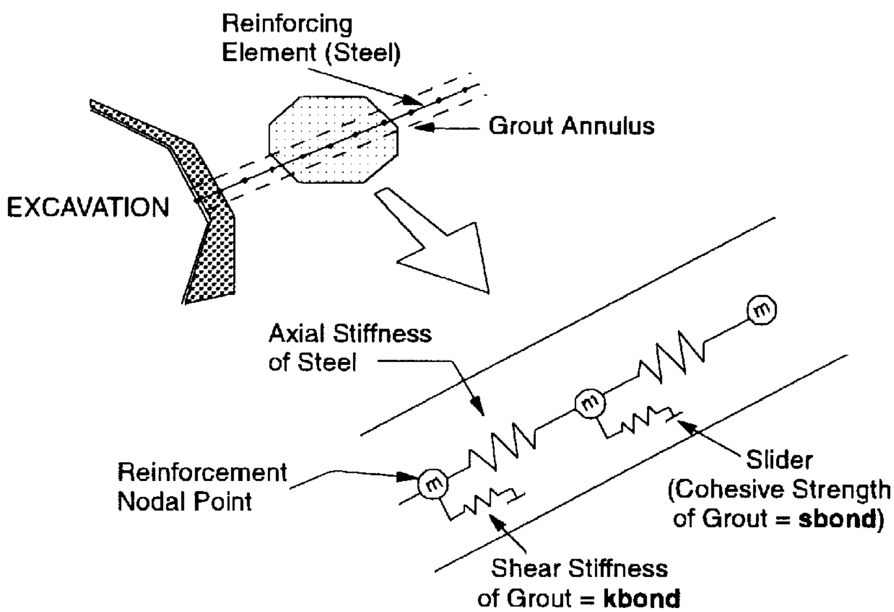


Figure 4.3.2 Conceptual representation of bolt in numerical modeling

Figure 4.3.2 shows the conceptual mechanical representation of fully grouted roof bolt. The axial behavior of a conventional reinforcement system is governed totally by the reinforcing element itself, which is usually steel or deformed rebar. The axial stiffness, which is closely related to the cross-sectional area and the Young's modulus of the bolt, is adequate to describe the axial behavior of the reinforcing element. The shear behavior of the grout annulus, during relative displacement between the bolt/grout interface and the grout/medium interface, is described by the grout shear stiffness, *kbond*, and intrinsic shear strength, *sbond*.

The cable element requires the following input parameters:

- Cross-sectional area, or radius, or diameter of the bolt;
- Young's modulus of the bolt;
- Tensile yield strength of the bolt;
- Compressive yield strength of the bolt;
- Grout shear stiffness, *kbond*;
- Grout intrinsic shear strength, *sbond*; and
- Frictional angle of the grout.

The cross-sectional area, Young's modulus, and yield strength of the bolt are usually specified on bolt manufacturer's specifications. They are thus easy to obtain. The properties associated with the grout are more difficult to estimate. In many cases, the following expression provides a reasonable estimation of *kbond*:

$$kbond = \frac{\pi G}{5\ln\left(1+\frac{2t}{d}\right)} \qquad \text{Equation 4.3.7}$$

where

G = Grout shear modulus;

t = Annulus thickness;

d = Diameter of the bolt.

Given the failure of the bolting system occurs at the grout/rock interface, *sbond* can be approximated by the following equation:

$$sbond = \pi(d+2t)\tau_I Q_B \qquad \text{Equation 4.3.8}$$

where

τ_I = One-half of the uniaxial compressive strength of the weaker of the rock and grout;

Q_B = The quality of the bond between the grout and rock (Q_B = 1 for perfect bonding)

If it is believed that the failure occurs at the bolt/grout interface rather than at the grout/rock interface, then the shear stress should be evaluated at this interface by replacing $(d + 2t)$ by d in Equation 4.3.8.

For partially grouted bolts, the free portion of the bolt does not have any bond with the rock. Under such circumstance, the values of *kbond* and *sbond* are set to zero.

3.6 MODEL CONSTRUCTION

FLAC is a command-driven program, requiring the user to provide a series of commands to control the operation of the program. Figure 4.3.3 is the flowchart of the process for building a FLAC model. In order to set up a model, three fundamental components of a problem must be specified:

- A finite difference grid, which defines the geometry of the problem;
- Constitutive behavior and material properties, which dictate the type of response the model will display upon disturbance such as excavation; and
- Boundary and initial conditions, which define the in-situ state; that is, the conditions before a change or disturbance in the problem is introduced.

After these conditions are defined, the initial equilibrium state is calculated for the model. Once the model responds as expected, alterations, such as making an excavation and putting bolts in the roof are made, and the resulting model response is determined.

FLAC uses an explicit time-marching method to solve a problem. The solution is reached after a series of computation steps called cycle steps. The history of mid-span

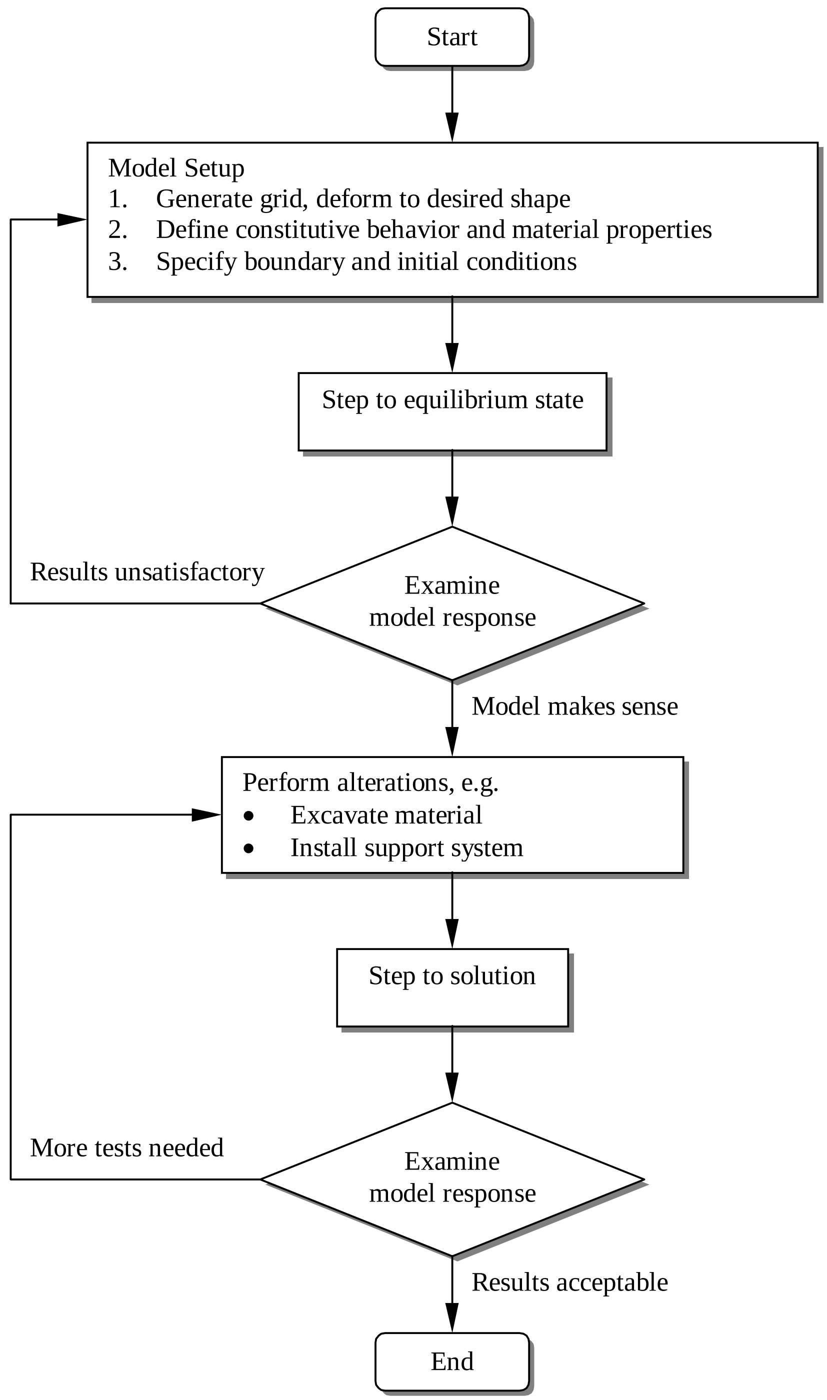


Figure 4.3.3 General steps of model building and problem solving in FLAC

vertical displacement and displacing speed is monitored to see whether the opening is stable or plastic movement occurs. A sharp increase in the magnitude of displacement is an indicator of plastic movement. By contrast, if the displacing speed converges toward zero or the vertical displacement settles down to a certain magnitude, the immediate roof of the opening then reaches its stable state.

The problem being modeled is broken down into two phases. Phase 1 includes grid generation, specification of constitutive behavior and material properties, definition of boundary and initial condition, computation of equilibrium state, and excavation of the opening. Phase 2 includes installation of bolts and problem-solving.

Table 4.3.1 is a list of FISH code for Phase 1, which comprises of five functions. Function *set_model* creates a finite difference grid according to the row and column numbers designated, as shown in Figure 4.3.4, then specifies the constitutive behavior and material for floor, coal, and roof respectively. The number of elements per row or column affects the accuracy of the solution. However, there is a trade-off. Increasing the total number of elements correspondingly enhances the accuracy of the solution but also significantly increases the time required to reach the solution. Function *gen_layers* specifies the geometry of the problem, which is 60 feet wide by 50 feet high, and generates layers based on the input of the number of layers and bedding spacing that are desired. Function *gen_interface* generates interfaces between layers and assigned mechanical properties to the interfaces that govern the behavior of the interfaces. Function *set_init* sets boundary conditions as shown in Figure 4.3.5. All four boundaries are artificial ones. The boundaries are determined such that most of the disturbance caused by the opening excavation can be confined. On both side boundaries only vertical movement is allowed; on the bottom boundary only horizontal movement is allowed. After giving commands for calculating equilibrium state, function *set_init* also applies an uniformly distributed force on the upper boundary, which is equivalent to the magnitude of the superincumbent pressure calculated using Equation 4.3.2. An opening of 20 feet wide by 6 feet high is then excavated near the bottom. Function *ini_value* provides an interface through which element density, number of layers, bedding plane spacing,

Table 4.3.1 FISH source code for modeling underground opening in FLAC

```
title
Model of Bolt Support System of Longwall Gateroad

***********************************************************************
* Function that generates the model of all zones
define set_model
      xgrid=30
      command
         grid xgrid ygrid
      end_command

      xmark = xgrid + 1
      ymark = ygrid + 1

      command
         mod mohr j=1,6
         *Material properties
         prop d = roofDen bulk = roofBulk shear = roofShear j=1,6
         prop coh = roofCoh ten = roofTen fric = roofFric j=1,6
      end_command

         index=8
      loop count (1,numLayers)
         command
            mod mohr j=index
            prop d = roofDen bulk = roofBulk shear = roofShear j=index
            prop coh = roofCoh ten = roofTen fric = roofFric j=index
         end_command
         index = index + 2
      end_loop

      command
         mod mohr j=index,ygrid
         prop d = roofDen bulk = roofBulk shear = roofShear
         j=index,ygrid
         prop coh = roofCoh ten = roofTen fric = roofFric j=index,ygrid
      end_command

end

***********************************************************************
* Function that generates the layers and the element mesh
define gen_layers
      command
         gen 0,0 0,4.57 18.29,4.57 18.29,0 i = 1,xmark j=1,7
      end_command

         ind1=4.57
         ind3=8
      loop count (1,numLayers)
         ind2=ind1+tlayer
         ind4=ind3+1
         command
            gen 0,ind1 0,ind2 18.29,ind2 18.29,ind1 i=1,xmark
            j=ind3,ind4
         end_command
         ind1=ind2
         ind3=ind4+1
      end_loop

      command
```

```
        gen 0,ind2 0,15.24 18.29,15.24 18.29,ind2 i=1,xmark
        j=ind3,ymark
      end_command
end

***********************************************************************
define gen_interface
      command
        int 1 Aside from 1,7 to xmark,7 Bside from 1,8 to xmark,8
        int 1 ks = intKs kn = intKn fric = intFric
      end_command

           ind1=9
           ind2=10
           maxCount = numLayers + 1
      loop count (2,maxCount)
           command
             int count Aside from 1,ind1 to xmark,ind1 Bside from
             1,ind2 to xmark ind2
             int count ks = intKs kn = intKn fric = intFric
           end_command
           ind1=ind1+2
           ind2=ind2+2
      end_loop
end

***********************************************************************
define set_init
      command
      *Initialize boundary movement
        fix x i = 1
        fix x i = xmark
        fix x y j =1

        set grav=10
        solve
        set large
        ini xdis = 0 ydis = 0

      *initialize stress condition
        apply pres = applyPre j = ymark
      end_command

           node = (18 * xgrid) / 120
           node1 = xgrid / 2 - node
           node2 = xgrid /2 + node + 1
      command
        mod null i=node1, node2 j=5,6
        plot hold grid

        his yd xvel yvel i = 16 j = 8
        his yd xvel yvel i = 16 j = 9
        his yd xvel yvel i = 16 j = 10
      end_command
end

*********************************************************************
define ini_value
      ; Set roof layer spacing
           ygrid = 30
           numLayers = 5
           tlayer = 0.305

      ; Material properties of surrounding rock
```

```
            YongE = 8000
            Poisson = 0.25
            OverBurd = 253
            roofCoh = 16e6
            roofTen = 1e6
            roofFric = 30

            roofDen = 2550
            applyPre = OverBurd * 20000
            roofBulk = YongE * 1000000 / (3*(1 - 2 * Poisson))
            roofShear = YongE * 1000000 /(2*(1 + Poisson))

      ; Mechanical properties of interface
            intKs = 5e7
            intKn = 2e9
            intFric = 30
end

***********************************************************************
* Beginning of modeling
ini_value
set_model
gen_layers
gen_interface
set_init
```

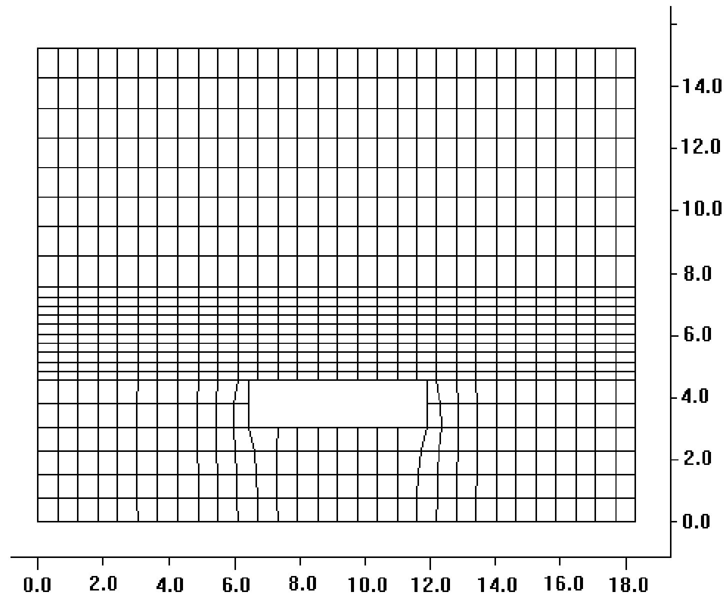


Figure 4.3.4 Initial stage of an underground opening

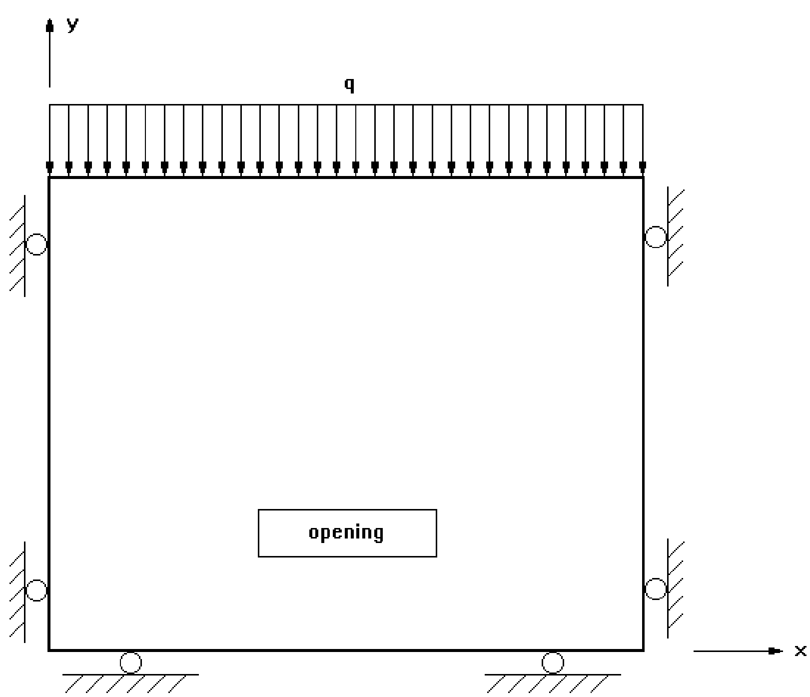


Figure 4.3.5 Boundary and initial stress conditions around the opening

material properties of surrounding rocks, and the mechanical properties of interfaces can be easy modified.

Table 4.3.2 is a list of FISH code for Phase 2. There is only one function named *groutAnchorBolt*. It defines bolt parameters such as bolt length, bolt spacing, grout length, and pretension. The bolt is simulated as partially grouted steel bolt, while the cable element is divided into 6 segments for better accuracy. The grout properties and the capacity of the bolts are selected to ensure that the bolting system itself does not fail.

3.7 EXPERIMENTATION

3.7.1 Parameter selection

The stability of an underground opening depends on many parameters. Material properties of surrounding rock, mechanical properties of bedding planes, geometry and material properties of bolt, bolt layout and pretension, and overburden thickness all collectively contribute. These parameters are carefully selected to make the model response and behave as closely to reality as practical. Table 4.3.3 lists all the parameters with magnitudes in both English units and Metric units. Metric units are used in FLAC computation.

Parameters on Table 4.3.3 remain unchanged once they are validated by the trial simulation of the model; so do their influences on the opening stability. Thus, the displacement and displacing speed of the mid-span point of the immediate roof are dependent on two variables: the number of layers within the bolting range and pretension applied to bolt on installation.

3.7.2 Validation of model response

Even though all the parameter values are representatives of real situation, the FLAC model may not necessarily respond as it does in reality. Adjustment of parameter values must be made such that the model eventually makes sense by testing the model by trial-

Table 4.3.2 FISH source code for modeling grouted anchored bolts in FLAC

```
***********************************************************************
*Install rock bolts
define groutAnchorBolt
      boltLen = 1.52
      groutLen = 0.4
      spacing = 1.22
      preTen = 30000

      xBegin = 7.32
      yBegin = 4.57 + boltLen
      yEnd = yBegin - groutLen
      gBegin = 13
      nodeNum = 6

      loop count (1,4)
            command
               struct cable beg xBegin yBegin end xBegin yEnd seg 5
               prop 1
               struct cable beg grid gBegin,8 end node nodeNUm seg 1
               prop 2 tension preTen
            end_command
               xBegin = xBegin + spacing
               gBegin = gBegin + 2
               nodeNum = nodeNUm + 7
      end_loop

      command
            struct prop 1 a 5e-4 e 8.2e10 y 5e9 kbond 1.15e10 sbond
            9.8e5
            struct prop 2 a 5e-4 e 8.2e10 y 5e9 kbond 1 sbond 1
      end_command
end

***********************************************************************
groutAnchorBolt
```

Table 4.3.3 List of values of parameters affecting underground opening stability

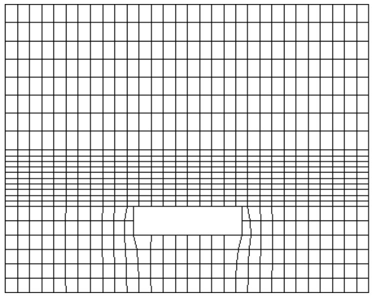
	Parameter name	Value	
		English unit	Metric unit
Roof and floor	Density	159 lbs/ft^3	2550 kg/m^3
	Young's modulus	1,160,000 psi	8×10^9 Pa
	Poisson's ratio	0.25	0.25
	Bulk modulus	773,000 psi	5.33×10^9 Pa
	Shear modulus	464,100 psi	3.20×10^9 Pa
	Cohesion	2,320 psi	1.6×10^7 Pa
	Friction angle	30°	30°
	Tensile strength	150 psi	1.0×10^6 Pa
Bedding plane	Normal stiffness	184 psi/inch	5.0×10^7 Pa/m
	Shear stiffness	7,370 psi/inch	2.0×10^9 Pa/m
	Friction angle	30°	30°
	Bedding spacing	1 foot	0.305 m
Grout	kbond	1,667,875 lbs/inch/inch	1.15×10^{10} N/m/m
	sbond	142 lbs/inch/inch	9.8×10^5 N/m/m
	Length	1.31 feet	0.4 m
Bolt	Length	5.0 feet	1.52 m
	Spacing	4.0 feet	1.22 m
	Diameter	1 inch	0.0252 m
	Young's modulus	11,893,280 psi	8.2×10^{10} Pa
	Tensile strength	725,200 psi	5.0×10^9 Pa

and-error. The expected responses of the model include that the immediate roof will collapse without appropriate support, that bed separation occurs in Zone 1 as described in Figure 4.2.1, and that all five zones developed around the opening vicinity area after excavation.

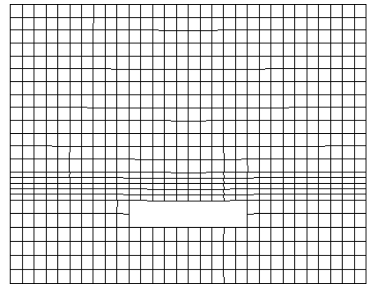
Figure 4.3.6 shows the process of the model responses in terms of the computation cycle steps. In some sense, the computation cycle steps represent the elapse of time, though every cycle step does not necessarily equal to a certain amount of time. Bed separation is noticeable after 3,000 cycle steps and plastic movement of the lowest layer has already token place at cycle step 4,000. In the FLAC model, plastic movement means collapsing; that is, roof fall starts happening at about cycle step 4,000. Also, rib expansion and floor heave take place at a very early stage, but the magnitude is not considered significant even after the immediate roof falls. Figures 4.3.7 through 4.3.9 show the vertical displacement of the whole area being modeled after 1,000, 2,000, and 6,000 cycle steps, respectively. It is apparent that the model with parameter values on Table 4.3.3 responds as designed. Therefore, further experiments are conducted.

3.7.3 Running the model

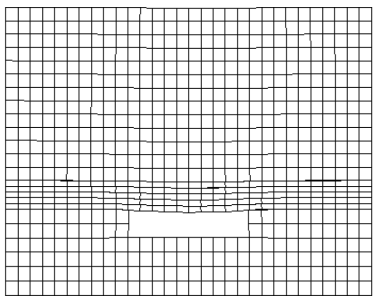
After the model responds as expected, the next step is to install the bolting system and investigate the impact of bolts on the stability of the immediate roof. In this research, the main objective is to find out how bedding plane spacing and pretension affect the deflection of the immediate roof. Therefore, the displacement and displacing speed of the mid-span point of the immediate roof are monitored with the number of layers within bolting range and the pretension on the bolts as independent variables. Eight different layer numbers are chosen. They are 5, 6, 7, 8, 9, 10, 11, and 12. Seventeen distinctive pretension is applied to the bolts and the values are 0, 0.45, 0.90, 1.35, 2.25 3.37, 4.50, 5.62, 6.74, 7.87, 8.99, 10.12, 11.24, 12.36, 13.49, 15.74, 17.98, 22.48, and 44.96 klbs. For each combination of the number of layers and pretension, the histories of displacement and displacing speed of the mid-span are computed. The amount of displacement and the magnitude of displacing speed are recorded every 10 computation



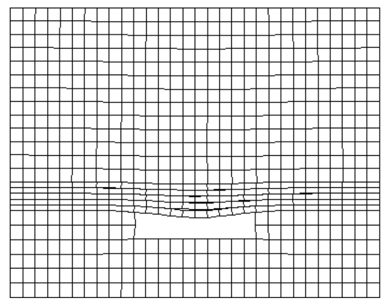
(a) 0 cycle step



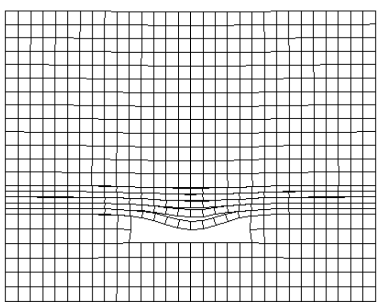
(b) 1000 cycle steps



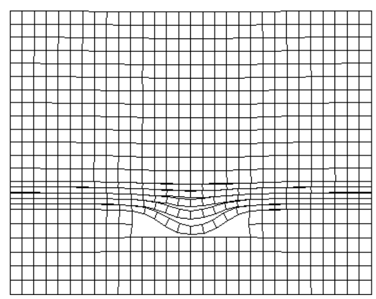
(c) 2,000 cycle steps



(d) 3,000 cycle steps



(e) 4,000 cycle steps



(f) 5,000 cycle steps

Figure 4.3.6 Process of roof failure without support

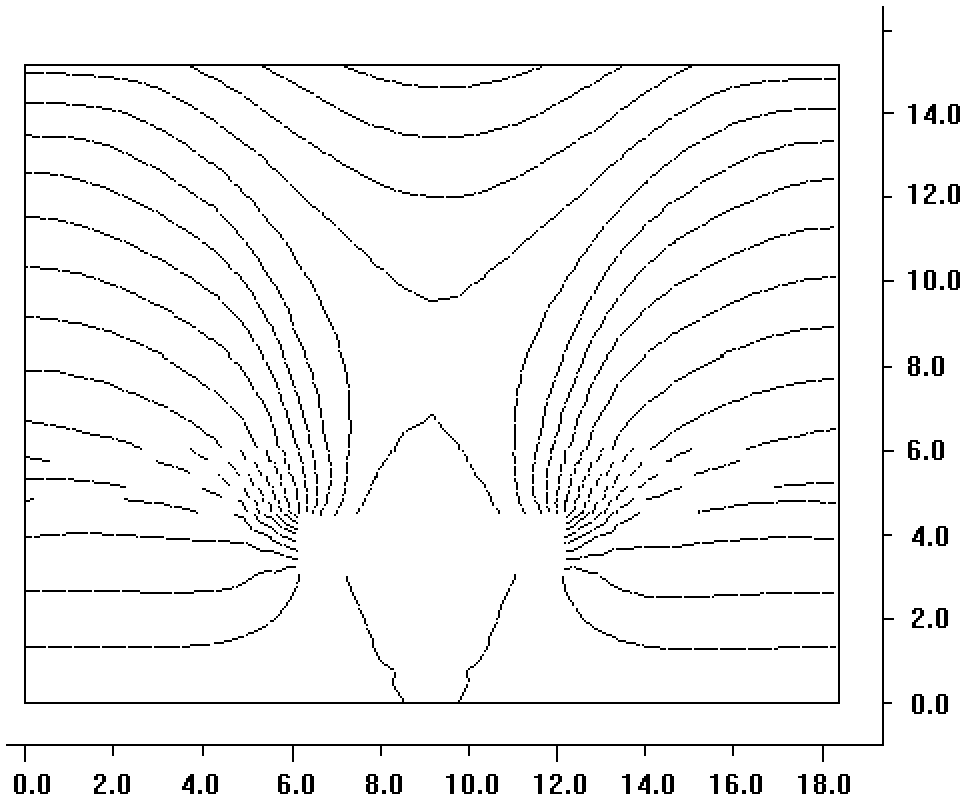


Figure 4.3.7 Vertical displacement after 1000 cycle steps

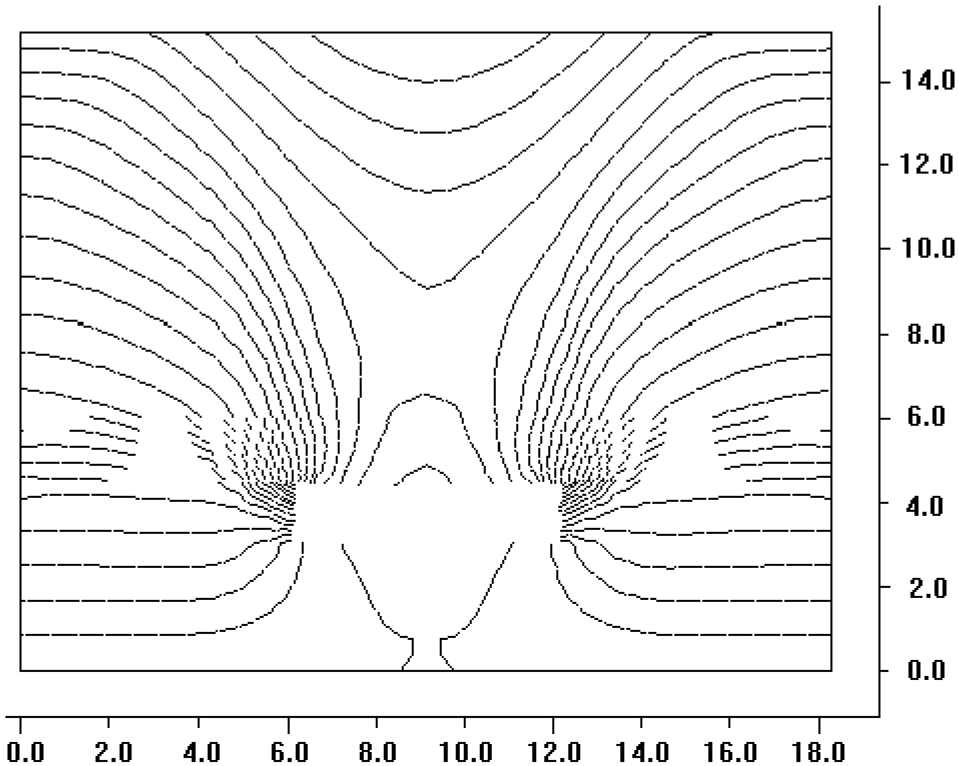


Figure 4.3.8 Vertical displacement after 2000 cycle steps

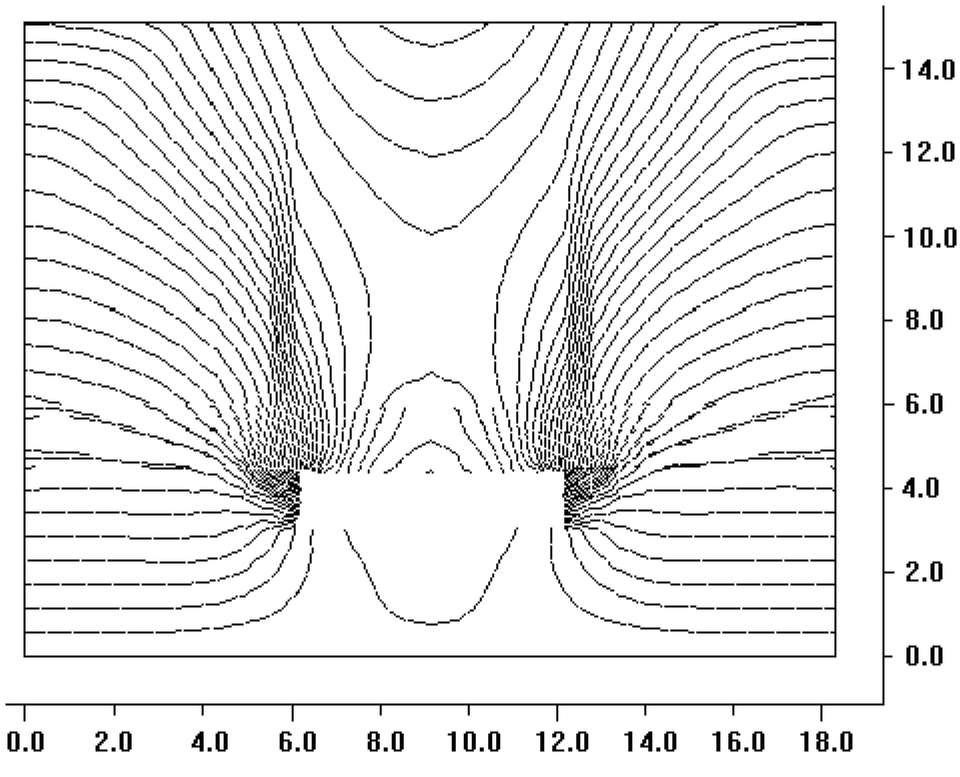


Figure 4.3 9 Vertical displacement after 6000 cycle steps

cycle steps. Once the displacing speed or the increment of displacement between cycle steps converges toward zero, the model reaches a new equilibrium state and the immediate roof remains stable, as shown in Figure 4.3.10. The displacement at this state is referred to as final roof displacement. Figures 4.3.11 through 4.3.18 show the mid-span displacement and displacing speed histories with the pretension having a constant value of 6.75 klbs and with the number of layers changing from 5, 6, 7, 8, 9, 10, 11, to 12. Figures 4.3.19 through 4.3.26 show the relationship of the mid-span final displacement and pretension at a certain number of layers within bolting range. Figures 4.27 through 4.3.30 depict the relationship of the mid-span final displacement and the number of layers with pretension of 0.0, 6.75, 11.25, and 44.95 klbs, respectively.

3.8 ANALYTICAL RESULTS

Based on the experiments conducted, it was observed that the selected bolting support system is competent to sustain the opening's stability. Regardless of the number of layers and pretension applied on installation the immediate roof remains stable, though the magnitudes of roof final displacement vary slightly for each combination of the number of layers and the pretension.

From Figures 4.3.11 through 4.3.18 and all the other recorded displacement and displacing speed histories, it is noticed that both displacement and displacing speed follow same patterns respectively. The immediate roof experiences a sharp deflection immediately after the excavation is made, followed by a transient moment of negligible movement. Before reaching the stable state, the immediate roof continues to sag with a slower and decreasing yet fluctuating speed. This pattern indicates that installing bolts immediately after excavation is beneficial. The sudden displacement helps build up axial force along the bolt, which is essential for beam building effect to develop. Manual pretension on installation is thus not necessarily required unless the immediate roof contains too many fractures that may offset the force produced naturally. Even though manual pretension is required, it only acts as a supplement to the force created by the

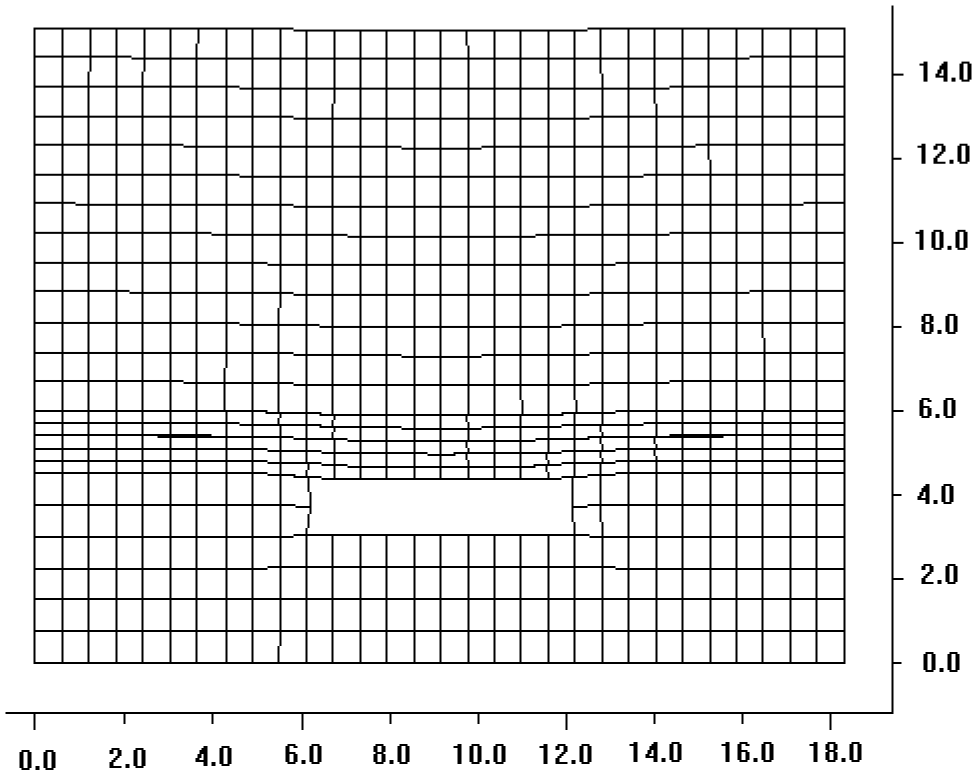


Figure 4.3.10 Final stage of the opening with bolting support

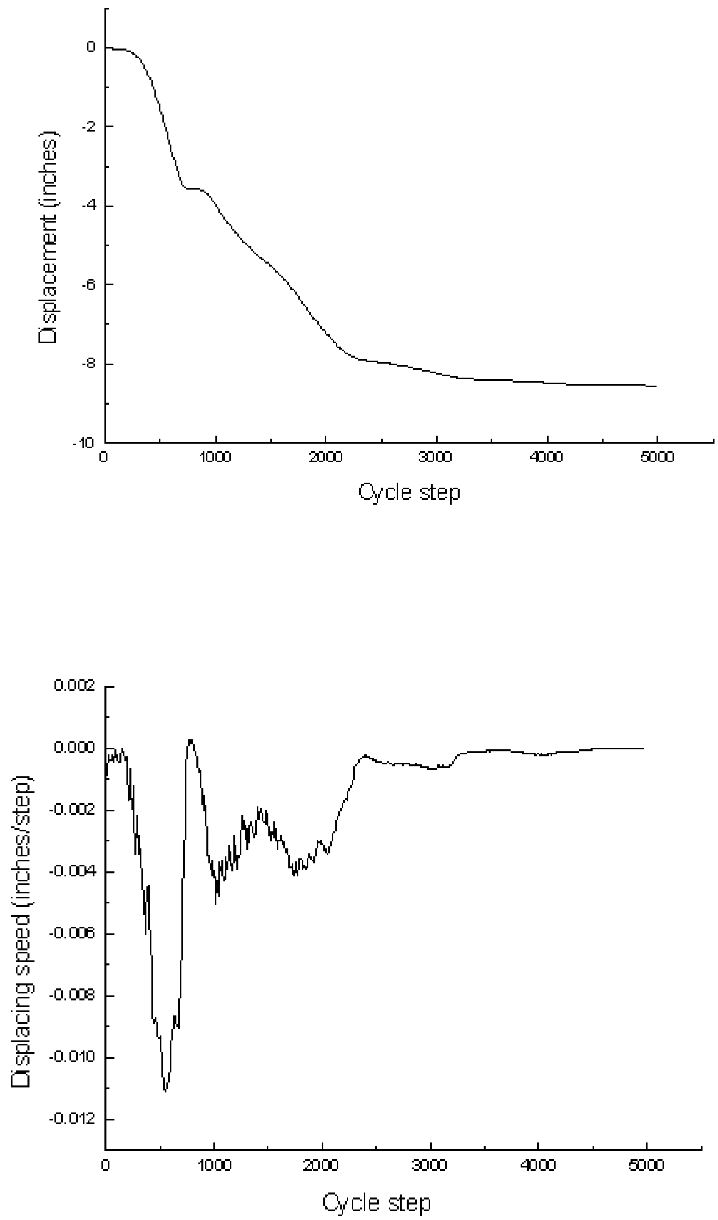
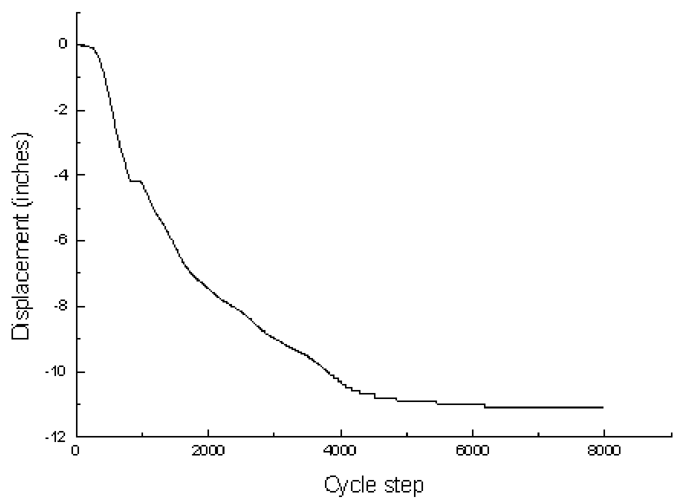


Figure 4.3.11 Mid-span displacement and displacing speed history (bolt length = 5 ft, number of layers = 5, pretension = 6.75klbs)



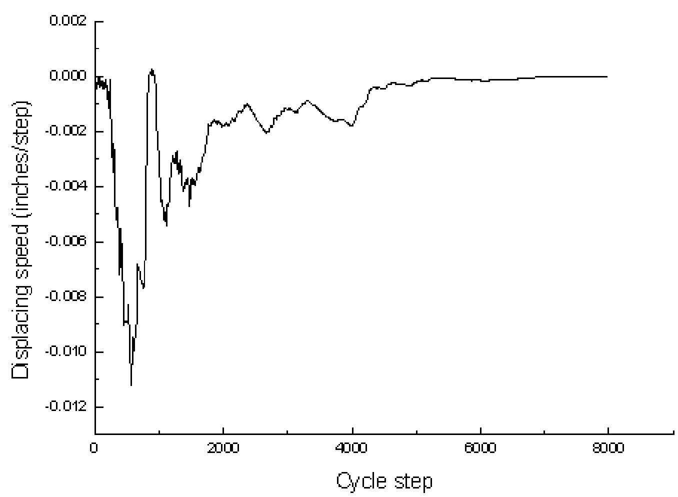


Figure 4.3.12 Mid-span displacement and displacing speed history (bolt length = 5 ft, number of layers = 6, pretension = 6.75klbs)

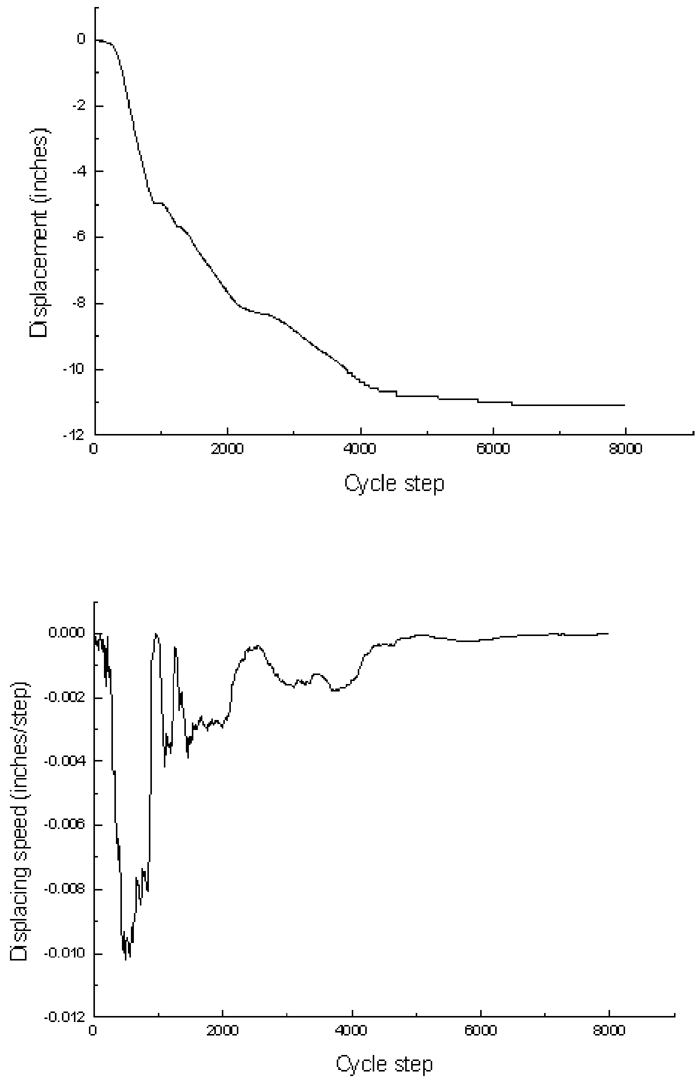
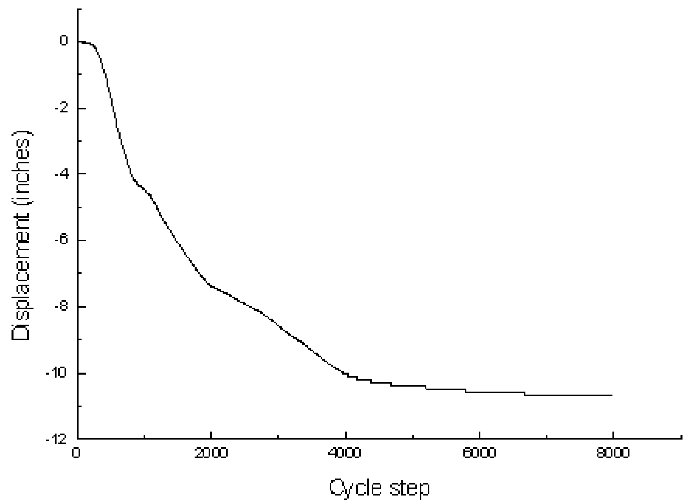


Figure 4.3.13 Mid-span displacement and displacing speed history (bolt length = 5 ft, number of layers = 7, pretension = 6.75klbs)



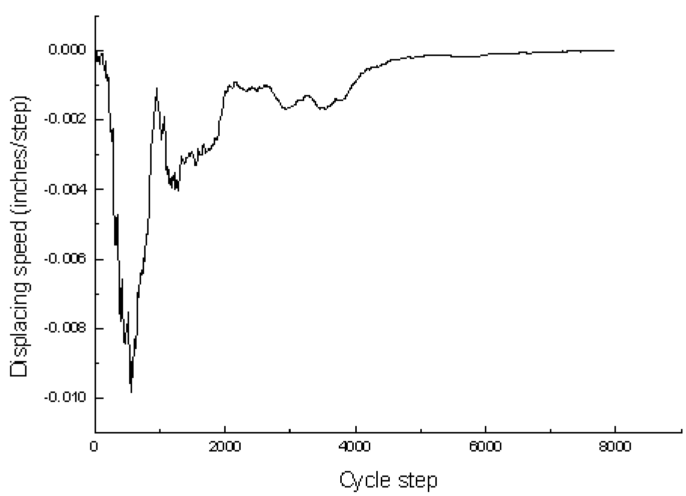
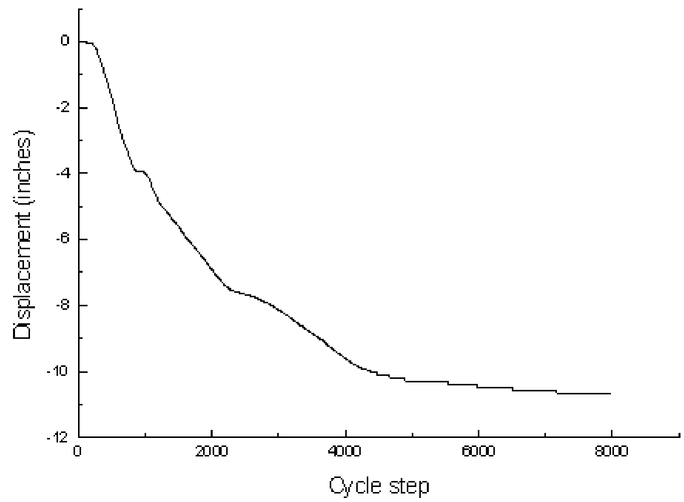


Figure 4.3.14 Mid-span displacement and displacing speed history (bolt length = 5 ft, number of layers = 8, pretension = 6.75klbs)



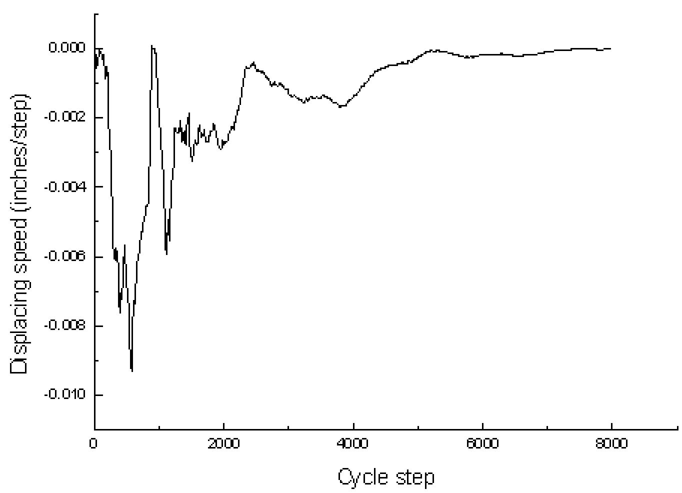


Figure 4.3.15 Mid-span displacement and displacing speed history (bolt length = 5 ft, number of layers = 9, pretension = 6.75klbs)

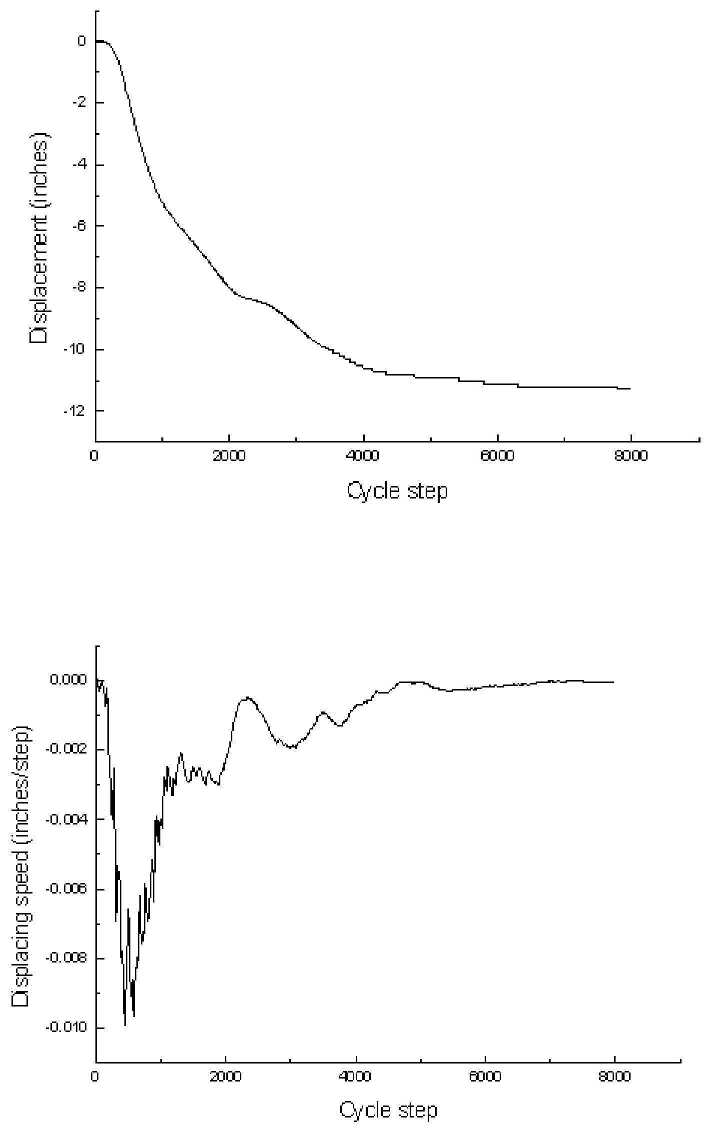
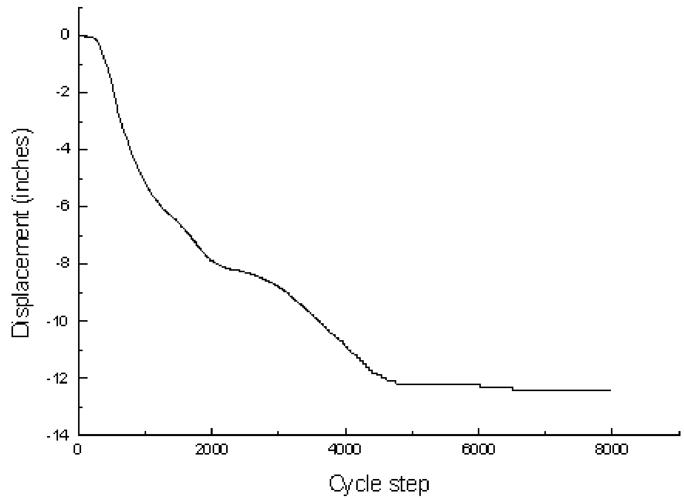


Figure 4.3.16 Mid-span displacement and displacing speed history (bolt length = 5 ft, number of layers = 10, pretension = 6.75klbs)



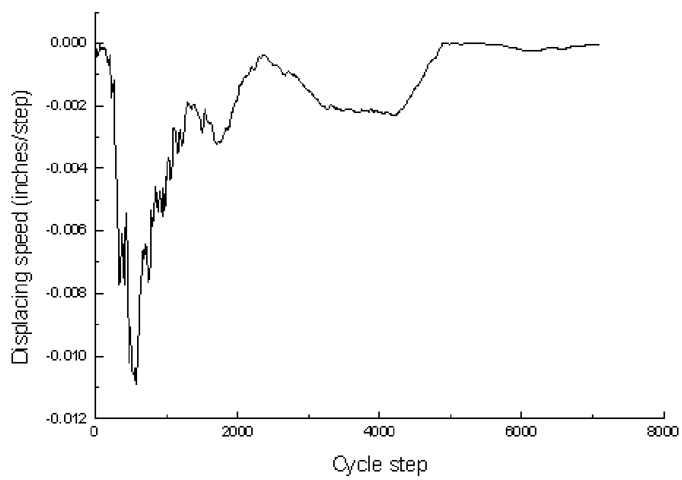


Figure 4.3.17 Mid-span displacement and displacing speed history (bolt length = 5 ft, number of layers = 11, pretension = 6.75klbs)

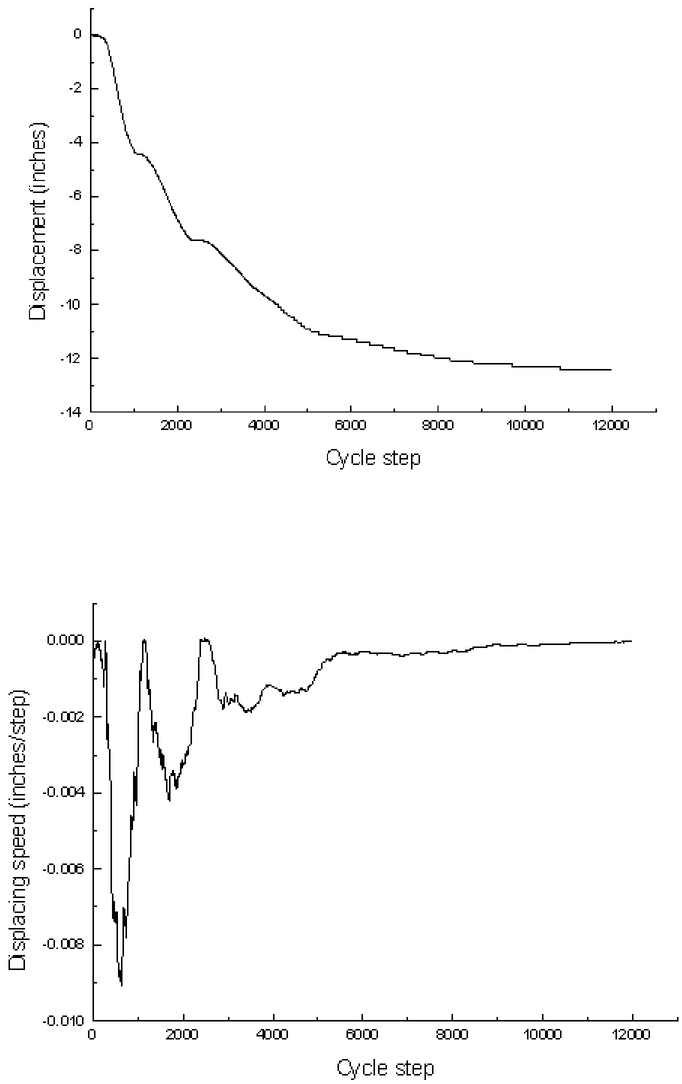


Figure 4.3.18 Mid-span displacement and displacing speed history (bolt length = 5 ft, number of layers = 12, pretension = 6.75klbs)

Table 4.3.4 Final displacement of mid-span versus bolt pretension (bolt length = 5 ft, number of layers = 5)

Pretension (klbs)	Displacement (inches)	Pretension (klbs)	Displacement (inches)
0.00	-8.61	8.99	-8.56
0.45	-8.59	10.12	-8.60
0.90	-8.57	11.24	-8.57
1.35	-8.67	12.36	-8.60
2.25	-8.53	13.49	-8.54
3.37	-8.61	15.74	-8.54
4.50	-8.56	17.98	-8.50
5.62	-8.56	22.48	-8.49
6.74	-8.55	44.96	-8.43
7.87	-8.57		

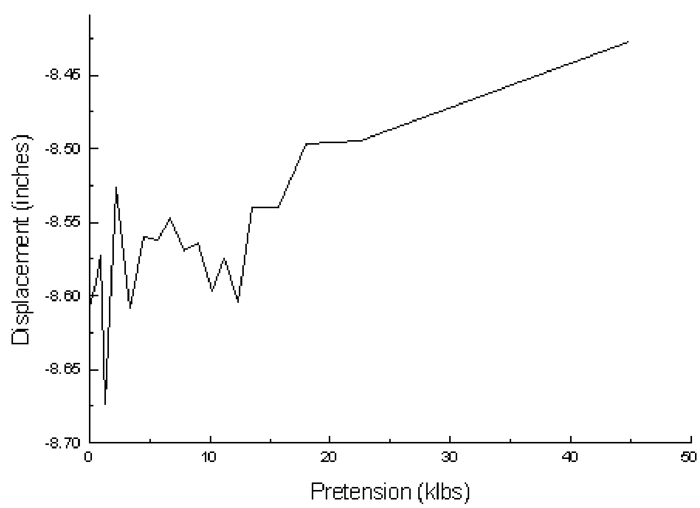


Figure 4.3.19 Correlation between final displacement and bolt pretension

(bolt length = 5 ft, number of layers = 5)

Table 4.3.5 Final displacement of mid-span versus bolt pretension

(bolt length = 5 ft, number of layers = 6)

Pretension (klbs)	Displacement (inches)	Pretension (klbs)	Displacement (inches)
0.00	-11.14	10.12	-11.11
1.12	-11.09	11.24	-11.05
2.25	-11.26	12.36	-10.95
3.37	-11.13	13.49	-11.24
4.50	-11.14	15.74	-11.10
5.62	-11.21	17.98	-11.26
6.74	-11.15	22.48	-10.97
7.87	-11.20	44.96	-10.84
8.99	-11.12		

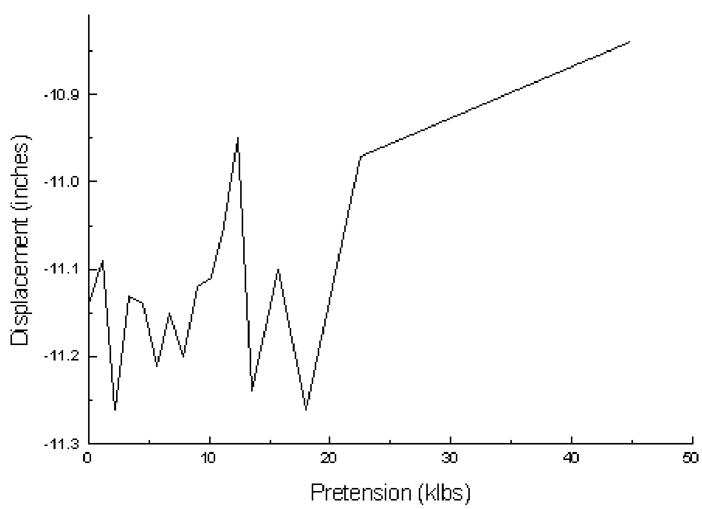


Figure 4.3.20 Correlation between final displacement and bolt pretension

(bolt length = 5 ft, number of layers = 6)

Table 4.3.6 Final displacement of mid-span versus bolt pretension

(bolt length = 5 ft, number of layers = 7)

Pretension (klbs)	Displacement (inches)	Pretension (klbs)	Displacement (inches)
0.00	-11.22	10.12	-10.98
1.12	-11.23	11.24	-11.07
2.25	-11.05	12.36	-11.09
3.37	-10.96	13.49	-11.22
4.50	-10.90	15.74	-11.01
5.62	-10.97	17.98	-11.09
6.74	-11.13	22.48	-10.86
7.87	-11.13	44.96	-10.83
8.99	-10.95		

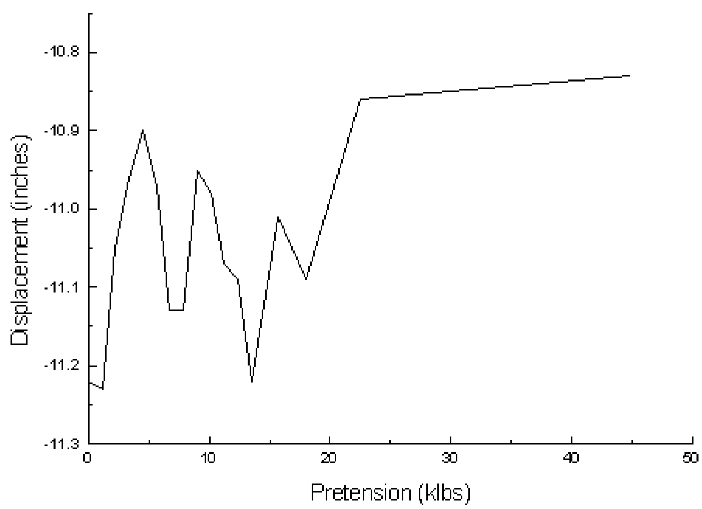


Figure 4.3.21 Correlation between final displacement and bolt pretension

(bolt length = 5 ft, number of layers = 7)

Table 4.3.7 Final displacement of mid-span versus bolt pretension (bolt length = 5 ft, number of layers = 8)

Pretension (klbs)	Displacement (inches)	Pretension (klbs)	Displacement (inches)
0.00	-10.91	10.12	-10.81
1.12	-10.75	11.24	-10.68
2.25	-10.99	12.36	-10.85
3.37	-10.77	13.49	-10.74
4.50	-10.82	15.74	-10.80
5.62	-10.74	17.98	-10.67
6.74	-10.70	22.48	-10.76
7.87	-10.94	44.96	-10.52
8.99	-10.79		

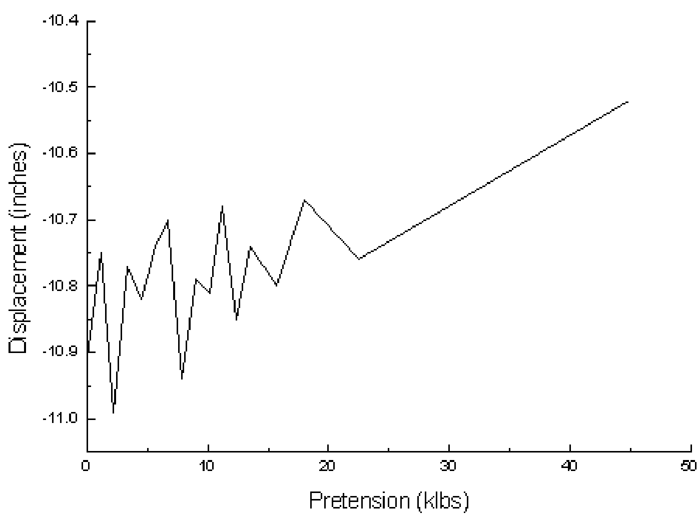


Figure 4.3.22 Correlation between final displacement and bolt pretension (bolt length = 5 ft, number of layers = 8)

Table 4.3.8 Final displacement of mid-span versus bolt pretension (bolt length = 5 ft, number of layers = 9)

Pretension (klbs)	Displacement (inches)	Pretension (klbs)	Displacement (inches)
0.00	-10.71	10.12	-10.61
1.12	-10.56	11.24	-10.47
2.25	-10.59	12.36	-10.82
3.37	-10.59	13.49	-10.67
4.50	-10.70	15.74	-10.52
5.62	-10.71	17.98	-10.54
6.74	-10.68	22.48	-10.44
7.87	-10.71	44.96	-10.42
8.99	-10.62		

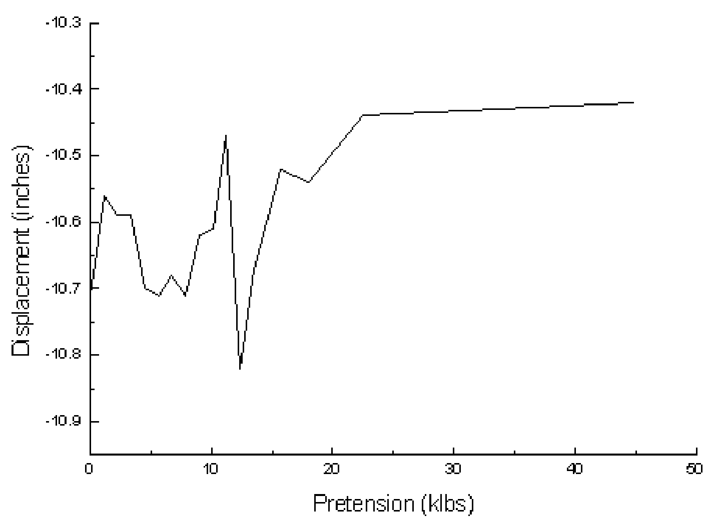


Figure 4.3.23 Correlation between final displacement and bolt pretension (bolt length = 5 ft, number of layers = 9)

Table 4.3.9 Final displacement of mid-span versus bolt pretension (bolt length = 5 ft, number of layers = 10)

Pretension (klbs)	Displacement (inches)	Pretension (klbs)	Displacement (inches)
0.00	-11.21	10.12	-11.28
1.12	-11.22	11.24	-11.31
2.25	-11.22	12.36	-11.27
3.37	-11.23	13.49	-11.27
4.50	-11.23	15.74	-11.32
5.62	-11.20	17.98	-11.22
6.74	-11.26	22.48	-11.24
7.87	-11.21	44.96	-11.22
8.99	-11.21		

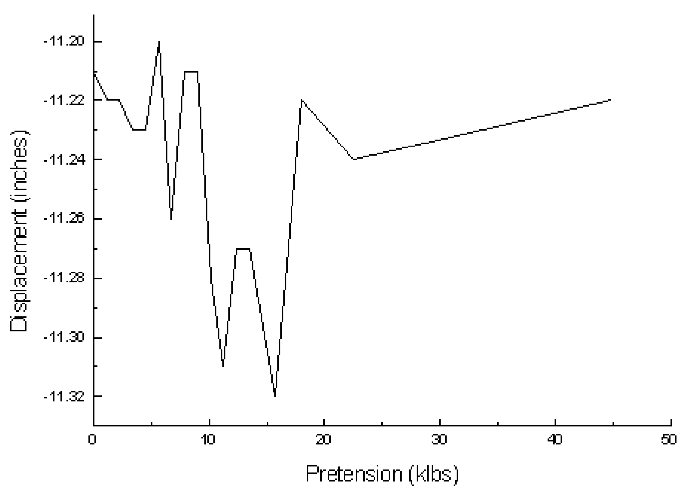


Figure 4.3.24 Correlation between final displacement and bolt pretension (bolt length = 5 ft, number of layers = 10)

Table 4.3.10 Final displacement of mid-span versus bolt pretension (bolt length = 5 ft, number of layers = 11)

Pretension (klbs)	Displacement (inches)	Pretension (klbs)	Displacement (inches)
0.00	-12.53	10.12	-12.43
1.12	-12.44	11.24	-12.42
2.25	-12.39	12.36	-12.46
3.37	-12.53	13.49	-12.32
4.50	-12.49	15.74	-12.50
5.62	-12.49	17.98	-12.42
6.74	-12.45	22.48	-12.44
7.87	-12.34	44.96	-12.45
8.99	-12.33		

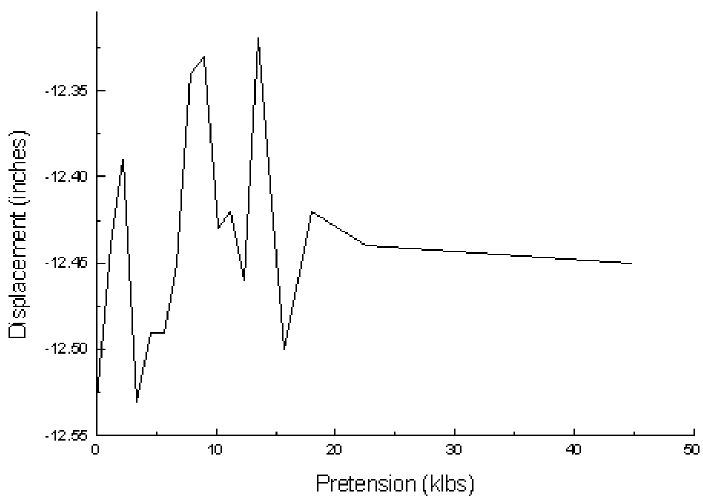


Figure 4.3.25 Correlation between final displacement and bolt pretension (bolt length = 5 ft, number of layers = 11)

Table 4.3.11 Final displacement of mid-span versus bolt pretension

(bolt length = 5 ft, number of layers = 11)

Pretension (klbs)	Displacement (inches)	Pretension (klbs)	Displacement (inches)
0.00	-12.31	14.69	-12.35
2.45	-12.32	17.14	-12.31
4.90	-12.35	19.58	-12.31
7.34	-12.40	24.48	-12.41
9.79	-12.44	48.96	-12.32
12.24	-12.38		

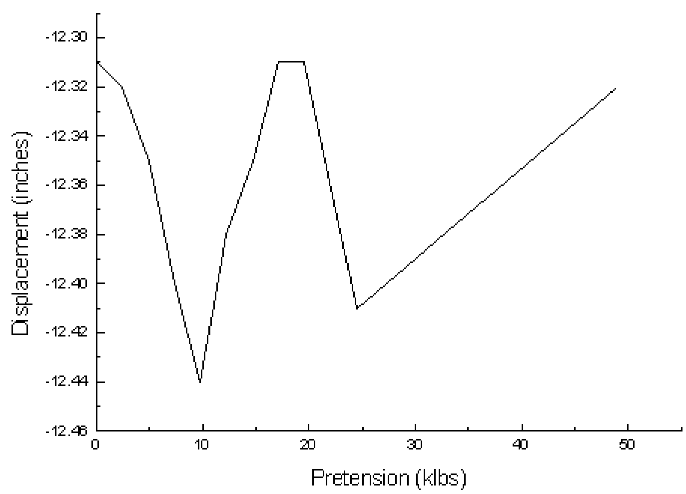


Figure 4.3.26 Correlation between final displacement and bolt pretension

(bolt length = 5 ft, number of layers = 12)

Table 4.3.12 Final displacement of mid-span versus number of layers

(bolt length = 5 ft, pretension = 0)

Number of layers	Displacement (inches)	Number of layers	Displacement (inches)
5	-8.61	9	-10.71
6	-11.14	10	-11.22
7	-11.22	11	-12.52
8	-10.91	12	-12.32

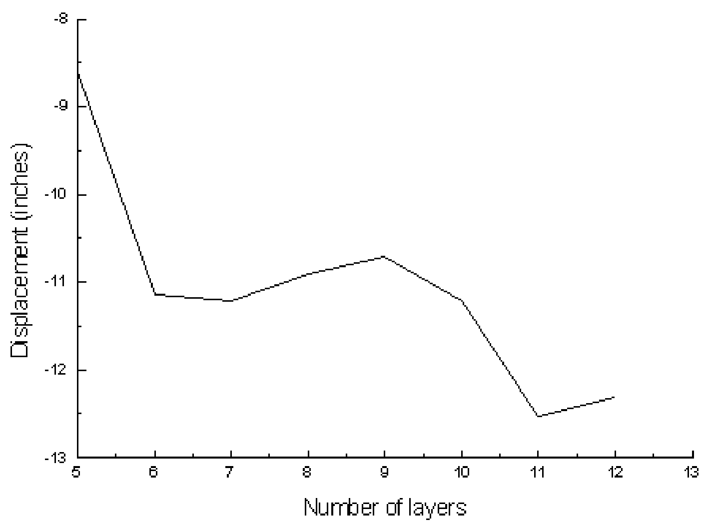


Figure 4.3.27 Correlation between final displacement and number of layers

(bolt length = 5 ft, pretension = 0)

Table 4.3.13 Final displacement of mid-span versus number of layers (bolt length = 5 ft, pretension = 6.75 klbs)

Number of layers	Displacement (inches)	Number of layers	Displacement (inches)
5	-8.55	9	-10.67
6	-11.15	10	-11.26
7	-11.14	11	-12.44
8	-10.71	12	-12.40

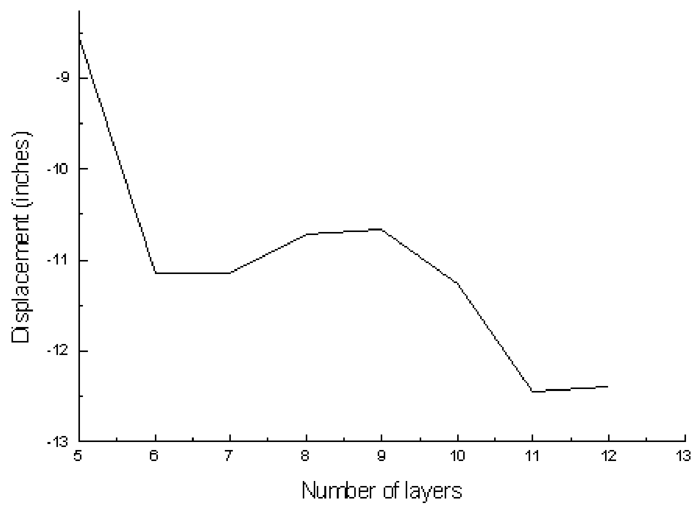


Figure 4.3.28 Correlation between final displacement and number of layers (bolt length = 5 ft, pretension = 6.75klbs)

Table 4.3.14 Final displacement of mid-span versus number of layers (bolt length = 5 ft, pretension = 11.25 klbs)

Number of layers	Displacement (inches)	Number of layers	Displacement (inches)
5	-8.57	9	-10.47
6	-11.05	10	-11.30
7	-11.06	11	-12.40
8	-10.67	12	-12.36

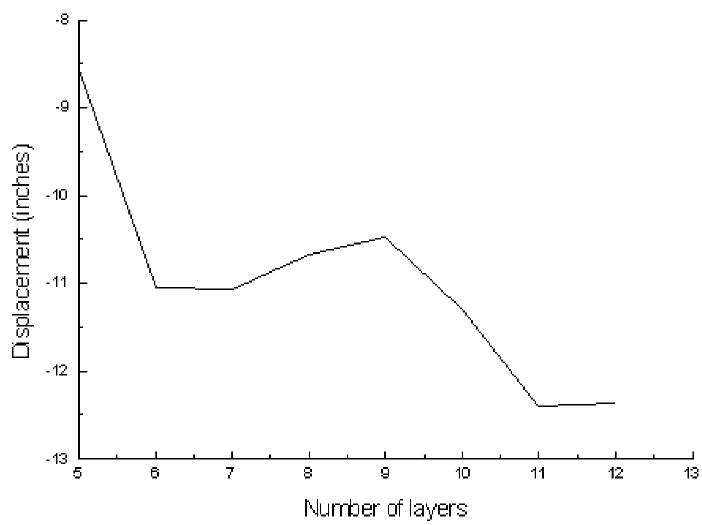


Figure 4.3.29 Correlation between final displacement and number of layers (bolt length = 5 ft, pretension = 11.25 klbs)

Table 4.3.15 Final displacement of mid-span versus number of layers (bolt length = 5 ft, pretension = 44.95 klbs)

Number of layers	Displacement (inches)	Number of layers	Displacement (inches)
5	-8.43	9	-10.43
6	-10.84	10	-11.22
7	-10.83	11	-12.44
8	-10.51	12	-12.32

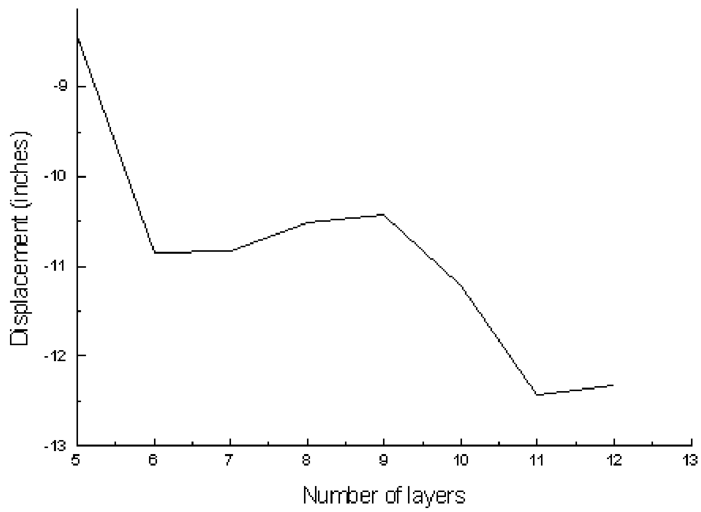


Figure 4.3.30 Correlation between final displacement and number of layers (bolt length = 5 ft, pretension = 44.95 klbs)

sudden movement and the amount is relatively small, alleviating the extent of artificial fracturing around the anchor.

Figures 4.3.19 through 4.3.26 show the impact of pretension on the immediate roof stability. When the number of layers is constant, the correlation between bolt pretension and the final roof displacement is very weak for all eight instances. Among them, the most significant is the case where the number of layers equals to 5. Linear regression is performed for this case.

$$Y = 0.00392X - 8.60 \qquad \text{Equation 4.3.9}$$

where

Y = Mid-span final displacement;

X = Bolt pretension on installation.

There is a tendency that, as the pretension increases, the magnitude of final displacement decreases. Based on Equation 4.3.9, this tendency, however, is insignificant, especially when compared to the interception, which is –8.60.

When studying optimum beam building effect in 1997, Stankus and Guo monitored the roof deflection under different bolting practices. Figure 4.3.31 shows the relationship of pretension applied to bolt at installation and the magnitude of roof sag with bolt length equal to 11, 8, and 5 feet, respectively. It is apparent that the difference of roof sag under different bolt lengths is insignificant after installed load exceeds 10,000 lbs. Also, the increasing installed load does not reduce roof sag in all cases.

However, it is well accepted in bolting practice that increasing pretension helps beam building effect, which in turn enhances bolting efficiency. But the result of this research does not support this claim due perhaps to the following reasons:

1. The beam created by bolting has a thickness of 5 feet, which is proved, by the experiment, to be strong enough to sustain the dead strata weight in Zone 1 and

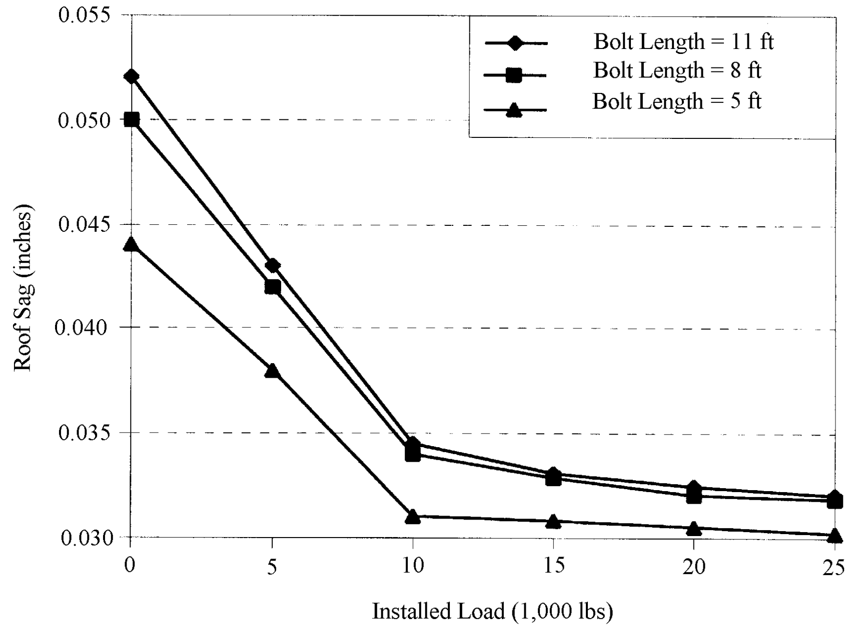


Figure 4.3.31 Relationship of installed load and roof sag with bolt length equal to 11, 8, and 5 feet (After Stankus, 1997)

Zone 2 in Figure 4.2.1 even though no pretension is applied. The binding effect created by pretension only makes the bolted beam as strong as a solid beam of the same thickness and of the same material.

2. When modeling bedding planes, it is assumed that there is no space between the sides of the interface. Furthermore, it is assumed that bolts are installed immediately after the opening is excavated. Thus, once the laminated strata start sagging, especially with the sudden movement of the immediate roof described previously, the axial force along a bolt builds up quickly and the manual pretension becomes redundant.

Figures 4.3.27 through 4.3.30 show the relationship of the number of layers and the mid-span final displacement with pretension equal to 0.0, 6.75, 11.25, and 44.95 klbs. It is evident that the final displacement increases as the number of layers within bolting range. To further investigate the relationship, 32 sets of data are used to perform linear regression and the result is shown in Figure 4.3.32.

$$Y = -0.4065X - 7.5415 \qquad \text{Equation 4.3.10}$$

where

Y = Mid-span final displacement;

X = Number of layer within bolting range.

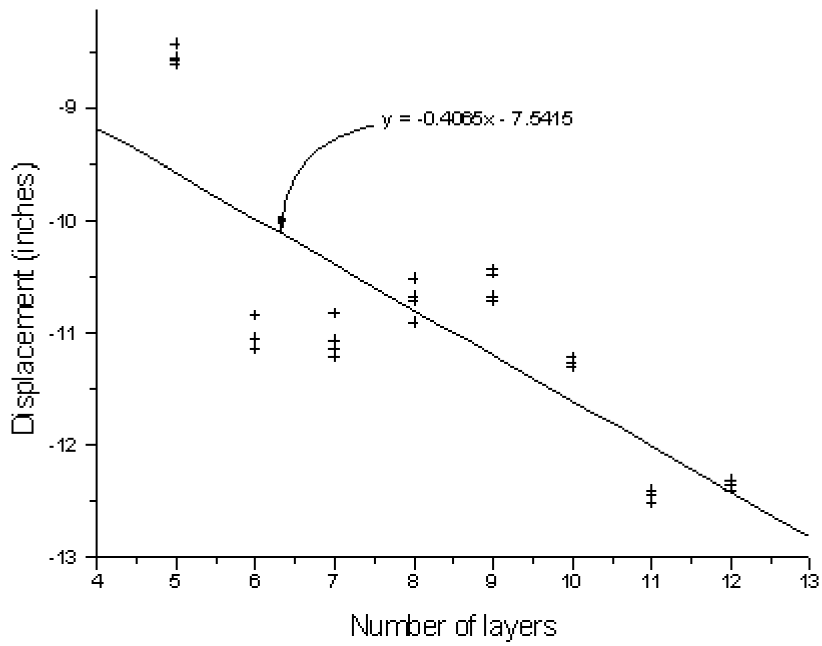


Figure 4.3.32 Linear regression of mid-span displacement versus number of layers

There are still more data sets available, but it is conceivable that adding more data sets into the linear regression model neither alters the result nor improves the accuracy significantly because the mid-span final displacement is almost independent of the pretension applied. On the other hand, the linear regression model represents only one special case; the relationship derived cannot be applied to situations with different rock properties or bolting parameters.

However, the relationship is very indicative. It is certain that the number of layers within bolting range or bedding plane spacing affects the movement of immediate roof significantly.

4. MINIMUM SOLID BEAM THICKNESS

From the experiment conducted so far, it is clear that the bolted beam is more than needed to support the dead weight of the strata within the pressure arch. In other words, bolt length is overdesigned. The optimum bolt length should be such that the bolted beam is just able to carry the dead weight of the strata in Zone 1 and Zone 2. However, due to the interaction caused by mining activity, a safety factor must be considered in determining the optimum bolt length. Theoretically, a bolted beam is as strong as a solid beam of the same material. Thus, finding the optimum bolt length is equivalent to finding the minimum solid beam thickness.

Given the uniformly distributed load acting upon the beam, displacement along the lower boundary of a bolted beam can be computed as described in Chapter 2. Due to pressure arching effect after the opening is made, the loading on a bolted beam is no longer uniformly distributed and very difficult to estimate. The method described in Chapter 2 is not applicable in such situations. Finding the thickness of a bolted beam cannot be achieved directly using the equations that have been established.

4.1 EXPERIMENTATION

The same FLAC model is still valid for determining minimum solid beam thickness. But two modifications must be made. One is that the bedding spacing is set to be the minimum solid beam thickness. The other is that no bolts are needed to be installed.

The parameters that affect the strength of the solid beam primarily include:

- Young’s modulus;
- Poisson’s ratio;

- Overburden thickness;
- Cohesion;
- Friction angle; and
- Tensile strength.

It is recognized that the roof span plays a very important role in the strength of the beam. However, in the United States, most of the entries in coal mines are about 18 – 22 feet wide. In this research, a constant roof span of 20 feet is used.

In this experiment, six independent variables are involved. Completely randomized design technique is employed to design the experiment. Each independent variable is randomly sampled 50 times, which are evenly distributed within its range, to form 50 groups of independent variables. For every group, the minimum solid beam thickness is computed. The results are shown in Table 4.4.1.

4.2 LINEAR MODEL

Based on results obtained, multiple linear regression is performed in an attempt to build a model that can be used to estimate the minimum solid beam thickness. A program called SAS is used for this analysis. Table 4.4.2 lists SAS code, and analysis results are presented in Appendix A. The minimum solid beam thickness can be predicted by the following expression:

$$\begin{aligned} Y = -1.26\times10^{-6} X_1 + 0.7721X_2 + 0.0016X_3 - 0.0001X_4 \\ - 0.0023X_5 - 0.0402X_6 + 5.0537 \end{aligned} \qquad \text{Equation 4.4.1}$$

where

X_1 = Young's modulus;

X_2 = Poisson's ratio;

X_3 = Overburden thickness;

X_4 = Cohesion;

Table 4.4.1 Experiment results

	E(psi)	v	OB(ft)	Coh(psi)	Ten(psi)	Fric()	Thick(ft)
1	1160320	0.25	830	2321	145	30	3.412
2	527946	0.35	627	580	145	37	4.298
3	746956	0.24	312	4235	87	40	1.968
4	864438	0.47	482	3655	1204	29	0.033
5	855736	0.20	955	4003	841	24	1.083
6	490235	0.14	1125	5338	696	30	3.510
7	349546	0.22	1073	2088	957	44	1.476
8	1073296	0.34	988	3829	189	21	3.740
9	651230	0.20	1073	1218	479	31	4.888
10	770162	0.16	705	1566	305	37	2.001
11	442372	0.12	1132	2727	638	30	2.526
12	1102304	0.31	833	2437	885	47	0.098
13	1116808	0.44	915	812	29	37	4.462
14	181300	0.20	312	1682	638	30	1.706
15	600466	0.34	978	3423	305	20	4.528
16	609168	0.43	932	3191	334	50	2.690
17	778865	0.19	1184	5686	580	60	2.723
18	409013	0.22	997	2379	131	28	4.790
19	433670	0.43	669	5280	218	38	2.592
20	256721	0.29	971	3655	218	32	4.068
21	335042	0.28	358	1740	131	45	3.018
22	726650	0.34	636	2030	116	43	2.690
23	1087800	0.16	761	1102	812	43	1.968
24	955814	0.43	417	696	638	31	2.592
25	976119	0.21	400	1856	276	33	2.493
26	738254	0.16	1033	3655	595	27	3.445
27	282828	0.35	978	5106	856	43	2.526
28	800621	0.18	1158	3945	203	29	3.642
29	147941	0.25	640	4815	203	21	4.331
30	343745	0.16	725	5686	754	39	2.986
31	319088	0.40	1037	4293	203	43	3.412
32	842682	0.44	522	754	102	59	3.084
33	1084899	0.43	1109	2785	44	51	2.559
34	826728	0.21	1010	2205	305	22	7.611
35	234965	0.28	935	4525	986	20	2.986
36	326340	0.35	1076	3307	493	34	3.707
37	317638	0.32	850	2146	102	28	4.626
38	419166	0.27	1119	1624	56	44	3.806
39	688940	0.12	266	4641	324	32	1.903
40	938409	0.37	827	2263	55	26	3.839
41	713597	0.28	837	4293	96	56	2.428
42	232064	0.28	656	4351	266	49	3.084
43	1116808	0.24	951	3365	342	45	2.395
44	1063143	0.29	407	4409	137	56	1.378
45	160994	0.10	410	3887	73	44	3.642
46	570007	0.15	482	4815	46	29	2.789
47	754208	0.16	1040	1624	279	30	3.806
48	291530	0.32	1175	4235	117	26	4.692
49	1019631	0.26	633	1972	305	23	3.150
50	620771	0.48	272	3597	610	21	3.314
51	590313	0.32	764	2495	34	33	4.462

Table 4.4.2 SAS code for linear regression and data analysis

```
OPTION NODATE LS = 72;
TITLE 'MINIMUM THICKNESS OF ROOF BEAM';
DATA BOLT;
INPUT H X1 X2 X3 X4 X5 X6 Y;
CARDS;
1       1160320         0.25    830     2321    145     30      3.412
2       527946          0.35    627     580     145     37      4.298
3       746956          0.24    312     4235    87      40      1.968
4       864438          0.47    482     3655    1204    29      0.033
5       855736          0.20    955     4003    841     24      1.083
6       490235          0.14    1125    5338    696     30      3.510
7       349546          0.22    1073    2088    957     44      1.476
8       1073296         0.34    988     3829    189     21      3.740
9       651230          0.20    1073    1218    479     31      4.888
10      770162          0.16    705     1566    305     37      2.001
11      442372          0.12    1132    2727    638     30      2.526
12      1102304         0.31    833     2437    885     47      0.098
13      1116808         0.44    915     812     29      37      4.462
14      181300          0.20    312     1682    638     30      1.706
15      600466          0.34    978     3423    305     20      4.528
16      609168          0.43    932     3191    334     50      2.690
17      778865          0.19    1184    5686    580     60      2.723
18      409013          0.22    997     2379    131     28      4.790
19      433670          0.43    669     5280    218     38      2.592
20      256721          0.29    971     3655    218     32      4.068
21      335042          0.28    358     1740    131     45      3.018
22      726650          0.34    636     2030    116     43      2.690
23      1087800         0.16    761     1102    812     43      1.968
24      955814          0.43    417     696     638     31      2.592
25      976119          0.21    400     1856    276     33      2.493
26      738254          0.16    1033    3655    595     27      3.445
27      282828          0.35    978     5106    856     43      2.526
28      800621          0.18    1158    3945    203     29      3.642
29      147941          0.25    640     4815    203     21      4.331
30      343745          0.16    725     5686    754     39      2.986
31      319088          0.40    1037    4293    203     43      3.412
32      842682          0.44    522     754     102     59      3.084
33      1084899         0.43    1109    2785    44      51      2.559
34      234965          0.28    935     4525    986     20      2.986
35      326340          0.35    1076    3307    493     34      3.707
36      317638          0.32    850     2146    102     28      4.626
37      419166          0.27    1119    1624    56      44      3.806
38      688940          0.12    266     4641    324     32      1.903
39      938409          0.37    827     2263    55      26      3.839
40      713597          0.28    837     4293    96      56      2.428
41      232064          0.28    656     4351    266     49      3.084
42      1116808         0.24    951     3365    342     45      2.395
43      1063143         0.29    407     4409    137     56      1.378
44      160994          0.10    410     3887    73      44      3.642
45      570007          0.15    482     4815    46      29      2.789
46      754208          0.16    1040    1624    279     30      3.806
47      291530          0.32    1175    4235    117     26      4.692
48      1019631         0.26    633     1972    305     23      3.150
49      620771          0.48    272     3597    610     21      3.314
50      590313          0.32    764     2495    34      33      4.462
;
PROC REG;
        MODEL Y = X1 X2 X3 X4 X5 X6;
        MODEL Y = X1 X2 X3 X4 X5 X6 / SELECTION = RSQUARE ADJRSQ CP;
        MODEL Y = X1 X2 X3 X4 X5 X6 / SELECTION = FORWARD SLENTRY = 0.25;
        MODEL Y = X1 X2 X3 X4 X5 X6 / SELECTION = BACKWARD SLSTAY = 0.25;
        MODEL Y = X1 X2 X3 X4 X5 X6 / SELECTION = STEPWISE SLENTRY = 0.25 SLSTAY = 0.25;
        MODEL Y = X1 X2 X3 X4 X5 X6 / P;
```

X_5 = Tensile strength; and

X_6 = Friction angle.

The coefficient of determination of this linear model, R^2, is 0.7182, which indicates that about 72 percent of the total variation in the minimum solid beam thickness can be accounted by knowing the six independent variables.

Forward selection, backward elimination, and stepwise regression methods are used to study the sensitivity of the independent variables. As shown in Appendix A, all three methods indicate that the six independent variables are required for a good prediction of the minimum solid beam thickness.

4.3 POLYNOMIAL MODEL

Trying to improve the prediction quality of the linear model, a polynomial model is built as below:

$$Y = 30.32X_1^{-0.02} + 1.13X_2^{0.4} - 41.26X_3^{-0.5} + 6.87X_4^{-0.05} + 6.43X_5^{-0.27} + 16.00X_6^{-0.2} - 33.29 \qquad \text{Equation 4.4.2}$$

where variables are defined previously.

However, this polynomial model does not give a better prediction than the linear model. Instead, the coefficient of determination is only 68%.

4.4 DIMENSIONLESS MODEL

A dimensionless analysis technique is used to analyze the data and a model is subsequently built as below:

$$Y = \frac{X_3^{\ 0.55}}{X_1^{\ 0.04} X_4^{\ 0.05} X_5^{\ 0.12} X_6^{\ 0.26}} \qquad \text{Equation 4.4.3}$$

where variables are defined previously.

In this model, Poisson's ratio becomes redundant in predicting the minimum solid beam thickness. The coefficient of determination of the model is around 70%.

4.5 OPTIMIZED MODEL

It is observed that the three different models can individually predict the minimum solid beam thickness with a coefficient of determination equal to 70%. They predict in three different ways, leading to three distinctive results even when the input parameters are same. Assigning a certain weight to each predicted result and then combining them together may improve the accuracy. Treating the three predicted minimum solid beam thicknesses as independent variables, an optimized model can be built as below:

$$Y = 0.3452\mathrm{Y}_1 + 0.3941\mathrm{Y}_2 + 0.2679\mathrm{Y}_3 \qquad \text{Equation 4.4.4}$$

where

Y_1 = Result predicted by the linear model;

Y_2 = Result predicted by the polynomial model; and

Y_3 = Result predicted by the dimensionless model.

Equation 4.4.4 can predict the minimum solid beam thickness with a coefficient of determination equal to78 %, which is significantly improved compared to the coefficient of determination of the three models individually.

4.6 DISCUSSIONS

The prediction qualities of the three models are relatively low. Even the optimized model can predict with only 78% confidence. There are at least two things that can be done to improve the accuracy: adding more data into the model or considering a better experimental design strategy. However, whenever geological parameters are involved, the variation of statistical models is always high.

The models developed above provide insightful information for determining bolt length when designing a bolting support system. However, due to the diversity of geological conditions from mine to mine, any results from numerical formulae are not absolute and only useful when used as reference values.

5. MODEL VALIDATION

This statistical model not only can be used to predict the minimum solid beam thickness but also indicates the impact of each parameter involved in the model on the determination of the MSBT. Figures 4.3.33 to 4.3.38 show the sensitivity of individual parameters that contribute to the minimum solid beam thickness. When comparing these results it should be noticed that overburden thickness is the most significant factor.

To validate this minimum solid beam thickness model, four case studies were conducted.

5.1 CASE 1: THE MIIKE COLLIERY

The Miike Colliery is located in Kyushu, Japan. It is a large undersea coal mine. The coal is mined by highly mechanized retreat longwall faces. The seam being mined is at a depth of about 1,000 ft below sea and its thickness varies between 13 to 14 ft. The seam is overlain and underlain by sandstone. The geomechanical properties of roof rocks are shown in Table 4.5.1 (Matsui and Furukawa, 1993). The gateroads are driven by a roadheader MRH S-65, with geometry of 16 ft wide and 9.2 ft high. Bolts are resin-grouted with length of 5.8 or 6.9 ft and density of 5 bolts per row. This bolt support system proved to be very successful in maintaining roof stability.

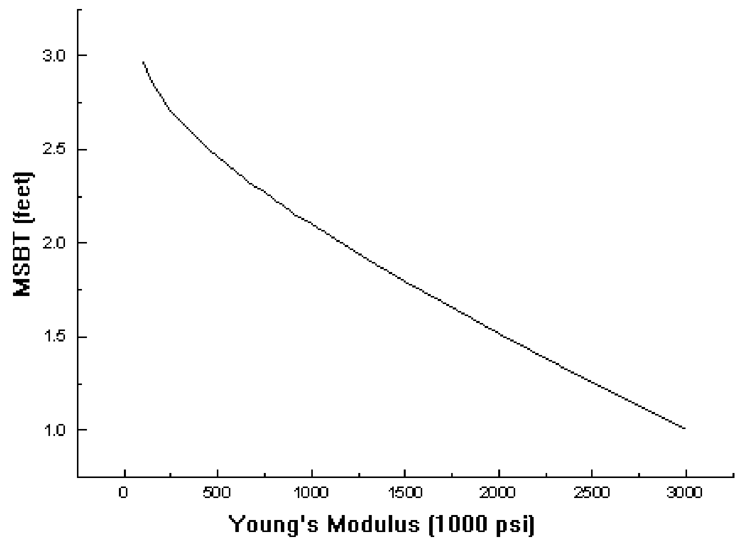


Figure 4.3.33 Relationship of MSBT and Young's modulus

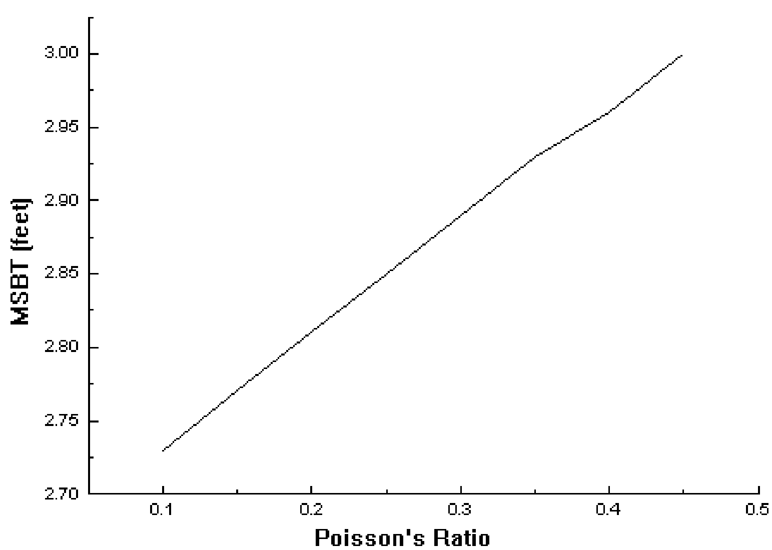


Figure 4.3.34 Relationship of MSBT and Poisson's ratio

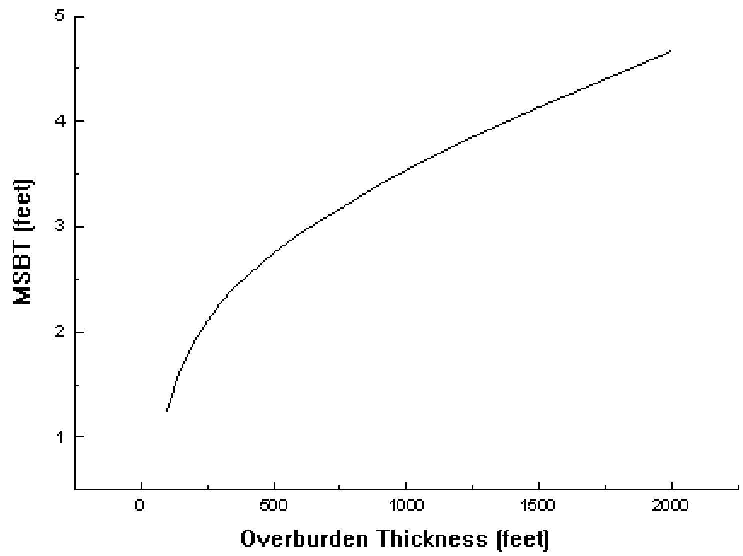


Figure 4.3.35 Relationship of MSBT and overburden thickness

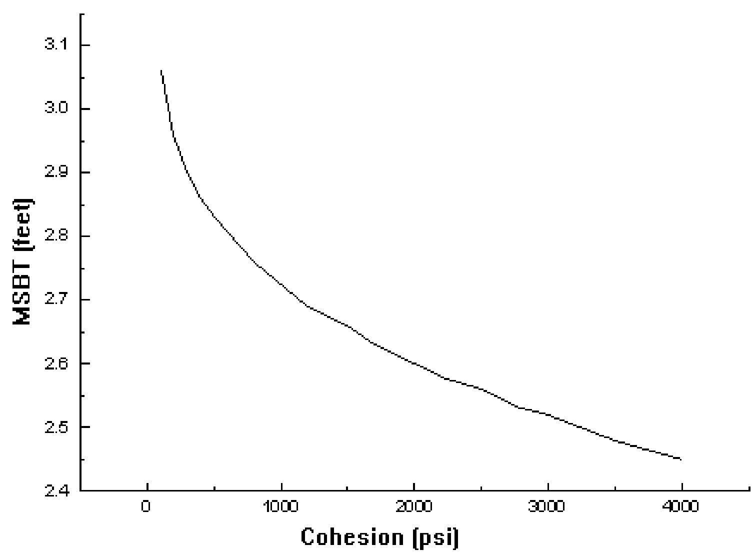


Figure 4.3.36 Relationship of MSBT and cohesion

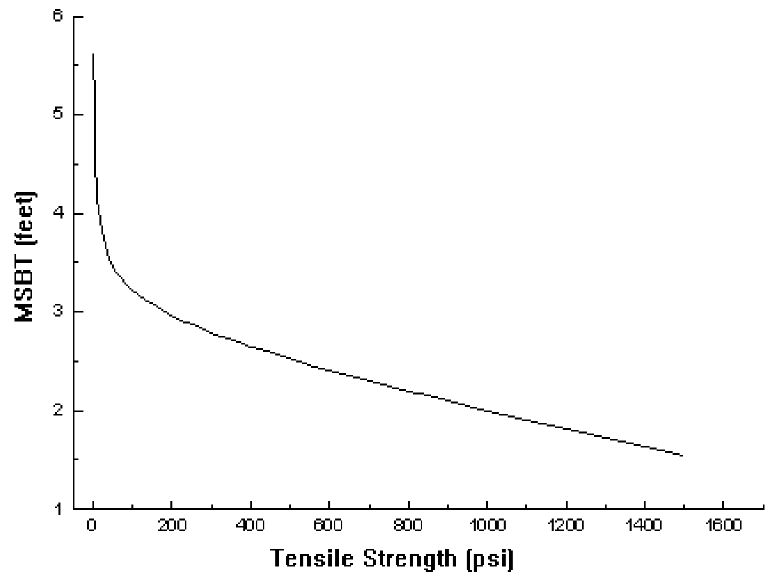


Figure 4.3.37 Relationship of MSBT and overburden thickness

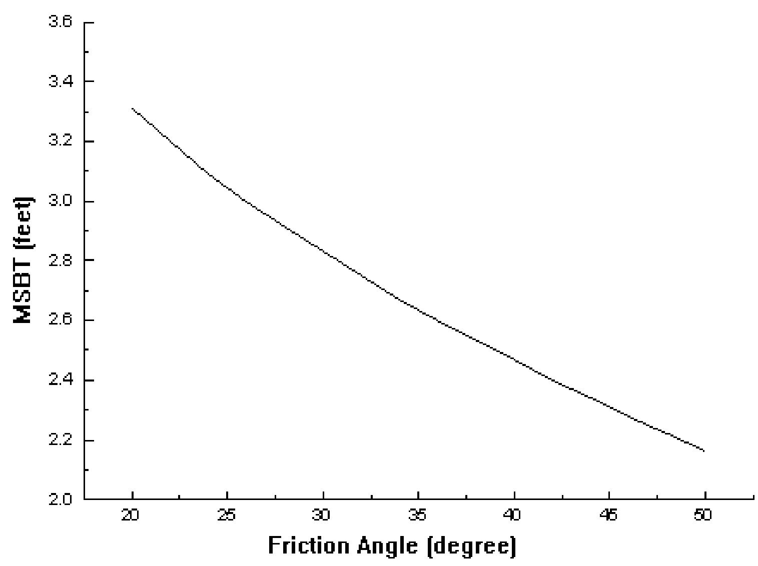


Figure 4.3.38 Relationship of MSBT and friction angle

Table 4.5.1 Predicted MSBT and safety factors for the Miike Colliery

	Condition	
	dry	**wet**
Rock type of roof	sandstone	
Young's Modulus (psi)	1,189,000	522,000
Poisson's ratio	0.25	0.25
Overburden thickness (ft)	1000	1000
Cohesion (psi)	3100	1295
Tensile strength (psi)	785	275
Friction angle (degree)	30	30
Predicted MSBT (ft)	2.63	3.94
Actual bolt length (ft)	5.8/6.9	
Safety factor	2.21/2.62	1.47/1.75

Predicted minimum solid beam thickness for dry and wet conditions are 2.63 and 3.94 ft, respectively. Compared to the length of the bolts actually used, safety factors for both conditions are 2.21/2.62 and 1.47/1.75. Based on the safety factors, it is apparent that the selected bolt length for wet roof condition is appropriate. However, for dry roof condition, the bolt length is overdesigned.

5.2 CASE 2: THE FOIDEL CREEK MINE

The Foidel Creek Mine is located 20 miles southwest of Steamboat Springs, Colorado, on the southeastern extension of the Washakie-Sand Wash basin. The seam being mined is the Wadge seam that averages 1,100 ft beneath the surface. The immediate roof is 1 to 3 ft thick and consists of bone coal, carbonaceous mudstones, channel sandstones, and thin coals. The roof also deteriorates with exposure to air, separates along thin bedding planes, and fractures into small blocks. The main roof is about 45 ft thick and consists primarily of mudstone and sandstone. Physical properties of the roof rock are shown Table 4.5.2.

The gateroad design uses the three-entry system. The tailgate contains 80ft square pillars on 100 ft centers and the headgate uses the yield-rigid pillar system. Both head- and tail-gates are 20 ft wide and 6 ft high. Five-foot point-anchored bolts are used as primary support with standard 5-ft square spacing. Roofs are successfully controlled by this bolting pattern.

Based on the physical properties of the surrounding rock, the minimum solid beam thickness predicted by the optimized model is 2.07 ft and the corresponding safety factor is 2.42. In this case, Young's modulus is relatively high, substantially reducing the predicted minimum solid beam thickness. Considering that the main roof is only 1-3 ft above the opening, beam building may not be the dominant bolting mechanism. Based on suspension theory, the minimum bolt length required in this case is 4 ft. In conjunction with the predicted minimum solid beam thickness, and taking into account the fact that the

Table 4.5.2 Roof rock physical properties of the Foidel Creek Mine (Haramy and Fejes, 1991)

Rock Type	Property	Maximum value	Minimum value	Mean value
Interbedded siltstone, black shale	E (psi)	4,960,000	1,740,000	2,600,000
	v	0.46	0.14	0.33
	Tensile strength (psi)			766
	Cohesion (psi)	6,108	3,220	4,747
	Friction angle	32		
Interbedded sandstone, siltstone, black shale	E (psi)	6,950,000	1,680,000	2,680,000
	v	0.45	0.19	0.34
	Tensile strength (psi)			557
	Cohesion (psi)	6,605	1,852	4,342
	Friction angle	32		
Sandstone	E (psi)	6,890,000	1,930,000	3,210,000
	v	0.48	0.12	0.31
	Tensile strength (psi)			718
	Cohesion (psi)	8,544	2,199	4,142
	Friction angle	32		
Pyritic black shale	E (psi)	3,070,000	2,420,000	2,760,000
	v	0.47	0.30	0.36
	Tensile strength (psi)			579
	Cohesion (psi)	4,924	3,799	4,357
	Friction angle	32		

immediate roof is prone to weathering, a recommended optimum bolt length of 4.5 ft should be appropriate.

5.3 CASE 3: CASE STUDY MINE #3

The immediate roof in this mine consists of a fine grained sandstone with some shale with average overburden depth of 600 feet. The entry is 16 feet wide and 9 feet high. Primary support consisted of a 6-foot torque/tension system fully encapsulated with a fast and slow set resin. The bolt was installed on a 4-foot by 4-foot pattern with four per row. Average installed tension was 12,000 lbs. No roof control problems were incurred with this system.

The minimum solid beam thickness approach was applied using the following parameters:

- Young's modulus = 800,000 psi;
- Poisson's ratio = 0.30;
- Overburden thickness = 600 feet;
- Cohesion = 2000 psi;
- Tensile strength = 400 psi; and
- Friction angle = 32°.

The predicted MSBT is 2.80 feet. The recommended optimal bolt length is 4.5 feet and the safety factor is 1.6. Taking into account the fact that this approach was based on 20 foot-wide entry, the recommended bolt length should be appropriate.

Stankus et al., using the Optimum Beaming Effect approach, suggested that no installed tension was needed and bolt length could be reduced from 6 feet to 5 feet. A headed #7 × 5-foot J-Bar, fully encapsulated, and non-tensioned bolt was implemented as primary support. No control problems had been reported and costs were reduced.

5.4 CASE 4: THE KITT NO. 1 MINE

Kitt No.1 mine was located in Philippi, West Virginia. It operated at a depth of 430 feet in the Middle (lower) Kittaning coal seam. The seam averaged about five feet in thickness. Mining was done with a continuous miner that cut a rectangular opening of 18 feet wide. There appeared to be little or no water in the seam although gas was evident in some areas. Roof strata consisted of a dark, hard shale, well laminated with individual layers rarely being over four inches or less than one inch in thickness. Few joints were present in the roof strata. Floor strata consisted of a marl type mudstone. Roof support was with five-foot resin bolts on four-foot centers.

The following mine site parameters are used to predicted the minimum solid beam thickness:

- Young's modulus = 347,000 psi;
- Poisson's ratio = 0.25;
- Overburden thickness = 430 feet;
- Cohesion = 2,010 psi;
- Tensile strength = 435 psi; and
- Friction angle = 36°.

The predicted minimum solid beam thickness is 2.85 feet. With a safety factor of 1.7, a five-foot bolt seems adequate.

However, a number of roof failures happened. Many roof bolts were still in place at the edge of the falls and appeared to be holding well. Analysis indicated that the falls were structurally related. It was very likely that there were significant tectonic stresses in those particular areas. Interaction from other workings above the current mining activities might have concentrated the overburden loads. Also it was believed that the changing lithology of these areas contributed to the falls. Longer mechanical bolts of eight to nine feet in length would not prevent the roof falls form occurring but could only increase the duration of the roof failure.

This case study shows that the minimum solid beam thickness approach may not work under some unique circumstances, although it is a valuable tool in optimizing bolt length. The predicted optimum bolt length can be practical only after the local in-situ conditions are thoroughly examined and taken into account.

CHAPTER 5
TROUBLE-SHOOTING OF BOLTING SYSTEMS

1. INTRODUCTION

With the methods developed previously, the design of optimum bolt support systems is possible. But, due to the ever-changing geological conditions, mines constantly experience bolted roof failures even though bolt support systems are designed using the most conservative design paradigm. This kind of roof failures is caused by adverse geological conditions, poor bolt installation practices, or malfunctioning supports. Quickly and correctly diagnosing the causes will enable the engineer to take appropriate remedial measures to prevent the same problem from happening again, eliminating factors leading to future ground control problems. Furthermore, the task becomes more complicated with the development and usage of new bolt types. A survey shows that at least seven different types of bolts are currently used to support longwall tailgates, let alone other underground structures. From the viewpoint of ground control engineers, it is a good thing to have a handy, vast bolt selection from which they can choose a certain type of bolt to meet their specific support requirements and unique geological settings. However, when problems with bolt support systems are encountered, determining the actual cause(s) becomes more difficult and requires intense expertise.

2. BOLTING FAILURE MODES

For point-anchored bolts, including mechanical, point-resin-anchored, and combination bolts, a certain amount of tension is always applied during installation to help create suspension, beam building, and keying effects. However, point anchors, especially expansion shell anchors, produce a high stress concentration against the surrounding rock, resulting in rock fracture and a tendency for the anchor to slip or creep with time. The magnitude of concentrated stresses is proportionally related to the tension applied to the bolts. Generally, the additional fracturing induced by pretension is insignificant compared to the additional reinforcement provided by the bolting system. But it has been observed that this type of fracturing may pass through the ends of the deepest bolts, and the fracturing is more likely to propagate as bolt spacing is reduced. Sometimes the bolting system works very well in the range of the supported area, but the entire section of the supported roof caves in as a unit. The fracturing, combined with high abutment pressure due to mining, could be a potential cause of roof failure (Jorstad, 1967).

By contrast, full-length-grouted bolts eliminate this problem, since no tension is required for them to function. Or, even if tension is applied, the stress induced is evenly distributed along the whole bolt length. Without too many disturbances to the roof, full-length-grouted bolts are more effective than point-anchored bolts in weak, highly laminated shales and other soft rocks. Even though they are more expensive, untensioned full-length-grouted bolts are gaining wide acceptance in many U.S. coal mines. Tensioned full-length-grouted bolts not only can establish good beam building effect but also produce suspension and keying effects.

The reinforcement ability of both point-anchored and full-length-grouted bolts is primarily accomplished by the frictional forces created by the physical interlocking along the anchor/rock interface or along the grout/rock contact. Loss of fractional forces is the most common cause of bolt failure and is referred to as anchorage failure. It could occur because of either the damage to the surrounding rock or the damage to the anchor itself. For tensioned bolts, anchorage failure results in loss of bolt tension and consequently loss of compressive force between the bolt ends. Suspension, beam building, and keying effects could be reduced significantly or sometimes totally lost, resulting in roof fall. Anchorage failure in untensioned full-length-grouted bolts results in diminished beam building effects, reduced load-carrying capacity, and decreased shear resistance to horizontal stresses.

Regularly conducting pull tests and torque tests can detect potential anchorage failure in time. The thorough way, however, is to find out the exact causes of the loss of anchorage capacity, and then adopting corresponding measures to prevent anchorage failure from occurring.

The main factors attributed to anchorage failure include:

- Weak rocks at the point-anchor area;
- Improper hole size;
- Adverse installation environment such as run-off water in the hole;
- Wear and damage of hardware components;
- Improper installation procedure; and
- Seismic force such as blast shock.

Also, it is very often that bolting failure occurs merely due to loading larger than the anchorage capacity of the bolts. The excessive loading may result from stress redistribution caused by mining interaction or extraordinary superincumbent pressure. When the excessive loading is neglected or underestimated at the time of roof support system design, the bolting system cannot withstand the extra loading. This kind of roof bolting failure results mainly from improper system design. The only remedy is to reassess the geological conditions and accordingly redesign the bolting system as described in Chapter 3. As a precaution, it is recommended that bolts be installed as soon as possible after the opening is excavated, since sagging and bed separation of roof strata increase with time, although some researchers pointed out that no relationship exists between time delay and stress distribution (Radcliffe and Stateham, 1978). On the other hand, for permanent support with bolts, weathering effects, which eventually lessen bolt yield strength and damage bolt components, should be taken into account. In such cases, coated rebar and/or full-length-grouted bolts must be considered.

3. BOLT CLASSIFICATION

In order to simplify the trouble-shooting procedure, rock bolts are classified into the following five categories in such a way that it is easy to emphasize problems unique to a particular support type (Mazzoni et al., 1996):

- Mechanical bolts: a mechanical bolt is defined as a smooth, headed, threaded bolt that uses a mechanical expansion anchor without grout, as shown in Figure 5.3.1. Expansion anchors are of two basic types, namely bail and standard. Bail-type anchors utilize a strap or wire to hold the shell leaves in place and initiate anchorage. The leaves of the standard-type shell are held in place with a support nut;
- Fully grouted bolts: a fully grouted bolt is defined as a headed deformed bar (steel bar having special deformations to provide interlocking between the steel and the grout) that uses a chemical grout for anchorage and is not pretensioned, as shown in Figure 5.3.2;
- Tension rebar bolts: a tensioned rebar bolt is defined as a threaded, deformed bar that uses a grout anchor and a tension nut, as shown in Figure 5.3.3. Tension nuts come in various forms such as square nuts in the shape of a bolt head that use either shear pins, aluminum plugs, or cast domes as the torque inhibitor;
- Combination/point anchored bolts: a combination/point anchored bolt is defined as a two-piece bolt that uses grout for anchorage and a tensioning coupler without an expansion anchor, as shown in Figure 5.3.4. The bolt has a length of threaded deformed bar coupled to a headed bolt. The tensioning coupler has a torque-inhibiting device which prevents thread take-up during grout mixing, then breaks free to permit the bolt to be tensioned. The torque inhibitor can be a shear pin or an aluminum plug;
- Mechanically anchored resin-assisted bolts: a mechanically anchored resin-assisted bolt is defined as a headed, threaded bolt that uses grout for primary anchorage, and utilizes a mechanical expansion anchor for tensioning and initiating anchorage, as shown in Figure 5.3.5. There are a number of variations of the bolt type, which all combines the excellent anchorage of grouted bolts with the quick installation time of mechanical bolts.

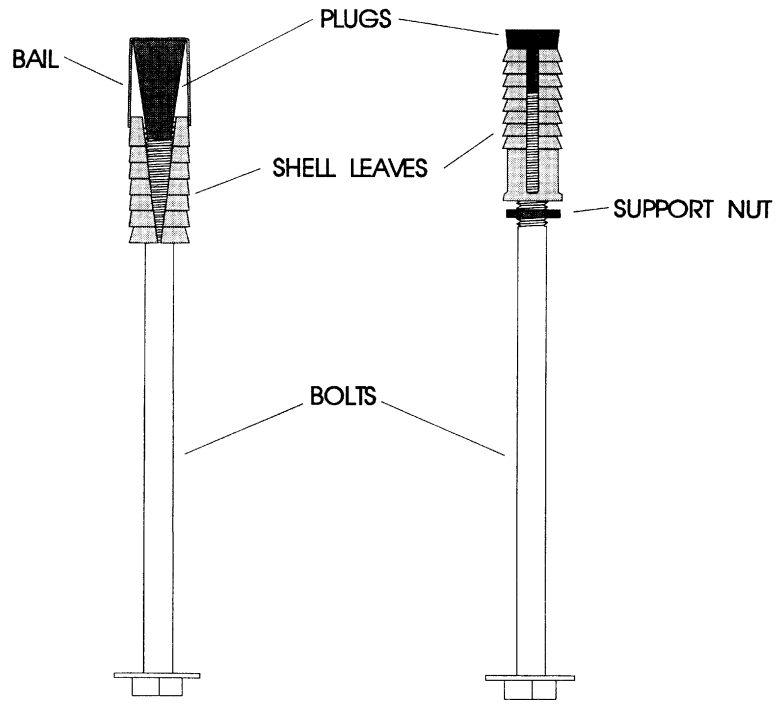


Figure 5.3.1 Mechanical bolts (Mazzoni et. al., 1996)

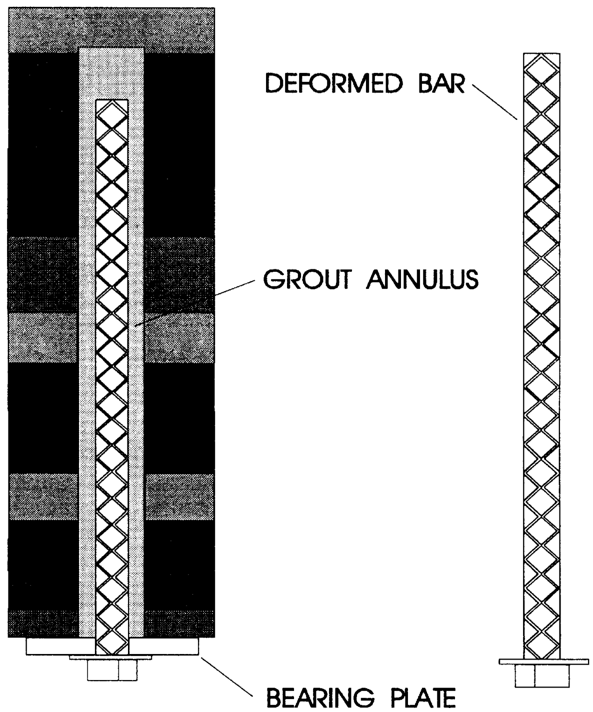


Figure 5.3.2 Fully grouted bolts (Mazzoni et. al., 1996)

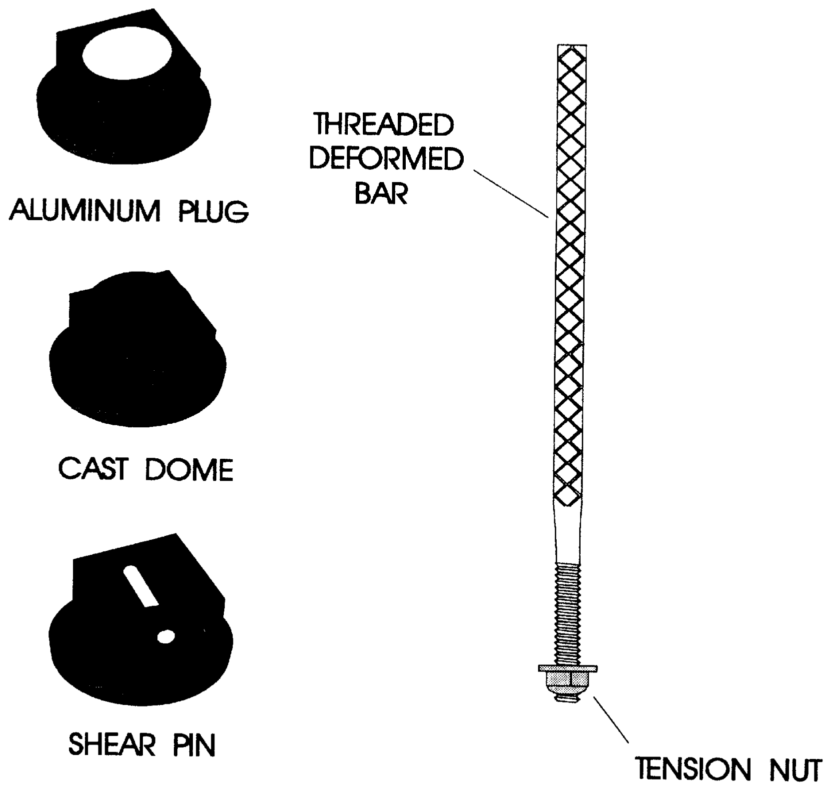


Figure 5.3.3 Tensioned rebar bolts (Mazzoni et. al., 1996)

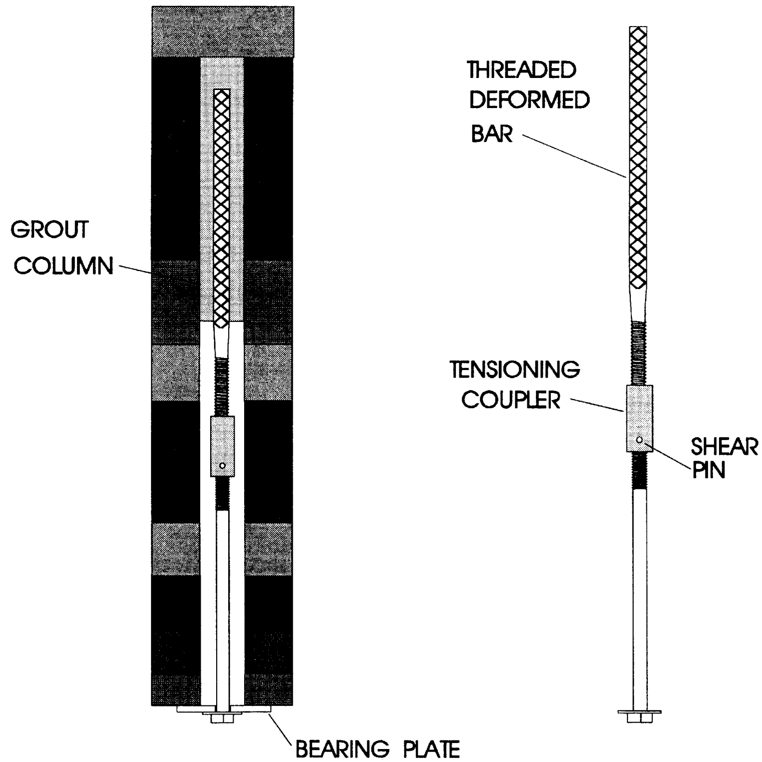


Figure 5.3.4 Combination/point anchored bolts (Mazzoni et. al., 1996)

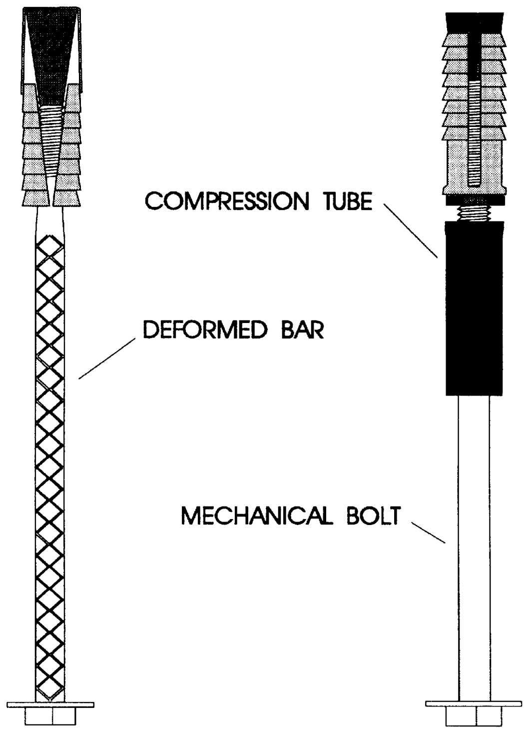


Figure 5.3.5 Mechanically anchored resin assisted bolts (Mazzoni et. al., 1996)

4. PREVIOUS WORK ON TROUBLE-SHOOTING

Since 1970, the Mine Safety and Health Administration, Roof Control Division (RCD) of the Pittsburgh Safety and Health Technology Center has been involved in the design and testing of roof supports to enhance the application and ensure the performance of various roof support system by advising manufacturers on the merits of new concepts, working with industry groups to develop effective roof support component specifications, and conducting hundreds of in-mine and over 1,000 laboratory investigations (Mazzoni, et al., 1996). Through this involvement, RCD accumulated an extensive amount of knowledge of the associated problems that could occur with these supports. Not until early 1980s was any effort made to piece that information together and systemize them for easy reference. At that time, mechanical bolts were used predominantly in the mining industry. A trouble-shooting chart, as shown in Figure 5.4.1, was derived from the amassed knowledge. Later a second trouble-shooting chart was made for fully grouted bolts, which were also widely used at that time. As new types of bolts were introduced, additional charts were generated for tensioned rebar bolts, combination/point anchored bolts, and mechanically anchored resin-assisted bolts. These charts were produced by following the same format and procedure as the original mechanical bolt trouble-shooting chart. In those charts, all problems that had occurred with a particular bolt type were listed and several possible reasons that would cause a problem to occur were identified. Furthermore, those reasons were categorized as most likely, secondary, and potential causes. Although these trouble-shooting charts are very valuable tools for analyzing bolt problems, they have limitations. They do not provide users with a logical procedure for trouble-shooting bolt problems, nor do they list methods for determining which probable causes actually result in the problem.

Realizing the shortcomings of those original charts, Mazzoni et al. (1992) developed a more comprehensive trouble-shooting guide, which contain a listing of problems for each bolt type, a diagnostic flow chart for each problem, a series of appendices outlining trouble-shooting methods, and the five original trouble-shooting charts for easy reference. This guide is more user-friendly. It also gives users a logical sequence for evaluating a

POSSIBLE CAUSE \ PROBLEM	EXCESSIVE TORQUE	EXCESSIVE BLEEDOFF	BOLT/PLUG THREADS STRIP	BOLT PULLS THROUGH PLATE	PLATE CRACKS	WASHER CRACKS	WASHER BADLY DEFORMED	PLATE FAILS AFTER INSTALLATION	BOLT BREAKS AFTER INSTALLATION	LOOSE PLATE	PLATE FAILS ON INSTALLATION	BOLT BREAKS ON INSTALLATION	SPRINGY BOLT	ERRATIC TORQUE-TENSION	HIGH TORQUE/LOW TENSION	TORQUE NOT ACHIEVED	ANCHOR WILL NOT SET	ANCHOR BREAKS ON INSTALLATION
DEFECTIVE BOLT									○			○						
DEFECTIVE THREADS	○		○						○			○	○	○	○		○	
DEFECTIVE PLATE				○	●			○			○			○				
DEFECTIVE ANCHOR		○	○										○	○		○	○	○
BAIL/PALNUT TOO STRONG	○											○	○	○	○		◒	◒
BAIL/PALNUT TOO WEAK																	○	○
WASHER TOO SOFT	○			○			○							◒	◒			
WASHER TOO HARD						●												
BOLT/ANCHOR THREADS JAM	○									○		○	◒	○	◒		○	
PLATE/WASHER MISMATCH	○			○		○	◒	○			○							
LIMITED THREAD ENGAGEMENT			●															
PLATE HOLE TOO LARGE				◒		○	○	○			○							
IRREGULAR ROOF SURFACE				○		○	○	○	○	◒	○	○						
POOR ROOF STRATA		●								◒						◒	○	
WET HOLE		●								○						○		
HOLE TOO SHORT										●			●	○	○	○		
HOLE DIAMETER TOO LARGE		○								○						●	●	○
HOLE DIAMETER TOO SMALL		○														○	○	●
ANCHOR PRE-EXPANDED																		◒
PLATE-BOLT GALLING	○													●	●			
EXCESSIVE THRUST	○													●	●	●		
SEVERE INSTALLATION ANGLE	◒			◒		◒	●	○	○		◒	◒		●	●			
ANCHORAGE EXCEEDED		○								○					○	○		
THREADS GALLED ON INSTALLATION	○		◒										◒	○	◒			
APPLIED TORQUE TOO HIGH	●	◒		○			○		◒		●	●				○		
APPLIED TORQUE TOO LOW		◒														◒		
SEVERE ROOF LOADING	●	○		●			◒	◒	●									
PLATE-BOLT MISMATCH		○		●				●			●							

● MOST LIKELY CAUSE ◒ SECONDARY CAUSE ○ POTENTIAL CAUSE

Figure 5.4.1 Trouble-shooting chart for mechanical bolts (Mazzoni et. al., 1996)

particular bolt problem, methods for determining the cause of the problem, and ways for correcting the problem. The flow charts, such as the one shown in Figure 5.4.2, provide a step-by-step approach for examining each possible cause of a problem in sequence, in the order that they are most likely to occur. This approach speeds up tremendously the process of identifying the actual cause of a particular problem. The most important feature of this trouble-shooting guide is the appendix. It lists not only the procedures that help users eventually determine whether the possible cause is indeed the problem source, but also provides corresponding suggestions for correcting the problems. Additionally, the Appendix offers techniques for further investigation if the suspected cause is not responsible. By identifying the bolt problem encountered, pinpointing the possible cause using the flow charts, and then performing the checks listed in the Appendix, users can systematically trouble-shoot a particular roof bolting problem.

It is apparent that trouble-shooting a roof bolting system requires knowledge and expertise acquired from the applications of bolting systems over many years. Most of the information is empirical and heuristic. It is very probable that as knowledge accumulates some currently well-established rules for trouble-shooting will one day prove to be incorrect or optimal, while some new and more complete information will be added. With the development and application of new types of more effective bolts or bolts with special purposes for unique geological conditions and support requirements, trouble-shooting the new bolts could be a formidable task. The trouble-shooting guide developed by RCD is quite comprehensive and useful for most of the problems encountered in reality; but its ability to accommodate new knowledge and to update current knowledge is limited. On the other hand, with the power of computer technology and the wide application of computers in mining operation, following the trouble-shooting guide is not very convenient. All of these aspects of trouble-shooting a particular bolt type lend itself to a new format involving knowledge-based expert system.

BOLT PULLS THROUGH PLATE

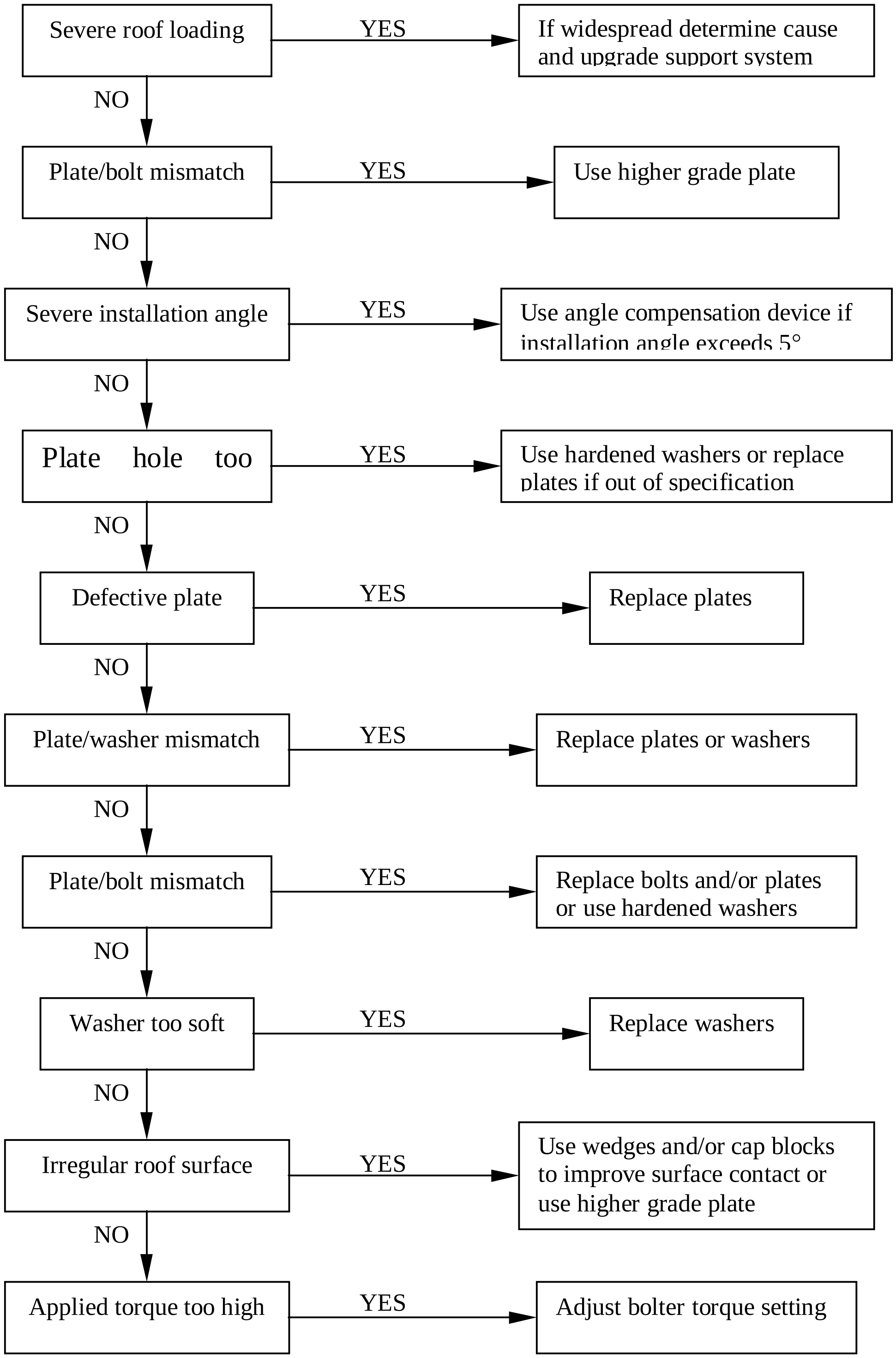


Figure 5.4.2 Flow chart for sequentially examining possible causes (Mazzoni et. al., 1996)

CHAPTER 6

KNOWLEDGE-BASED EXPERT SYSTEM FOR TROUBLE-SHOOTING BOLTING SYSTEMS

1. INTRODUCTION

In the past 20 years, knowledge-based expert system (KBES), a branch of fast-growing artificial intelligence, has found its applications in a wide range of domains. From medical diagnosis, trouble-shooting complex equipment, and financial analysis to decision-making in the mining industry such as mining method selection and processing plant control, KBES offers the opportunity to organize human expertise and experience into a form that computer can manipulate, thus improving both productivity and improving the quality of decisions. Through KBES, the knowledge or expertise in a specific domain is abstracted and stored in a way that an inexperienced user can utilize the computer's inferencing capacity to solve problems and make decisions nearly as well as an expert. Initially, KBES is developed to preserve knowledge in a particular domain because experts may die, retire, get sick, move on to other fields, and otherwise become unavailable, causing the knowledge they possess to disappear. Today, realizing the problem of knowledge loss, many experts document their expertise and leave the problem of application up to the reader. Hence the problem of saving knowledge does not have the highest priority any more, leaving the main goal of KBES to make knowledge more widely available and expertise more readily applicable.

A knowledge-based expert system is a software system capable of supporting the explicit representation of knowledge in some specific competence domain and of exploiting it through appropriate reasoning mechanisms to provide high-level problem-solving performance. It consists of three major components: a knowledge base, an inference engine, and a user interface, as shown in Figure 6.1.1. The knowledge base contains all the facts, ideas, relationships, and interactions of a particular domain. The inference engine analyzes the knowledge and draws conclusions from it. The user

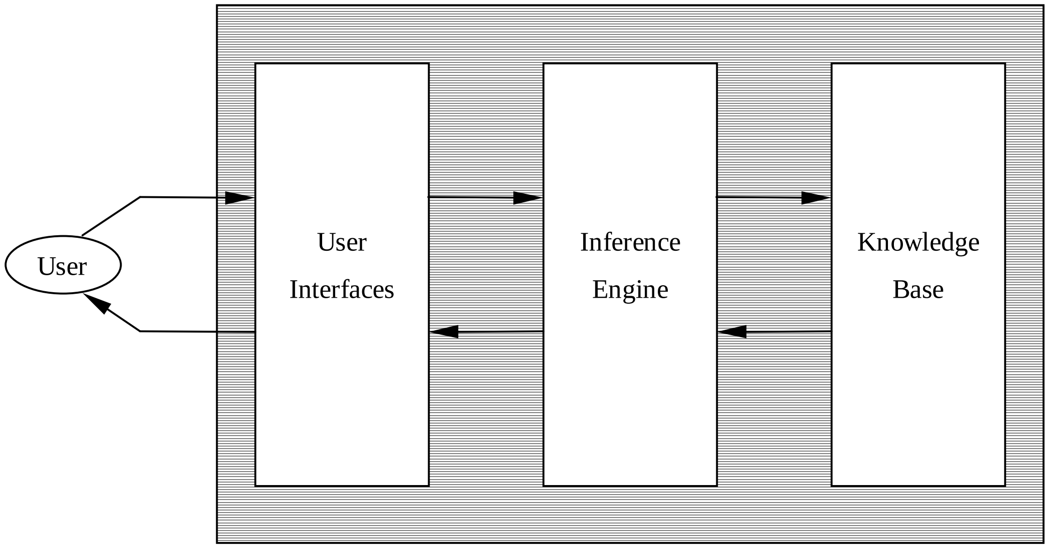


Figure 6.1.1 General block diagram of an expert system (After Frenzel, 1987)

interface permits new knowledge to be entered into the knowledge base and implements communication with the user. The user also provides the KBES with some initial information about the problem to be solved. Then the KBES proceeds to search for a solution and reach a conclusion. The output may be a recommended cause of action or the selection of the best alternative from a number of choices.

A well-implemented knowledge-based expert system furnishes users with a vast volume of expertise quickly and economically. It permits non-experts to do the work of experts, improves productivity by increasing work output by improving efficiency, saves time in accomplishing a specific objective, simplifies some operations, and automates repetitive, tedious, or overly complex processes. Compared to the human experts, a KBES has its advantages and disadvantages, as listed in Table 6.1.1. Despite its shortcomings, KBES still provides valuable and insightful solution to problems in a certain subject area. Especially for its constant availability and affordability, it is becoming more and more popular in areas where extensive expertise is required.

Table 6.1.1 Comparison of human and artificial expertise

	Advantages	**Disadvantages**
Human expertise	Creative Adaptive Sensory experience Broad focus Commonsense knowledge	Perishable Difficult to transfer Difficult to document Unpredictable Expensive
Artificial expertise	Permanent Easy to transfer Easy to document Consistent Affordable	Uninspired Needs to be told Symbolic input Narrow focus Technical knowledge

KBES development can be divided into eight major steps, as shown in Figure 6.1.2. These steps are highly interdependent and overlapping. As the development steps progress, each step modifies and refines the work done in the previous step. Especially after entering the testing and debugging step, it is very likely to go back to the very first step and start all over again step-by-step until the system works properly or program bugs get fixed.

2. KBES CANDIDATE EVALUATION

Knowledge-based expert systems have been successfully used to solve many different types of problems. However, researchers believe that for a KBES to be successful, the candidate application should fall into the categories shown in Table 6.2.1. Trouble-shooting bolting systems involves inferring malfunctions of a bolting system, based on the identification of the problem encountered, and then prescribing remedies for the malfunctions according to the knowledge accumulated from field application experience and laboratory investigations. Thus trouble-shooting bolting systems belongs to both categories of diagnosis and debugging described in Table 6.2.1.

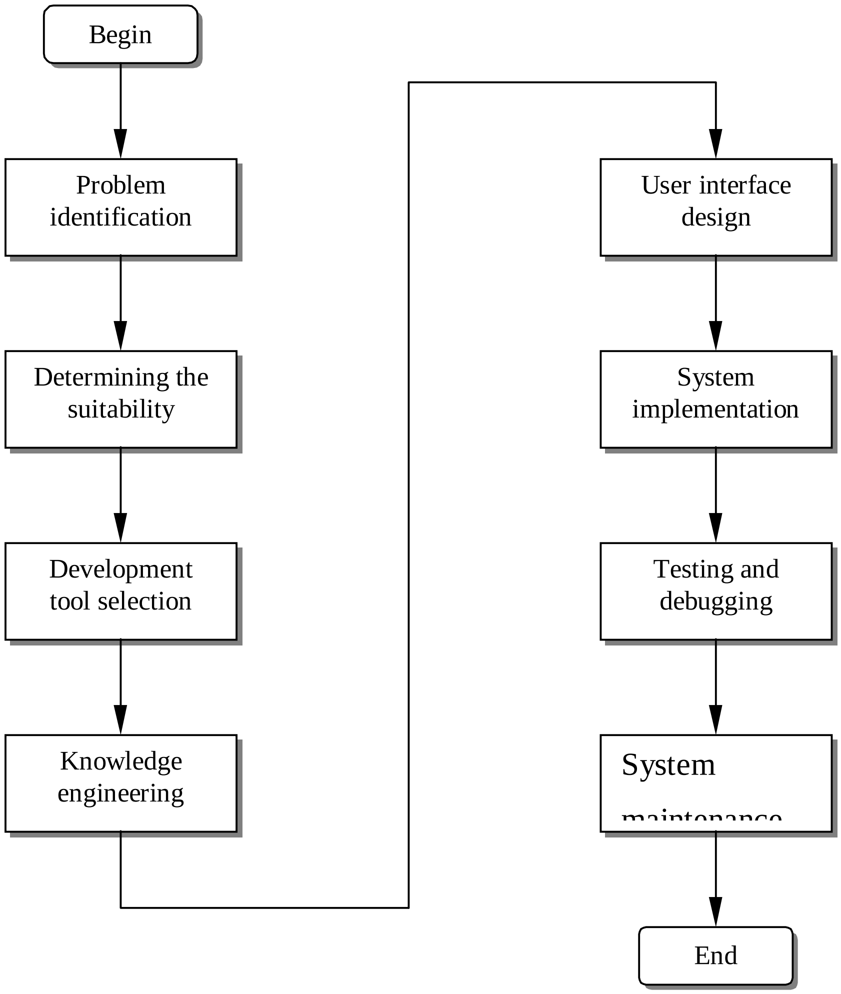


Figure 6.1.2 Major steps of developing a KBES

Table 6.2.1 Generic categories of KBES applications (Hayes-Roth et al., 1983)

Category	Problem addressed
Interpretation	Inferring situation descriptions form sensor data
Prediction	Inferring likely consequences of given situations
Diagnosis	Inferring system malfunctions from observables
Design	Configuring objects under constraints
Planning	Designing actions
Monitoring	Comparing observations to expected outcomes
Debugging	Prescribing remedies for malfunctions
Repair	Executing plans to administer prescribed remedies
Instruction	Diagnosing, debugging, and repairing student behavior
Control	Governing overall system behavior

Knowledge-based expert systems are usually more complex than conventional software. Designing and building a KBES is a major undertaking. It requires a lot of planning, design, programming, and debugging effort to create such a program from scratch. It is very important to examine in detail and evaluate the suitability of the application candidate. The suitability of an application candidate is measured by an overall candidate value (CValue), which is computed by combining the weighed scores of the features of the application. The higher the CValue, the better the candidate suits for the application of knowledge-based expert system. The features of an application include essential features and desirable features, both of which can be further classified into three groups: the users and their management, the task, and the expert. An essential feature is a feature that current expert system technology requires in order for the application to be a success. A desirable feature is a feature that is not necessarily required by current expert system technology; however, experience shows that without these features the difficulty of the candidate KBES project can significantly increase (Slagle and Wick, 1988).

To evaluate the application candidate, first assign a weight from 0 to 10 to each feature, both essential and desirable, depending on its relative importance. Then assign a feature value from 0 (absent) to 10 (fully present) for each feature and multiply the feature value by its corresponding feature weight to obtain a feature score. Sum the feature scores and weights over all essential and desirable features respectively. The overall candidate value is obtained by dividing the sum of all scores by the sum of all weights:

$$CValue = \frac{\sum_{i=1}^{n} weight_i \times value_i}{\sum_{i=1}^{n} weight_i} \qquad \text{Equation 6.2.1}$$

where n is the total number of all features.

Table 6.2.2 shows the weights and values for essential features of trouble-shooting bolting systems and Table 6.2.3 shows the weights and values of desirable features. The overall candidate value is 9.31, as shown in Table 6.2.4, which indicates that the selection of trouble-shooting bolting systems as a knowledge-based expert system candidate is appropriate and that the application is promising and has a very good chance to be a success.

3. DEVELOPMENT TOOL SELECTION

KBES development tools are programming systems that simplify the job of constructing a knowledge-based expert system. Selection of an appropriate development tool can significantly ease the developer's burden of coding and debugging and affect the overall performance of the KBES. It is thus wise to spend some time exploring what tools are available and what features they have and then among them choosing the best.

Generally, there are two basic types of KBES development tools: languages and shells. Languages are programming languages used with computer to create new

Table 6.2.2 Essential features evaluation form

Score	=	Weight	×	Value		
100	=	10	×	10	:	Recipients agree on high payoff
100	=	10	×	10	:	Recipients have realistic expectations
100	=	10	×	10	:	Project has management commitment
90	=	10	×	9	:	Task is not natural language intensive
63	=	7	×	9	:	Task is knowledge intensive
80	=	8	×	10	:	Task is heuristic in nature
100	=	10	×	10	:	Test cases are available
70	=	7	×	10	:	Incremental growth is possible
80	=	10	×	8	:	Task requires no common sense
80	=	8	×	10	:	Task does not require optimal solution
100	=	10	×	10	:	Task will be performed in future
70	=	7	×	10	:	Task not essential to deadline
72	=	8	×	9	:	Task easy, gut not too easy
100	=	10	×	10	:	An expert exists
100	=	10	×	10	:	Expert is a genuine expert
100	=	10	×	10	:	Expert is committed to entire project
80	=	8	×	10	:	Expert is cooperative
80	=	8	×	10	:	Expert is articulate
80	=	8	×	10	:	Expert has successful history
80	=	8	×	10	:	Expert uses symbolic reasoning
35	=	7	×	5	:	Hard to transfer expertise
80	=	10	×	8	:	Expert does not use physical skills
100	=	10	×	10	:	Experts agree on good solutions
70	=	10	×	7	:	Expert does not need creativity
2010	=	214				

Table 6.2.3 Desirable features evaluation form

Score	=	Weight	×	Value		
80	=	8	×	10	:	Management committed to follow-on
40	=	4	×	10	:	Insertion into work place smooth
40	=	4	×	10	:	System interacts with user
40	=	4	×	10	:	System can explain reasoning
40	=	4	×	10	:	System can intelligently question user
40	=	4	×	10	:	Task identified as problem area
40	=	4	×	10	:	Solution are explainable
45	=	5	×	9	:	Task does not require real-time response
40	=	8	×	5	:	Similar expert systems built before
50	=	5	×	10	:	Task performed in many location
30	=	3	×	10	:	Task performed in hostile environment
40	=	4	×	10	:	Task involves subjective factors
15	=	3	×	5	:	Expert unavailable in future
40	=	4	×	10	:	Expert intellectually attached to project
50	=	5	×	10	:	Expert does not feel threatened
12	=	2	×	6	:	Expertise loosely organized
642	=	71				

Table 6.2.4 Overall Evaluation form

Score		Weight		CValue
2010		214		
642		71		
2652	/	285	=	9.31

software. Languages used for KBES applications can be categorized into assembly languages, problem-oriented languages such as FORTRAN and PASCAL, symbolic-manipulation languages such as LISP and PROLOG, and object-oriented languages such as C, C++, and JAVA. The high level languages are usually much easier to use and among them object-oriented languages are more efficient. Assembly languages produce high-speed code, but coding and debugging are big challenges for programmers. If high system performance is not absolutely necessary, assembly languages should never be considered. The most popular and widely used programming language for KBES applications is LISP, although PROLOG is also widely used. Both of them are specially designed for artificial intelligence applications to manipulate massive symbolic data. They also provide automatic memory management, sophisticated editing and debugging aids, and uniform treatment of program code and data. However, they fail to provide guidance on how to represent knowledge or mechanisms for accessing the knowledge base. Although languages are suitable for developing KBES with great flexibility to developer, all high level program languages need considerable amount of code and require very competent and experienced programmer to accomplish the task.

A knowledge-based expert system shell or generator is a special software package designed specifically to help build a KBES. It provides a development environment, which includes an empty knowledge base, an inference engine, a user interface for programming, and a user interface for running the system. The programming interface typically comprises of a series of specialized editors for creating rules, defining functions, and coding methods, and some debugging tools. The user at this stage usually programs in a declarative way without concerning the workings of the inference engine. Thus, the total effort of building a KBES is significantly reduced, leaving a great deal of coding work automatically done by the facilities provided by the shell. However, the automatically generated code is not as efficient as that written by individual programmer since the code generators are intended to generally deal with as many situations as possible. The deterioration of the KBES performance due to code inefficiency is not distinctively noticeable, especially when the KBES is of moderate or small size. Nevertheless, with the advance of hardware technology, the increasing computation speed of modern computers helps overcome the code inefficiency.

There are many expert system shells available. The most widely used shells are KEE, ART, and Goldworks, all of which are built using LISP. Nexpert, ProKappa, and Kappa-PC, which are built using C or C++, are becoming more popular as the power of these high-level programming languages is being realized and appreciated.

Trouble-shooting bolting systems is a medium-sized application. The ultimate users of this application will be mine engineers who deal with bolting problems from day to day and college students who want to get familiar with bolting problems. Thus, it is desired that the KBES can be installed and run on personal computers such that it can be easily accessed. Kappa-PC is a development tool that allows a knowledge-based expert system to be created and run on personal computers. The operating system it requires is Windows version 3.1 or later. The minimum RAM is 8 megabytes. The major features Kappa-PC provides include:

- Object Oriented Programming: Objects and Methods are the building blocks of every Kappa-PC application.
- The KAL Programming Language: KAL is the programming language used in Kappa-PC. It is an interpreted language designed for rapid prototyping. It comes with a source level debugger, KALView. Before delivering an application written in KAL, it needs to be compiled into C using the KAL compiler.

- Rule Based Reasoning: As an alternative to object-oriented programming, Kappa-PC offers an inference engine for rule-based reasoning, both forward-chaining and backward-chaining.
- Creating Graphical User Interfaces: Kappa-PC is also an object-oriented graphic user interface (GUI) development tool. In addition to its set of predefined images, Kappa-PC applications can take advantage of Visual Basic (VBX) controls with a simple plug-in interface.
- Integrating with Windows: Applications developed with Kappa-PC can take advantage of the potentials offered by the Microsoft Windows environment.

4. KNOWLEDGE ENGINEERING

Knowledge engineering is the process of transferring and transforming problem-solving expertise and experience in a certain domain from knowledge sources into a computer program. It includes several stages before producing a knowledge-based expert system. These stages can be characterized as knowledge acquisition, knowledge abstraction, and knowledge representation. This is the most essential step among all eight steps of building a KBES. It affects the problem-solving quality of the expert system, determines the architecture of knowledge organization, and as a result affects the system performance.

4.1 KNOWLEDGE ACQUISITION

The knowledge acquisition process of this project is unique. The Mine Safety and Health Administration (MSHA), Roof Control Division (RCD) of the Pittsburgh Safety and Health Technology Center accumulated massive knowledge about trouble-shooting roof support systems from its involvement in the design and application of bolting practices in mining industry and laboratory investigations. It also documented and formulated the knowledge for the mining industry, academic institutions, and research agencies to share. Its publications are the primary knowledge source for this project.

The knowledge engineer himself is a graduate student in Mining Engineering and had been working as mining engineer in mine design business for eight years. He is not an expert in roof bolting system, but possesses a considerable amount of knowledge. In addition, some of the members of his advisory committee of graduate study are experts in roof bolting systems with many years of academic and practical experience. All of them serve as the secondary knowledge sources.

4.2 KNOWLEDGE ABSTRACTION AND REPRESENTATION

The term *knowledge* in KBES has a different meaning from that of ordinary English. In expert systems, knowledge means the information a computer needs before the expert system can behave intelligently. It needs to be abstracted from the knowledge source and represented in ways that a computer can manipulate efficiently. There are two predominant ways supported by most expert system shells to represent the knowledge. One is rule-based representation. This method uses *IF condition THEN action* statements to represent knowledge. When the condition in *IF* part is satisfied or matched, that rule will be fired and the action specified by the *THEN* part is then triggered. The action may affect the outside world such as causing text to be printed, directing program control such as testing and firing other rules, or instructing the system to reach a conclusion such as adding a new fact to the knowledge base. This method is especially useful when the expert system requires intensive reasoning for solving a problem. The other knowledge representation method is frame-based. It uses a network of classes and instances (also called nodes or frames) connected by relations and organized into a hierarchical structure. Figure 6.4.1 shows the relationships between class and instance. Each node represents a concept that may be described by attributes and/or values associated with the node. The topmost nodes represent general concepts and the lower nodes represent more specific instances of those concepts. Frame-based representation is able to clearly document information in a natural way and modularize the information, permitting relatively easy system expansion and maintenance. This method is also fully supported by the power of object-oriented programming languages such as information encapsulation and inheritance.

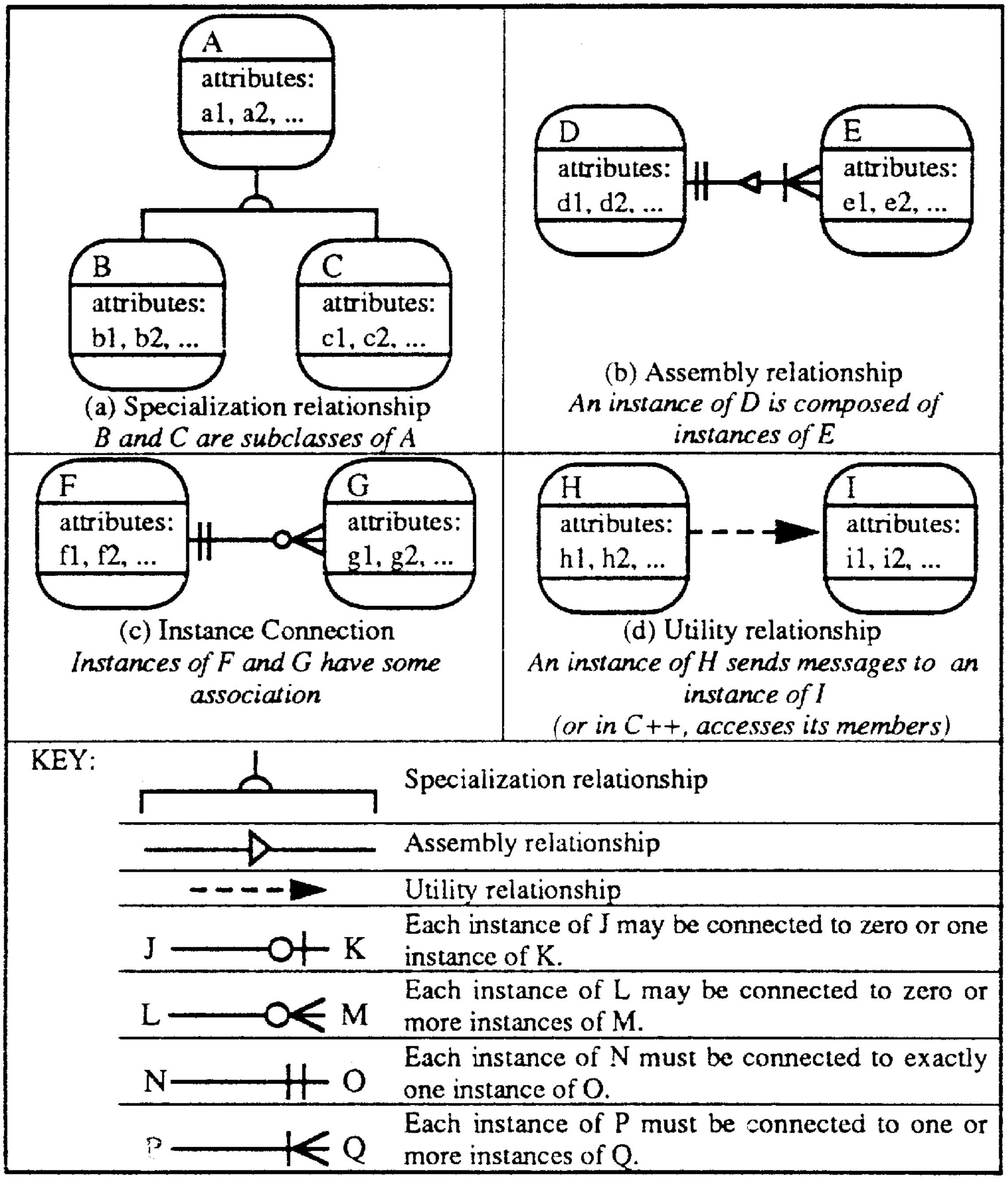


Figure 6.4.1 Class and instance relationship (After Coad and Yourdon, 1990)

Kappa-PC supports both methods of knowledge representation. But the nature of trouble-shooting bolting systems makes frame-based representation more efficient and insightful. In Kappa-PC environment, taxonomic process follows a general format as below:

Class → Subclass → Subclass → …… → Instances

In the application of trouble-shooting bolting systems, the topmost class is labeled as Bolt, which has a very broad meaning and semantically includes all types of bolts. Then it is classified into five subclasses: mechanical bolts, fully grouted bolts, tensioned rebar bolts, combination/point anchored bolts, and mechanically anchored resin assisted bolts. All of these types represent a class of bolts that have the common attributes and components. The definitions of these types are described in Chapter 5. Each of the five subclasses is further divided into subclasses, which are components of different types of bolts such as plate, washer, and anchor. Torque and tension are classified as bolt components, though they are actually not. They are treated as abstract components. Each component is instantiated into instances representing possible problems that the parent component may encountered. Figures 6.4.2 to 6.4.6 show the hierarchical architectures for trouble-shooting bolting systems. Attributes of a frame are represented as a set of slots, which in turn can also have attributes such as VALUE. The allowable values of VALUE can be numerical, boolean, or string values. Most of the attributes of the problems encountered by bolt components have boolean values to indicate whether the problem is present or not. A series of actions can take place, as specified beforehand, before and after change of the VALUE or if the VALUE is accessed. The actions are defined by methods and, like the actions in the THEN part of a rule, may affect the outside world, direct program control, or instruct the system to reach a conclusion. Methods are attached to a frame and can be accessed through calls from slots of the same frame. Method is a kind of function manipulating attributes of the frame to which it attaches and attributes of other frames as well. In conjunction with slots, the function of methods is very similar to that of rules in rule-based knowledge representation.

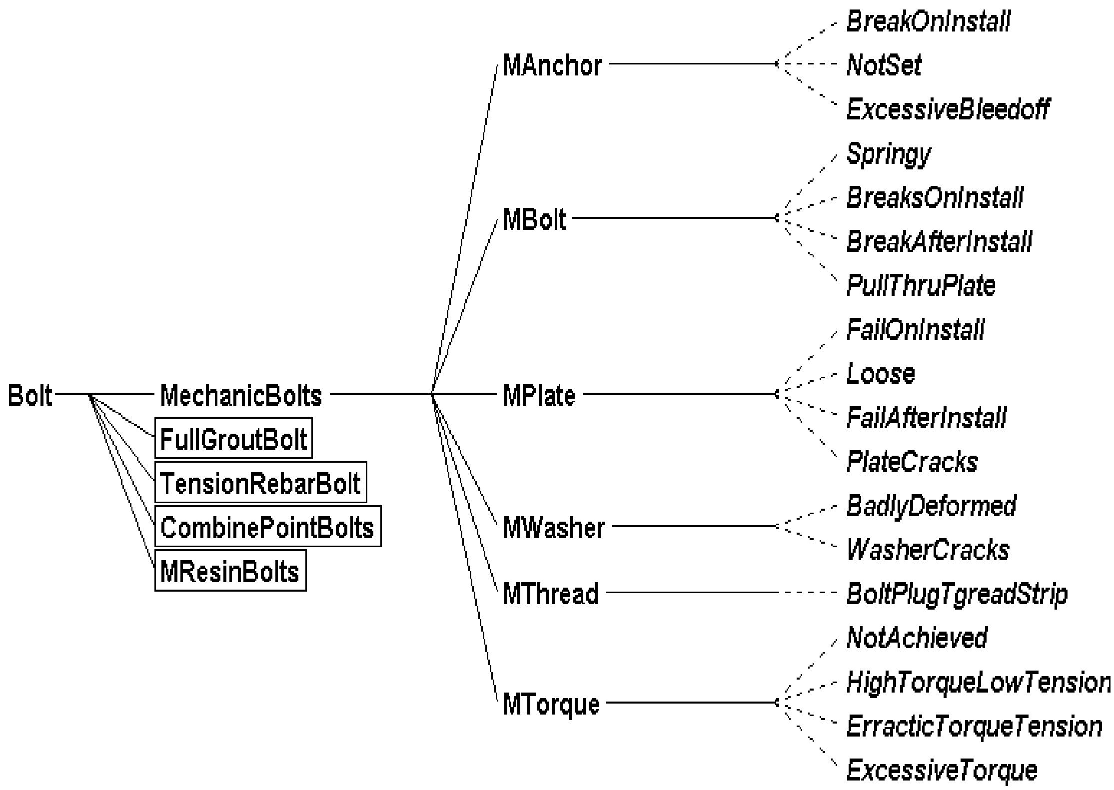


Figure 6.4.2 Hierarchical architecture of mechanical bolts

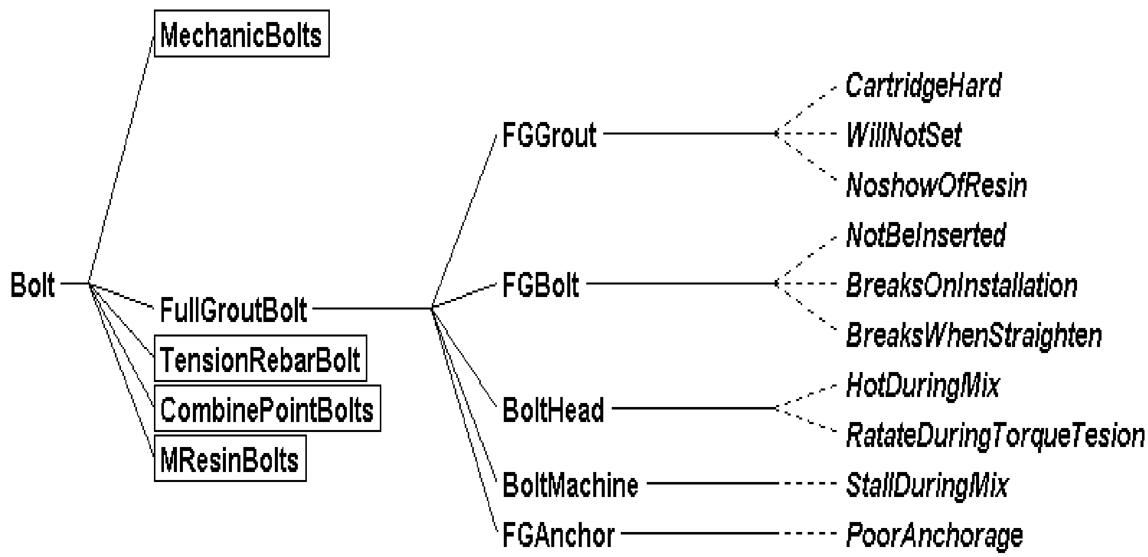


Figure 6.4.3 Hierarchical architecture of fully grouted bolts

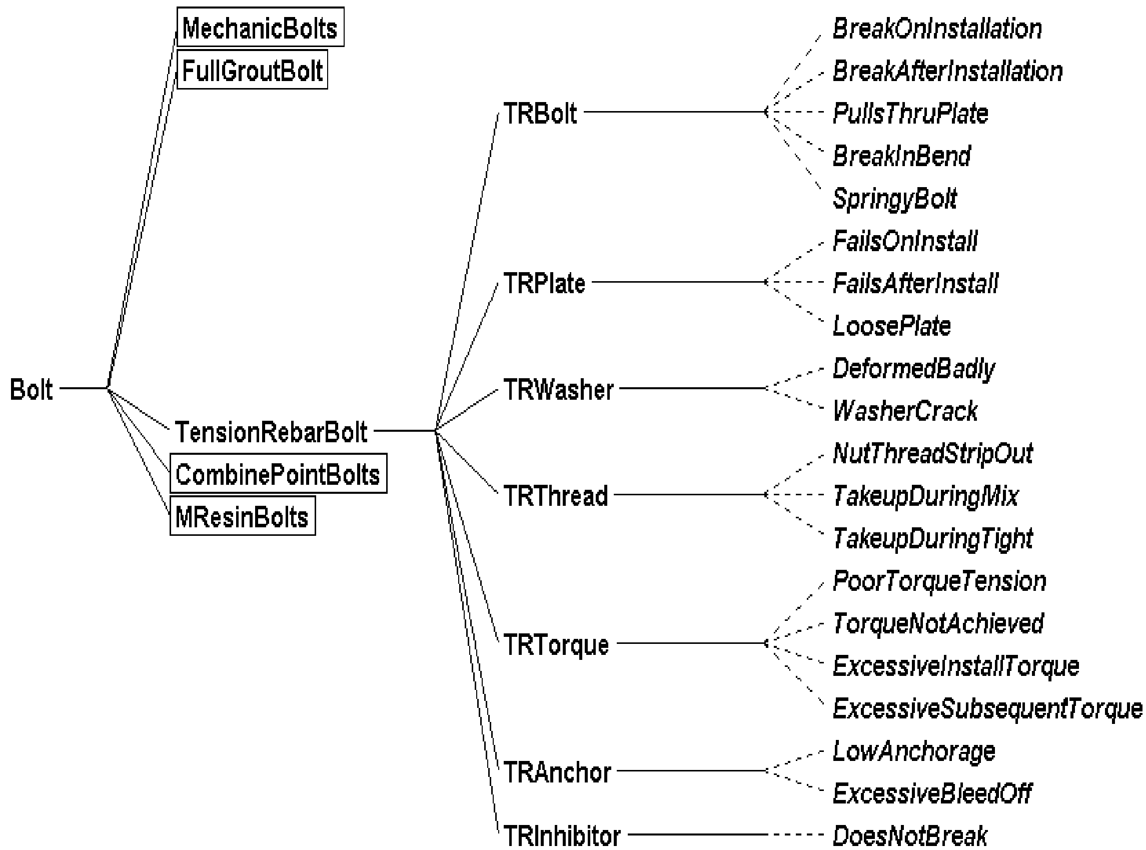


Figure 6.4.4 Hierarchical architecture of tensionedrebar bolts

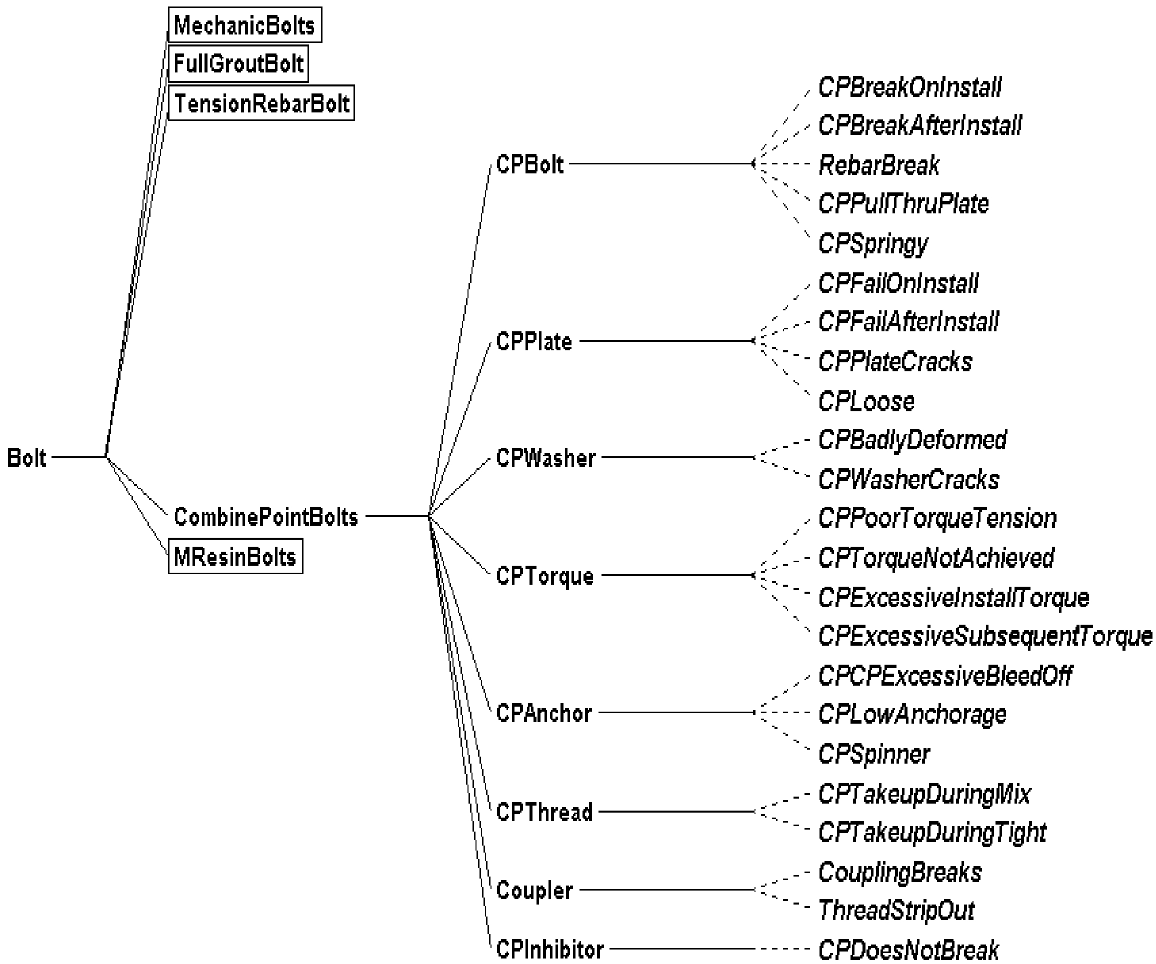


Figure 6.4.5 Hierarchical architecture of combination/point anchored bolts

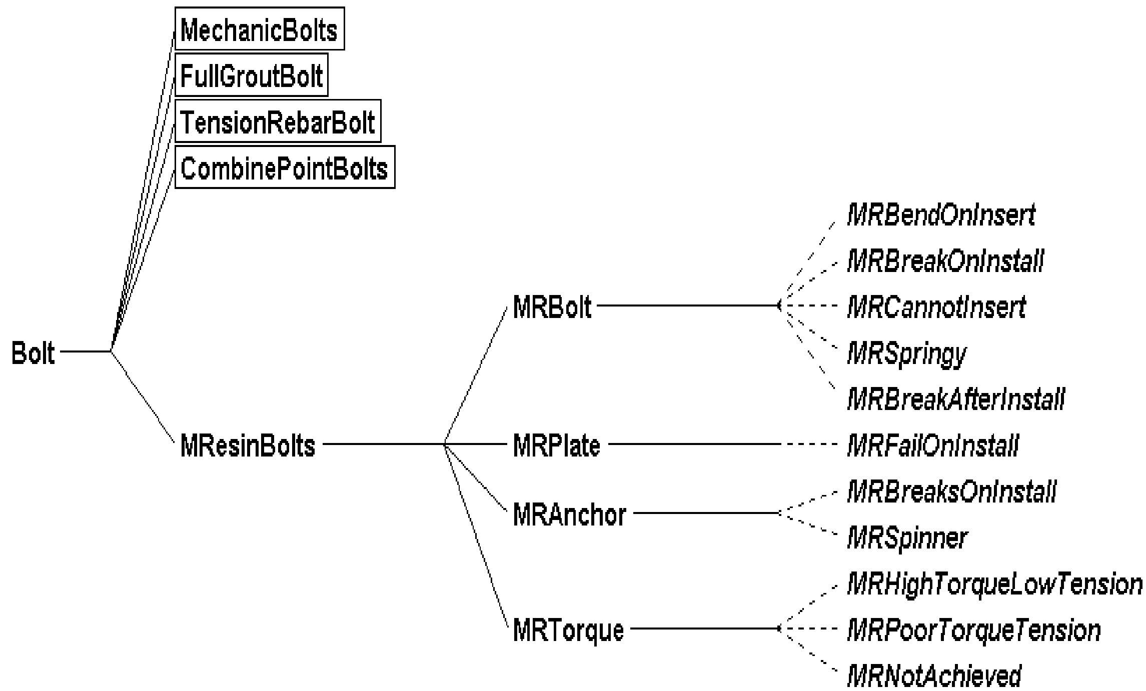


Figure 6.4.6 Hierarchical architecture of mechanically anchored resin assisted bolts

Kappa-PC allows knowledge engineer to define external functions. The functions can be called either by methods or directly by images in user interfaces such as radiobuttons, sliders, and checkboxes. This feature provides great flexibility for knowledge engineer to manipulate knowledge and substantially simplifies knowledge representation. Trouble-shooting methods and remedial procedures are implemented as external functions.

5. USER INTERFACE DESIGN

User interface, as the name indicates, is the interface between the user and the knowledge-based expert system. The user communicates with the system via these interfaces. Generally, user interfaces obtain information that are needed to solve a particular problem by asking the user to answer questions or prompting the user to provide relevant information in preformatted way. Considering that KBES users are neither experts of trouble-shooting a bolting system nor very experienced computer users, the interface must be friendly and encouraging. At least it should not frustrate the user. On the other hand, the user interfaces should be designed in such a way that they collect information effectively and present the whole expert system structurally without worsening the overall performance. A good graphic user interface also reduces the learning time required to get started for first-time users and the number of mistakes users make when using the system. Regardless of how well the knowledge base is organized or how effective the system's performance, the quality of the user interface design alone may determine the fate of the knowledge-based expert system. In addition to imbedding those design principals to the process of user interface design for trouble-shooting bolting systems, background colors and font sizes and colors are carefully chosen to enhance the visual effect of the interfaces.

Kappa-PC provides powerful tools for creating graphic user interfaces. In addition to session windows, predefined images, custom menus, and dialog windows, it can import Visual Basic (VBX) controls to meet knowledge engineer's specific requirements. The graphic user interfaces of trouble-shooting bolting systems are classified into four types in terms of their functionality: general information, bolt type selection, problem-solving, and output.

General information interface is the first window invoked after the KBES is launched, as shown in Figure 6.9.1. It contains information about the KBES title, developer, version series number, and development date.

Bolt type selection interface has three sections: program title, type selection section that comprises of five buttons representing five different bolts along side the detailed bolt type definitions, and message section containing messages for actions to take in this interface, as shown in Figure 6.9.2.

Problem-solving interface has five versions. Each of them is customized for different bolt types. Figure 6.9.3 is an example of this kind of interface. Except for the interface for mechanically anchored resin assistant bolt, all of them have the same layout, which includes five sections: interface title; a section with five buttons linking to the other counterpart interfaces; a section that lists all possible problems with a certain type of bolts and message instructing the user what to do in this section; a section used to list most likely, secondary, and potential causes; and a section consisting of two buttons linking back to bolt type selection interface and exiting the KBES respectively. Since mechanically anchored resin assisted bolts often have the same installation problems encountered as with mechanical bolts, the interface only lists the most typical problems and the pertaining possible causes. For other problems associated with mechanically anchored resin assisted bolts, trouble-shooting procedure is identical to that of mechanical bolts.

Problem-solving interface is the essential part of the user interface design, serving as the direct bridge between the user and the knowledge-based expert system. All the information needed for solving a problem is obtained here. To facilitate the usage, all possible problems are listed. The user can make selection according to the problem encountered. After the selection is made, a set of methods will be executed and as a result all probable causes are listed in another section and grouped into three categories. Then the user examines the list sequentially to identify the actual cause.

There are two types of output interfaces. One is for viewing the analysis results. It is designed as a pop-up window, as shown in Figure 6.9.5. It displays the trouble-shooting method and remedial procedures for the problem the user identified. This window has two command buttons. The user can click one simply to close the viewing window and click the other to spawn the printing window. The other type output interface is actually a *Notepad* text editor, as shown in Figure 6.9.6. It displays the type of the bolt, problem encountered, actual cause, trouble-shooting method, and remedy. The content here is preformatted. But the user can edit it as needed and print the analysis result by issuing a print command from the pull-down menu of the editor.

6. SYSTEM IMPLEMENTATION

System implementation is the process of turning the design concepts and abstracted knowledge into a working computer program. Kappa-PC provides various development tools for entering and editing code and creating graphic user interfaces. After the knowledge engineering and the interface design are successfully carried out, system implementation becomes a relatively simple routine task, though tedious and time-consuming. All methods and external functions are written in KAL, an object-oriented programming language supported by Kappa-PC environment. For better readability and maintainability, the source code is programmed and documented in accordance to the common standards practiced by student programmers at Virginia Tech.

7. TESTING AND DEBUGGING

After the completion of the system implementation, the system must be tested to see whether it functions as designed. If not, it is then the knowledge engineer's responsibility to detect the bug and fix it. A thorough system testing is done not only by the knowledge engineer but the experts as well. All the different paths within the system are tested using hypothetical cases. After making sure that the system works properly, the system is then expert-validated. A final modification is made before the system is ready for distribution.

8. SOFTWARE DISTRIBUTION AND MAINTENANCE

Kappa-PC produces a text file, which is interpretable by the Kappa-PC environment, instead of an executable file. Kappa-PC uses this text file as input file. It reads in the file, interprets it, and then internally translates it into an executable file, significantly reducing the execution speed. Another major drawback is that the knowledge-based expert system is insecure because the text file is accessible and editable by the expert system users. The user thus may unintentionally modify the file and cause fatal errors to occur. Kappa-PC provides some utilities that help translate the KAL file into C files, but the procedure is quite tricky.

Usually KAL files are too large to be translated into a single C file. This limitation is imposed by the C compiler because an object code segment in the Microsoft Windows environment can not exceed 64k. The KAL file must be broken down into several small files to meet this limitation. The simple and safe way of partitioning the KAL file is to separate KAL functions, objects, and methods by putting the functions file first. The *kalmake.com* utility takes the partitioned files in the order they should be compiled followed by the flags to be passed on to the KAL compiler. The *kalmake.com* utility produces a makefile that can be used with a *make* facility to simplify the compilation process. The generated makefile contains all the statements and dependencies necessary to compile the KAL files, C files, and RC file, and then links the resulting files into a new Dynamic Linked Library (DLL) file. Microsoft C 8.0 compiler and linker are used. The compiled DLL file is executable but dependent on the Kappa-PC environment. After compilation, execution speed is at least three times faster than that of a KAL file and the possibility of accidentally destroying the KBES is totally eliminated.

All the files required for executing the knowledge-based expert system are packed into a 1.44M high-density diskette. This KBES requires a Windows 95 or later version

Windows operating system. To install the KBES to a computer, run the setup.exe program in the distribution diskette. For some computers, the setup program may not be able to create the startup program icon automatically. The user can create one manually by following the user's guide of the Windows operating system. The startup program icon is not the only way of launching the KBES. Alternatively, the user can start the program by double-clicking the file named *BoltSystem.exe*. The program has the best visual results on computers with resolution of 800×600.

Like any conventional software, knowledge-based expert system requires intensive maintenance efforts. Knowledge in the domain of trouble-shooting bolting systems is dynamic. Some knowledge may become outdated and more new knowledge will be accumulated as the roof bolting technology is practiced more and more widely. On the other hand, unexpected bugs may be encountered by the end-users and fed back to the system developer. To keep the knowledge-based expert system valid and updated, the system must be modified to accommodate those changes. With the Kappa-PC development tool, modifying the knowledge base itself is relatively easy and quick. The challenging part is to determine what new knowledge is needed and what old knowledge is not. After entering new knowledge, part of the user interface needs minor modification to reflect knowledge base changes. Since the KBES uses frame-based method to represent knowledge, adding or deleting knowledge involves tremendous programming. Program documentation and readability play vital roles in reducing the difficulty of software maintenance. The system must be tested and validated again after any modification, though the testing process need not be exhaustive. Only the part that is affected by the change of knowledge must be tested and validated thoroughly.

9. SAMPLE EXECUTION

Suppose that a mine uses combination/point anchored bolts as the primary support method to support underground openings. During installation, a springy bolt is encountered. The engineer wants to identify the actual cause and take corresponding action to correct the problem.

First, launch the KBES by clicking the program startup icon or double-clicking on the executable file from Windows Explorer. A console window shows up followed by the logo of Kappa-PC, which is then replaced by three small widows. These three windows stay for a while when the expert system is being loaded. After the program is loaded, the general information interface (Figure 6.9.1) appears in the center of the displaying area if the computer has a resolution of 800×600. Clicking on the *OK* button closes the general information interface and invokes the bolt type selection window, as shown in Figure 6.9.2. Since the bolt type used in the mine is combination/point anchored bolts, clicking on the button labeled *Combination/Point Anchored Bolts* leads the user to the problem-solving interface for this type of bolt, as shown Figure 6.9.3. There are 23 possible problems associated with combination/point anchored bolts listed in this interface. Select the one that matches the problem the user encountered during installation — in this case, springy bolt. After the selection, the most likely, secondary, and potential causes of a springy bolt are listed on the second half of the interface, as shown in Figure 6.9.4. The user now should browse through all eight possible causes. Because the causes' names are quite meaningful, it is possible for the user to identify the actual cause merely by being prompted by the possible causes' name. If the actual cause is identified — for example, the torque inhibitor is too strong/weak — then clicking on that cause calls up the viewing window that displays the trouble-shooting method and remedy, as shown in Figure 6.9.5. If the user cannot identify the actual cause this way, then it is necessary to start from the most likely causes, click on each possible cause sequentially, read the trouble-shooting method, and do what the trouble-shooting method suggests, until the actual cause is found. Clicking *Print* button on viewing window spawns a Notepad text editor with analysis result, as shown in Figure 6.9.6. The user can print out the result by using *print* command in the pull-down file menu. Of course, the user can reformat and edit the content of the analysis result if it is needed.

Once finishing the use of the KBES, clicking on *Exit* button on the problem-solving interface pops up a dialog window. This dialog window is designed to avoid accidental abortion of the program and unwanted data loss.

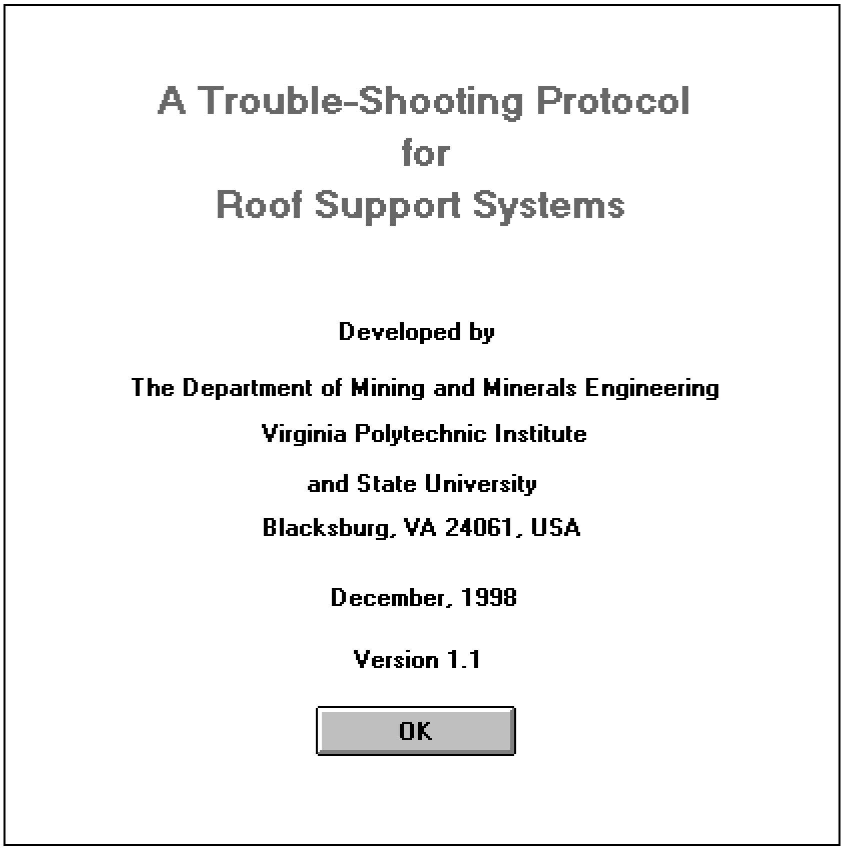


Figure 6.9.1 General information interface

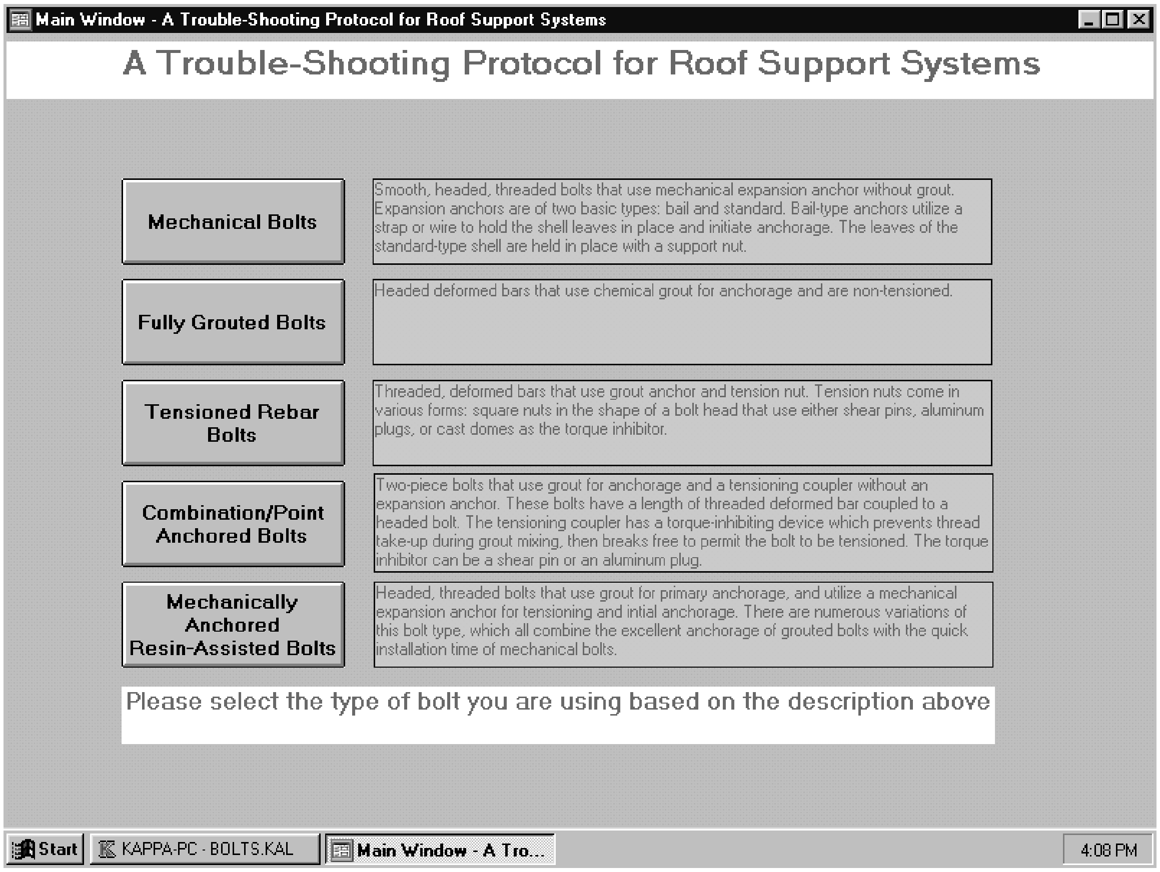


Figure 6.9.2 Bolt type selection interface

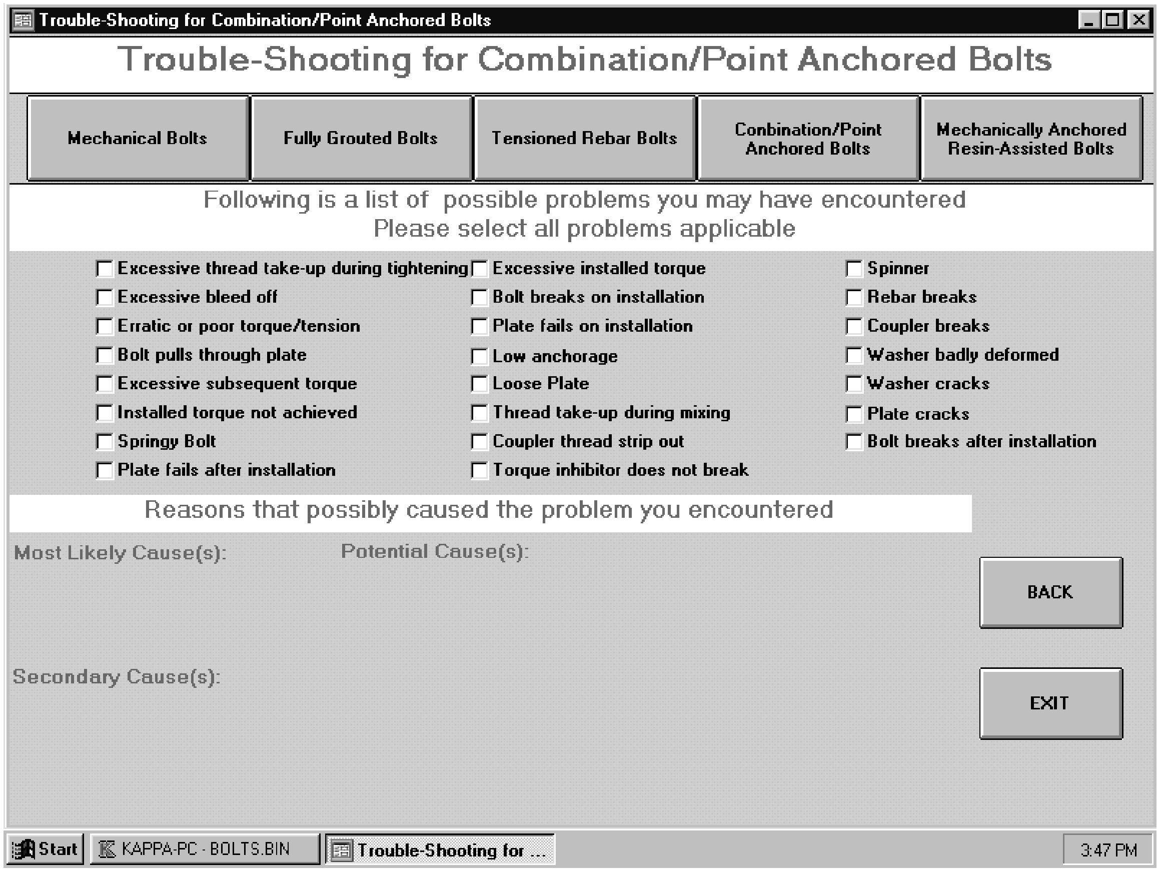


Figure 6.9.3 Problem-solving interface for combination/point anchored bolts

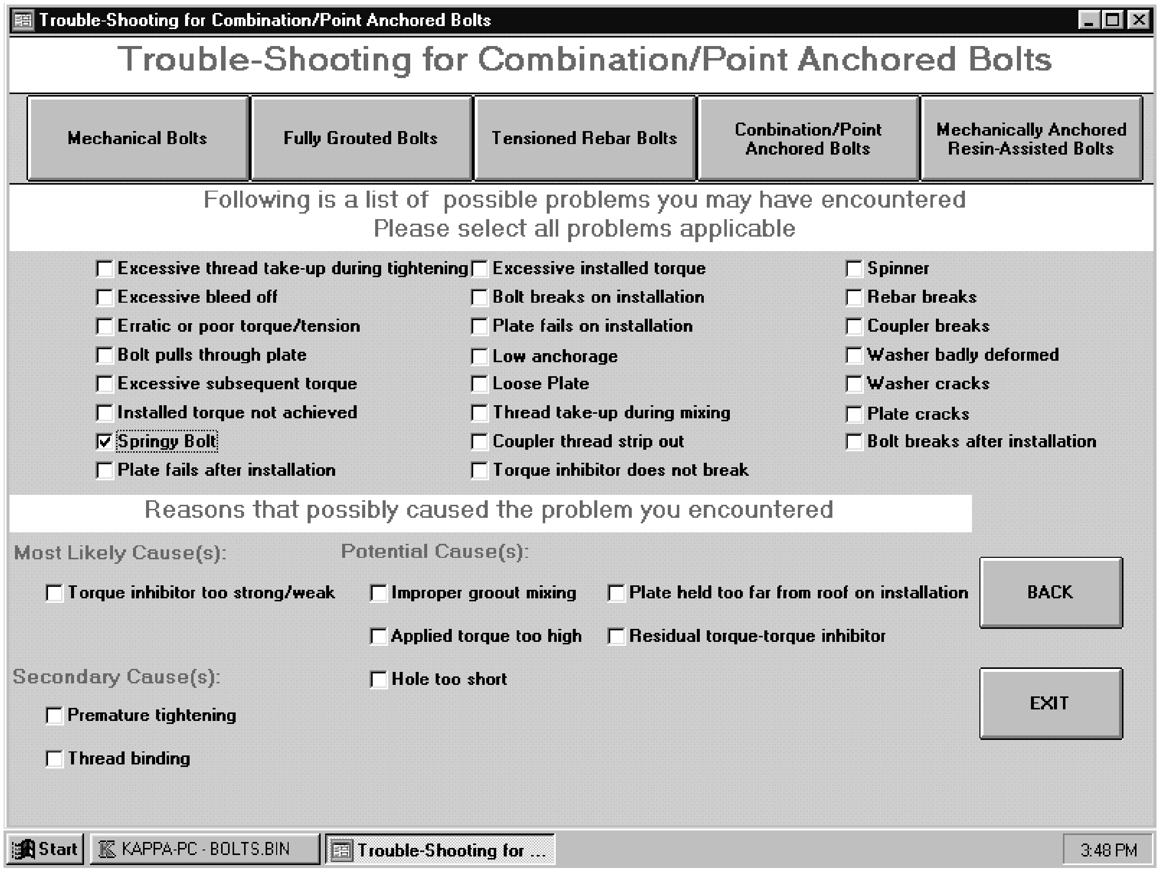


Figure 6.9.4 Problem-solving interface after problem identified

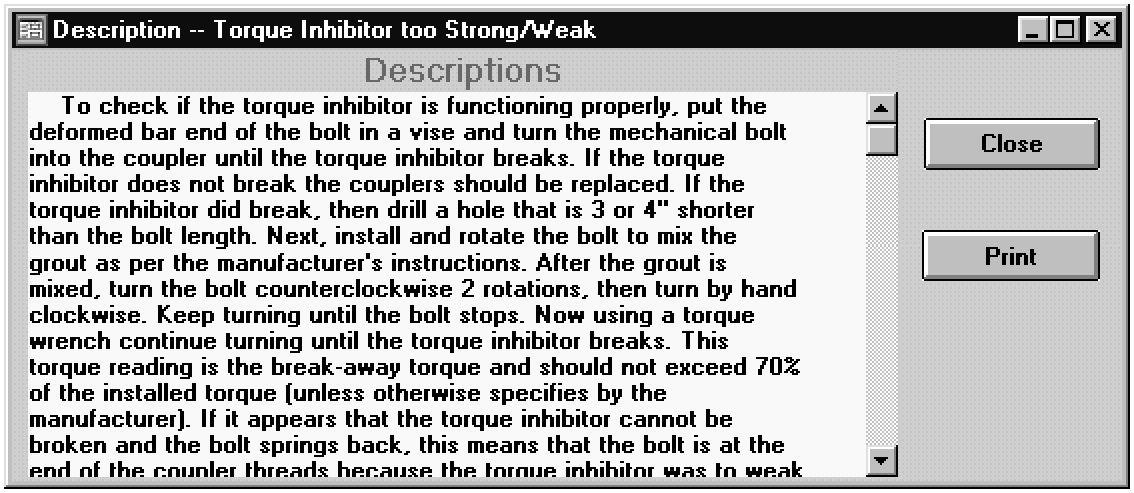
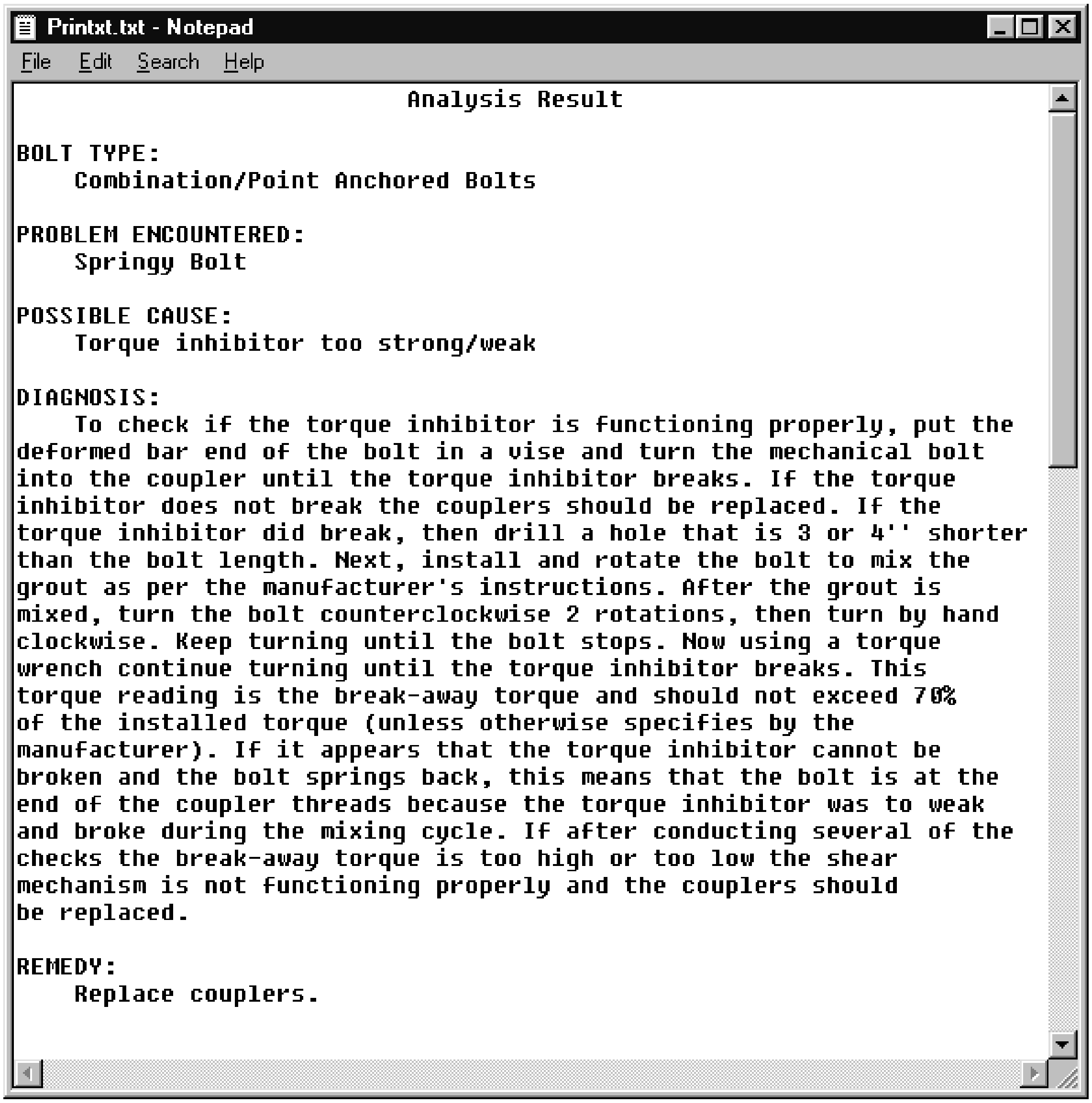


Figure 6.9.5 Viewing window displaying analysis result



Printxt.txt - Notepad

File Edit Search Help

Analysis Result

BOLT TYPE:
Combination/Point Anchored Bolts

PROBLEM ENCOUNTERED:
Springy Bolt

POSSIBLE CAUSE:
Torque inhibitor too strong/weak

DIAGNOSIS:
To check if the torque inhibitor is functioning properly, put the deformed bar end of the bolt in a vise and turn the mechanical bolt into the coupler until the torque inhibitor breaks. If the torque inhibitor does not break the couplers should be replaced. If the torque inhibitor did break, then drill a hole that is 3 or 4'' shorter than the bolt length. Next, install and rotate the bolt to mix the grout as per the manufacturer's instructions. After the grout is mixed, turn the bolt counterclockwise 2 rotations, then turn by hand clockwise. Keep turning until the bolt stops. Now using a torque wrench continue turning until the torque inhibitor breaks. This torque reading is the break-away torque and should not exceed 70% of the installed torque (unless otherwise specifies by the manufacturer). If it appears that the torque inhibitor cannot be broken and the bolt springs back, this means that the bolt is at the end of the coupler threads because the torque inhibitor was to weak and broke during the mixing cycle. If after conducting several of the checks the break-away torque is too high or too low the shear mechanism is not functioning properly and the couplers should be replaced.

REMEDY:
Replace couplers.

Figure 6.9.6 Notepad text editor displaying the analysis result

CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS

1. CONCLUSIONS

Based on the research, the following conclusions can be reached:

1. As a major support method of underground structures, rock bolting is very effective and efficient, greatly reducing fatal accidents and timber consumption. Structures reinforced by bolts are typically very reliable and long lasting. However, a rational basis for all bolting system designs has still not been fully achieved. Fortunately, the successful and unsuccessful bolting practices of the last 50 years provide abundant empirical experiences for bolt utilization.

2. The immediate roof experiences a sharp deflection immediately after the opening is excavated, following by a transient moment of negligible movement and then the immediate roof keeps sagging to reach a stable state or finally cave in. This displacing pattern indicates that installing bolts immediately after excavation is beneficial since the sudden movement helps build up axial force along the bolt, which is essential for beam building effect to develop.

3. If bolts are installed immediately after the opening is excavated, pretension becomes redundant in maintaining the stability of the opening. But there is a subtle trend that indicates that the deflection of the immediate roof decreases as the pretension increases. Only if the immediate roof contains many fractures is pretension required as a supplement to the forces generated by the initial roof movement.

4. The number of bedding planes in the immediate roof has a significantly impact on the magnitude of vertical displacement of the immediate roof strata. A relationship between them was established using linear regression. Although the

correlation is based on a specific case, it is generally indicative that the more the numbers of the layers within the bolting range the less stable the immediate roof.

5. Given the properties of immediate roof strata and the overburden thickness, the minimum solid beam thickness can be predicted with acceptable accuracy by statistical models. The predicted value is very useful in determining the optimum bolt length when designing a bolting system.

6. The successful development of the knowledge-based expert system for trouble-shooting bolting systems demonstrates that the application of KBES in this domain is promising. Frame-based knowledge representation method in conjunction with the power of object-oriented programming enables the knowledge to be abstracted and represented for the computer to manipulate efficiently and for the knowledge engineer to update the knowledge base with great flexibility. Since the knowledge in this domain is dynamically growing, the KBES protocol is by no means complete at this stage. Alternatively, due to the diversity of factors affecting the ground control problem, the KBES protocol is not intended to solve all problems encountered in bolting practice. However, this trouble-shooting protocol provides an ideal tool for mine engineers who deal with bolting problems from day to day and in the education process for students who want to be familiar with bolting problems.

2. RECOMMENDATIONS

Experiments based on the FLAC numerical model show that pretension applied to bolts on installation has no significant effect on reducing the magnitude of displacement in the immediate roof. This conclusion conflicts with opinions that are well accepted by the mining industry. The main reason for this conclusion is perhaps the assumption that both sides of the interface are directly and perfectly in contact. For future investigation, it is recommended that the bedding planes be modeled more completely by adding filling materials to represent gas and water pressure.

The quality of the statistical model predicting the minimum solid beam could be improved. The model needs additional data sets to improve its accuracy. Also, better experimental design technique can be considered as an alternative way to enhance prediction quality.

Theoretically, a bolted beam has the same strength as solid beam of same geometry and material. In practice, it is found that the bolted beam is always weaker. Before the minimum solid beam thickness can be used to determine the optimum bolt length, it is necessary to explore the strength difference between these two types of beams.

Due to the difficulty of estimating the horizontal stress in terms of overburden thickness, horizontal stress is ignored in the FLAC models. However, in many mines, horizontal stress plays a substantial role in bolted roof failure. It is recommended that horizontal stress be added into the FLAC models as independent variable and its effects on the opening stability be thus investigated.

The knowledge in trouble-shooting bolting systems is constantly growing. The developed KBES should reflect the latest information in this domain. Therefore, it is a periodical task to assess new knowledge and add it to the knowledge base.

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APPENDIX A

SAS OUTPUT OF LINEAR REGRESSION AND VARIABLE SENSITIVITY ANALYSIS

MINIMUM THICKNESS OF ROOF BEAM 1

Model: MODEL1
Dependent Variable: Y

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Prob>F
Model	6	45.82152	7.63692	19.863	0.0001
Error	43	16.53225	0.38447		
C Total	49	62.35378			

Root MSE	0.62006	R-square	0.7349
Dep Mean	3.02690	Adj R-sq	0.6979
C.V.	20.48490		

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	T for H0: Parameter=0	Prob > |T|
INTERCEP	1	5.053691	0.56374541	8.964	0.0001
X1	1	-0.000001258	0.00000031	-4.085	0.0002
X2	1	0.772335	0.88431854	0.873	0.3873
X3	1	0.001578	0.00032194	4.901	0.0001
X4	1	-0.000127	0.00006683	-1.908	0.0631
X5	1	-0.002294	0.00029642	-7.739	0.0001
X6	1	-0.040195	0.00851159	-4.722	0.0001

MINIMUM THICKNESS OF ROOF BEAM 2

N = 50 Regression Models for Dependent Variable: Y

Number in Model	R-square	Adjusted R-square	C(p)	Variables in Model
1	0.30588887	0.29142823	66.57142	X5
1	0.11402658	0.09556880	97.68777	X6
1	0.11378508	0.09532227	97.72694	X3
1	0.09984955	0.08109641	99.98702	X1
1	0.01481499	-.00570970	113.77798	X4
1	0.00173983	-.01905725	115.89852	X2
2	0.48147743	0.45941264	40.09434	X5 X6
2	0.45714756	0.43404746	44.04018	X3 X5
2	0.42189738	0.39729727	49.75708	X1 X5
2	0.30721554	0.27773535	68.35626	X4 X5
2	0.30604228	0.27651216	68.54654	X2 X5
2	0.22068680	0.18752453	82.38955	X3 X6
2	0.20495467	0.17112295	84.94099	X1 X3
2	0.19088381	0.15645333	87.22302	X1 X6
2	0.15398018	0.11797933	93.20807	X1 X4
2	0.13815687	0.10148270	95.77431	X3 X4
2	0.12930170	0.09225071	97.21045	X4 X6
2	0.11967410	0.08221342	98.77186	X2 X6
2	0.11841105	0.08089662	98.97670	X2 X3
2	0.10763636	0.06966344	100.72415	X1 X2
2	0.01528887	-.02661374	115.70112	X2 X4
3	0.62636653	0.60199913	18.59613	X3 X5 X6
3	0.56792149	0.53974246	28.07479	X1 X5 X6
3	0.56329483	0.53481406	28.82514	X1 X3 X5
3	0.48234947	0.44858965	41.95292	X4 X5 X6
3	0.48203204	0.44825152	42.00440	X2 X5 X6
3	0.46212487	0.42704605	45.23296	X3 X4 X5
3	0.45735907	0.42196945	46.00588	X2 X3 X5
3	0.44434757	0.40810936	48.11609	X1 X4 X5
3	0.42318957	0.38557150	51.54751	X1 X2 X5

3	0.30754690	0.26238691	70.30252	X2 X4 X5
3	0.29077513	0.24452134	73.02257	X1 X3 X6
3	0.27426178	0.22693103	75.70072	X1 X3 X4
3	0.24529862	0.19607896	80.39799	X3 X4 X6
3	0.24035900	0.19081720	81.19910	X1 X4 X6
3	0.23064564	0.18047036	82.77442	X2 X3 X6
3	0.21747562	0.16644142	84.91034	X1 X2 X3
3	0.20373367	0.15180326	87.13902	X1 X2 X6
3	0.15743298	0.10248296	94.64809	X1 X2 X4
3	0.14005900	0.08397589	97.46582	X2 X3 X4
3	0.13236346	0.07577847	98.71389	X2 X4 X6
4	0.70491389	0.67868402	7.85727	X1 X3 X5 X6
4	0.63034129	0.59748274	19.95150	X3 X4 X5 X6

MINIMUM THICKNESS OF ROOF BEAM 3

Number in Model	R-square	Adjusted R-square	C(p)	Variables in Model
4	0.62881043	0.59581580	20.19978	X2 X3 X5 X6
4	0.59573227	0.55979736	25.56442	X1 X3 X4 X5
4	0.58450356	0.54757054	27.38550	X1 X4 X5 X6
4	0.57189109	0.53383696	29.43100	X1 X2 X5 X6
4	0.56692921	0.52843403	30.23572	X1 X2 X3 X5
4	0.48271750	0.43673683	43.89323	X2 X4 X5 X6
4	0.46214167	0.41433205	47.23023	X2 X3 X4 X5
4	0.44469222	0.39533152	50.06020	X1 X2 X4 X5
4	0.35458007	0.29720941	64.67465	X1 X3 X4 X6
4	0.30922791	0.24782595	72.02989	X1 X2 X3 X6
4	0.28067786	0.21673812	76.66016	X1 X2 X3 X4
4	0.25101513	0.18443870	81.47088	X2 X3 X4 X6
4	0.24753373	0.18064784	82.03549	X1 X2 X4 X6
5	0.73016040	0.69949681	5.76277	X1 X3 X4 X5 X6
5	0.71242532	0.67974638	8.63906	X1 X2 X3 X5 X6
5	0.63197277	0.59015149	21.68691	X2 X3 X4 X5 X6
5	0.59735679	0.55160188	27.30095	X1 X2 X3 X4 X5
5	0.58678283	0.53982634	29.01584	X1 X2 X4 X5 X6
5	0.36554537	0.29344825	64.89629	X1 X2 X3 X4 X6
6	0.73486362	0.69786784	7.00000	X1 X2 X3 X4 X5 X6

MINIMUM THICKNESS OF ROOF BEAM 4

Forward Selection Procedure for Dependent Variable Y

Step 1 Variable X5 Entered R-square = 0.30588887 C(p) = 66.57141650

	DF	Sum of Squares	Mean Square	F	Prob>F
Regression	1	19.07332649	19.07332649	21.15	0.0001
Error	48	43.28045001	0.90167604		
Total	49	62.35377650			

Variable	Parameter Estimate	Standard Error	Type II Sum of Squares	F	Prob>F
INTERCEP	3.76992321	0.21007813	290.37141378	322.04	0.0001
X5	-0.00203256	0.00044193	19.07332649	21.15	0.0001

Bounds on condition number: 1, 1

Step 2 Variable X6 Entered R-square = 0.48147743 C(p) = 40.09434401

	DF	Sum of Squares	Mean Square	F	Prob>F
Regression	2	30.02193627	15.01096814	21.82	0.0001
Error	47	32.33184023	0.68791149		
Total	49	62.35377650			

Variable	Parameter Estimate	Standard Error	Type II Sum of Squares	F	Prob>F
INTERCEP	5.46407245	0.46260540	95.97189413	139.51	0.0001
X5	-0.00224978	0.00038983	22.91194836	33.31	0.0001
X6	-0.04490390	0.01125566	10.94860979	15.92	0.0002

Bounds on condition number: 1.019896, 4.079583

MINIMUM THICKNESS OF ROOF BEAM 5

Step 3 Variable X3 Entered R-square = 0.62636653 C(p) = 18.59613104

	DF	Sum of Squares	Mean Square	F	Prob>F
Regression	3	39.05631874	13.01877291	25.71	0.0001
Error	46	23.29745776	0.50646647		
Total	49	62.35377650			

Variable	Parameter Estimate	Standard Error	Type II Sum of Squares	F	Prob>F
INTERCEP	4.25147624	0.48988523	38.14534296	75.32	0.0001
X3	0.00155224	0.00036752	9.03438247	17.84	0.0001
X5	-0.00237287	0.00033576	25.29566344	49.95	0.0001
X6	-0.04409067	0.00965976	10.55144182	20.83	0.0001

Bounds on condition number: 1.027639, 9.169749

Step 4 Variable X1 Entered R-square = 0.70491389 C(p) = 7.85726611

	DF	Sum of Squares	Mean Square	F	Prob>F
Regression	4	43.95404344	10.98851086	26.87	0.0001
Error	45	18.39973306	0.40888296		
Total	49	62.35377650			

Variable	Parameter Estimate	Standard Error	Type II Sum of Squares	F	Prob>F
INTERCEP	4.83648150	0.47150696	43.02113395	105.22	0.0001
X1	-0.00000104	0.00000030	4.89772470	11.98	0.0012
X3	0.00151036	0.00033045	8.54199378	20.89	

```
0.0001
X5           -0.00239818      0.00030177      25.82311589      63.16
0.0001
X6           -0.04060564      0.00873763       8.83048339      21.60
0.0001
                     MINIMUM THICKNESS OF ROOF BEAM                    6

Bounds on condition number:     1.034034,     16.35685
------------------------------------------------------------------------

Step 5   Variable X4 Entered        R-square = 0.73016040   C(p) =
  5.76277040

                DF         Sum of Squares      Mean Square          F
Prob>F

Regression       5            45.52825848       9.10565170      23.81
0.0001
Error           44            16.82551802       0.38239814
Total           49            62.35377650

                Parameter        Standard          Type II
Variable         Estimate           Error   Sum of Squares          F
Prob>F

INTERCEP       5.27194581      0.50396619      41.84608639     109.43
0.0001
X1            -0.00000123      0.00000031       6.22409872      16.28
0.0002
X3             0.00156240      0.00032059       9.08225407      23.75
0.0001
X4            -0.00013431      0.00006620       1.57421504       4.12
0.0485
X5            -0.00230937      0.00029510      23.41885230      61.24
0.0001
X6            -0.03962585      0.00846370       8.38210189      21.92
0.0001

Bounds on condition number:     1.145295,     26.89152
------------------------------------------------------------------------

No other variable met the 0.2500 significance level for entry into the
model.

 Summary of Forward Selection Procedure for Dependent Variable Y

        Variable   Number   Partial     Model
Step    Entered        In      R**2      R**2        C(p)           F
Prob>F

   1    X5              1    0.3059    0.3059     66.5714     21.1532
0.0001
   2    X6              2    0.1756    0.4815     40.0943     15.9157
0.0002
   3    X3              3    0.1449    0.6264     18.5961     17.8381
0.0001
   4    X1              4    0.0785    0.7049      7.8573     11.9783
0.0012
   5    X4              5    0.0252    0.7302      5.7628      4.1167
                     MINIMUM THICKNESS OF ROOF BEAM                    7

0.0485
                     MINIMUM THICKNESS OF ROOF BEAM                    8

     Backward Elimination Procedure for Dependent Variable Y

Step 0    All Variables Entered     R-square = 0.73486362   C(p) =
  7.00000000

                DF         Sum of Squares      Mean Square          F
Prob>F
```

```
Regression       6           45.82152161       7.63692027      19.86
0.0001
Error           43           16.53225489       0.38447104
Total           49           62.35377650

                Parameter        Standard          Type II
Variable         Estimate           Error   Sum of Squares          F
Prob>F

INTERCEP       5.05369104      0.56374541      30.89690525      80.36
0.0001
X1            -0.00000126      0.00000031       6.41563293      16.69
0.0002
X2             0.77233487      0.88431854       0.29326313       0.76
0.3873
X3             0.00157768      0.00032194       9.23339596      24.02
0.0001
X4            -0.00012749      0.00006683       1.39911227       3.64
0.0631
X5            -0.00229405      0.00029642      23.02838761      59.90
0.0001
X6            -0.04019504      0.00851159       8.57406980      22.30
0.0001

Bounds on condition number:     1.161135,     38.81763
------------------------------------------------------------------------

Step 1   Variable X2 Removed        R-square = 0.73016040   C(p) =
  5.76277040

                DF         Sum of Squares      Mean Square          F
Prob>F

Regression       5           45.52825848       9.10565170      23.81
0.0001
Error           44           16.82551802       0.38239814
Total           49           62.35377650

                Parameter        Standard          Type II
Variable         Estimate           Error   Sum of Squares          F
Prob>F

INTERCEP       5.27194581      0.50396619      41.84608639     109.43
0.0001
```



```
X1            -0.00000123      0.00000031       6.22409872      16.28
0.0002
X3             0.00156240      0.00032059       9.08225407      23.75
0.0001
X4            -0.00013431      0.00006620       1.57421504       4.12
0.0485
X5            -0.00230937      0.00029510      23.41885230      61.24
0.0001
X6            -0.03962585      0.00846370       8.38210189      21.92
0.0001

Bounds on condition number:     1.145295,     26.89152
------------------------------------------------------------------------

All variables left in the model are significant at the 0.2500 level.

Summary of Backward Elimination Procedure for Dependent Variable Y


        Variable   Number   Partial     Model
Step    Removed        In      R**2      R**2        C(p)           F
Prob>F

   1    X2              5    0.0047    0.7302      5.7628      0.7628
```

```
0.3873
                     MINIMUM THICKNESS OF ROOF BEAM                    10

           Stepwise Procedure for Dependent Variable Y

Step 1   Variable X5 Entered        R-square = 0.30588887   C(p) =
 66.57141650

                DF         Sum of Squares      Mean Square          F
Prob>F

Regression       1            19.07332649      19.07332649      21.15
0.0001
Error           48            43.28045001       0.90167604
Total           49            62.35377650

                Parameter        Standard          Type II
Variable         Estimate           Error   Sum of Squares          F
Prob>F

INTERCEP       3.76992321      0.21007813     290.37141378     322.04
0.0001
X5            -0.00203256      0.00044193      19.07332649      21.15
0.0001

Bounds on condition number:            1,            1
------------------------------------------------------------------------

Step 2   Variable X6 Entered        R-square = 0.48147743   C(p) =
 40.09434401

                DF         Sum of Squares      Mean Square          F
Prob>F

Regression       2            30.02193627      15.01096814      21.82
0.0001
Error           47            32.33184023       0.68791149
Total           49            62.35377650

                Parameter        Standard          Type II
Variable         Estimate           Error   Sum of Squares          F
Prob>F

INTERCEP       5.46407245      0.46260540      95.97189413     139.51
0.0001
X5            -0.00224978      0.00038983      22.91194836      33.31
0.0001
X6            -0.04490390      0.01125566      10.94860979      15.92
0.0002

Bounds on condition number:     1.019896,     4.079583
------------------------------------------------------------------------
                     MINIMUM THICKNESS OF ROOF BEAM                    11

Step 3   Variable X3 Entered        R-square = 0.62636653   C(p) =
 18.59613104

                DF         Sum of Squares      Mean Square          F
Prob>F

Regression       3            39.05631874      13.01877291      25.71
0.0001
Error           46            23.29745776       0.50646647
Total           49            62.35377650

                Parameter        Standard          Type II
Variable         Estimate           Error   Sum of Squares          F
Prob>F

INTERCEP       4.25147624      0.48988523      38.14534296      75.32
0.0001
```

```
X3             0.00155224      0.00036752       9.03438247      17.84
0.0001
X5            -0.00237287      0.00033576      25.29566344      49.95
0.0001
X6            -0.04409067      0.00965976      10.55144182      20.83
0.0001

Bounds on condition number:     1.027639,     9.169749
------------------------------------------------------------------------

Step 4   Variable X1 Entered        R-square = 0.70491389   C(p) =
  7.85726611

                DF         Sum of Squares      Mean Square          F
Prob>F

Regression       4            43.95404344      10.98851086      26.87
0.0001
Error           45            18.39973306       0.40888296
Total           49            62.35377650

                Parameter        Standard          Type II
Variable         Estimate           Error   Sum of Squares          F
Prob>F

INTERCEP       4.83648150      0.47150696      43.02113395     105.22
0.0001
X1            -0.00000104      0.00000030       4.89772470      11.98
0.0012
X3             0.00151036      0.00033045       8.54199378      20.89
0.0001
X5            -0.00239818      0.00030177      25.82311589      63.16
0.0001
X6            -0.04060564      0.00873763       8.83048339      21.60
0.0001
                     MINIMUM THICKNESS OF ROOF BEAM                    12

Bounds on condition number:     1.034034,     16.35685
------------------------------------------------------------------------

Step 5   Variable X4 Entered        R-square = 0.73016040   C(p) =
  5.76277040

                DF         Sum of Squares      Mean Square          F
Prob>F

Regression       5            45.52825848       9.10565170      23.81
0.0001
Error           44            16.82551802       0.38239814
Total           49            62.35377650

                Parameter        Standard          Type II
Variable         Estimate           Error   Sum of Squares          F
Prob>F

INTERCEP       5.27194581      0.50396619      41.84608639     109.43
0.0001
X1            -0.00000123      0.00000031       6.22409872      16.28
0.0002
X3             0.00156240      0.00032059       9.08225407      23.75
0.0001
X4            -0.00013431      0.00006620       1.57421504       4.12
0.0485
X5            -0.00230937      0.00029510      23.41885230      61.24
0.0001
X6            -0.03962585      0.00846370       8.38210189      21.92
0.0001

Bounds on condition number:     1.145295,     26.89152
------------------------------------------------------------------------
```

All variables left in the model are significant at the 0.2500 level.
No other variable met the 0.2500 significance level for entry into the model.

Summary of Stepwise Procedure for Dependent Variable Y

Step	Variable Entered	Variable Removed	Number In	Partial R**2	Model R**2	C(p)	F	Prob>F
1	X5		1	0.3059	0.3059	66.5714	21.1532	0.0001
2	X6		2	0.1756	0.4815	40.0943	15.9157	0.0002
3	X3		3	0.1449	0.6264	18.5961	17.8381	0.0001
4	X1		4	0.0785	0.7049	7.8573	11.9783	0.0012

Step	Variable Entered	Variable Removed	Number In	Partial R**2	Model R**2	C(p)	F	Prob>F
5	X4		5	0.0252	0.7302	5.7628	4.1167	0.0485

Model: MODEL6
Dependent Variable: Y

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Prob>F
Model	6	45.82152	7.63692	19.863	0.0001
Error	43	16.53225	0.38447		
C Total	49	62.35378			

Root MSE	0.62006	R-square	0.7349
Dep Mean	3.02690	Adj R-sq	0.6979
C.V.	20.48490		

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	T for H0: Parameter=0	Prob > |T|
INTERCEP	1	5.053691	0.56374541	8.964	0.0001
X1	1	-0.000001258	0.00000031	-4.085	0.0002
X2	1	0.772335	0.88431854	0.873	0.3873
X3	1	0.001578	0.00032194	4.901	0.0001
X4	1	-0.000127	0.00006683	-1.908	0.0631
X5	1	-0.002294	0.00029642	-7.739	0.0001
X6	1	-0.040195	0.00851159	-4.722	0.0001

Obs	Dep Var Y	Predict Value	Residual
1	3.4120	3.2621	0.1499
2	4.2980	3.7552	0.5428
3	1.9680	2.4443	-0.4763
4	0.0330	0.6959	-0.6629
5	1.0830	2.2339	-1.1509
6	3.5100	2.8369	0.6731
7	1.4760	2.2465	-0.7705
8	3.7400	3.7589	-0.0189
9	4.8880	3.5815	1.3065
10	2.0010	2.9341	-0.9331
11	2.5260	3.3586	-0.8326
12	0.0980	0.9905	-0.8925
13	4.4620	3.7748	0.6872

14	1.7060	2.5884	-0.8824
15	4.5280	4.1638	0.3642
16	2.6900	2.9070	-0.2170
17	2.7230	1.6214	1.1016
18	4.7900	4.5527	0.2373
19	2.5920	3.1950	-0.6030
20	4.0680	4.2343	-0.1663
21	3.0180	3.0821	-0.0641
22	2.6900	3.1522	-0.4622
23	1.9680	1.2777	0.6903
24	2.5920	2.0428	0.5492
25	2.4930	2.4227	0.0703
26	3.4450	2.9620	0.4830
27	2.5260	2.1681	0.3579
28	3.6420	3.8781	-0.2361
29	4.3310	4.1467	0.1843
30	2.9860	1.8664	1.1196
31	3.4120	3.8558	-0.4438
32	3.0840	2.4553	0.6287
33	2.5590	3.2646	-0.7056
34	2.9860	2.8067	0.1793
35	3.7070	3.6918	0.0152
36	4.6260	4.6092	0.0168
37	3.8060	4.3962	-0.5902
38	1.9030	2.0781	-0.1751
39	3.8390	4.0039	-0.1649
40	2.4280	2.6742	-0.2462
41	3.0840	2.8785	0.2055
42	2.3950	2.3121	0.0829
43	1.3780	1.4550	-0.0770
44	3.6420	3.1436	0.4984
45	2.7890	3.3278	-0.5388
46	3.8060	3.8163	-0.0103
47	4.6920	4.9344	-0.2424
48	3.1500	3.0948	0.0552
49	3.3140	2.3705	0.9435

Obs	Dep Var Y	Predict Value	Residual
50	4.4620	4.0410	0.4210

Sum of Residuals	0
Sum of Squared Residuals	16.5323
Predicted Resid SS (Press)	23.7128

VITA

JunLu Luo was born in January, 1966 in Jiangle County, Fujian Province, People's Republic of China. In July, 1987, he received his Bachelor of Engineering in Mining Engineering from the Department of Mining and Geological Engineering, Fuzhou University. He worked for the Tongluoping Colliery, Fujian, China for about half a year and then worked for eight years as mining engineer and assistant project manager for the Fujian Provincial Coal Mine Design Institute. In 1995, granted a research assistantship by the Department of Mining and Minerals Engineering, Virginia Polytechnic Institute and State University, he started his graduate studies in the United States. After completion of his degree of Master of Science in Mining Engineering in 1997, he continued to pursue a Ph.D. in the same area, which is expected to finish in June 1999.

JunLu Luo