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ABSTRACT

Roof bolting is the most popular method for underground openings in the mining industry, especially in the bedded deposits such as coal. In fact, all U.S. underground coal mine entries are roof-bolted as required by law.

However, roof falls still occur frequently in the roof bolted entries. The two possible reasons are: the lack of knowledge of and technology to detect the roof geological conditions in advance of mining, and lack of roof bolting design criteria for modern roof bolting systems.

This research is to develop a method for predicting the roof geology and stability condition in real time during roof bolting operation. Based on this information, roof bolting design criteria for modern roof bolting systems will be developed for implementation in real time.

For the prediction of roof geology and stability condition in real time, a micro processor was used and a program developed to monitor and record the drilling parameters of roof bolter. These parameters include feed pressure, feed flow (penetration rate), rotation pressure, rotation rate, vacuum pressure, oil temperature of hydraulic circuit, and signals for controlling machine. From the results of a series of laboratory and underground tests so far, feed pressure is found to be a good indicator for identifying the voids/fractures and estimating the roof rock strength. The method for determining quantitatively the location and the size of void/fracture and estimating the roof rock strength from the drilling parameters of roof bolter was developed. Also, a set of computational rules has been developed for in-mine roof using measured roof drilling parameters and implemented in MRGIS (Mine Roof Geology Information System), a software package developed to allow mine engineers to make use of the large amount of roof drilling parameters for predicting roof geology properties automatically.

For the development of roof bolting criteria, finite element models were developed for tensioned and fully grouted bolting designs. Numerical simulations were performed to investigate the mechanisms of modern roof bolting systems including both the tension and fully grouted bolts. Parameters to be studied are: bolt length, bolt spacing, bolt size/strength, grout annulus, in-situ stress condition, overburden depth, and roof geology (massive strata, fractured, and laminated or thinly-bedded). Based on the analysis of the mechanisms of both bolting systems and failure modes of the bolted strata, roof bolting design criteria and programs for modern roof bolting systems were developed. These criterion and/or programs were combined with the MRGIS for use in conjunction with roof bolt installation.
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<td>( \theta )</td>
<td>Angle Between the Horizontal Axis and Major Principal Stress</td>
</tr>
<tr>
<td>D</td>
<td>Bolt Diameter</td>
</tr>
<tr>
<td>L</td>
<td>Bolt Length</td>
</tr>
<tr>
<td>T</td>
<td>Bolt Pre-tension</td>
</tr>
<tr>
<td>r</td>
<td>Bolt Row Spacing</td>
</tr>
<tr>
<td>BSHL_SF</td>
<td>Bolt Shear Load Safety Factor</td>
</tr>
<tr>
<td>S</td>
<td>Bolt Spacing</td>
</tr>
<tr>
<td>( E_C )</td>
<td>Coal’s Young’s Modulus</td>
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<tr>
<td>( \mu )</td>
<td>Coefficient of Friction on the Bedding Plane</td>
</tr>
<tr>
<td>C</td>
<td>Cohesion of the Material</td>
</tr>
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<td>( C_b )</td>
<td>Cohesion of the Bedding Plane</td>
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<td>( t_n )</td>
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</tr>
<tr>
<td>w</td>
<td>Entry Width</td>
</tr>
<tr>
<td>FP(_C)</td>
<td>Feed Pressure for Compensation Run</td>
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<tr>
<td>FP(_{DR})</td>
<td>Feed Pressure When Drilling in Rock</td>
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<tr>
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<td>F(_{RS})</td>
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<td>( \sigma_h )</td>
<td>Horizontal Stress</td>
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<td>( h_r )</td>
<td>Horizontal-to-vertical Stress Ratio</td>
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<tr>
<td>( E_0 )</td>
<td>Initial Young’s Modulus</td>
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<td>ISHS_SF</td>
<td>Interface Shear Stress Safety Factor</td>
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\( \varphi \) Internal Friction Angle of the Material

\( \sigma_{x,\text{max}} \) Maximum Horizontal Stress Across the Entry at the Entry Corner

\( \tau_{xy,\text{max}} \) Maximum Shear Stress Across the Entry at the Entry Corner

BP\text{\scriptsize \( _{\text{net}} \)} Net Bit Position

\( \sigma_b \) Normal Stress on the Bedding Plane

F\text{\scriptsize \( _{L} \)} Oil Leakage Factor

T Oil Temperature

H Overburden Depth

\( \varphi_p \) Peak Friction Angle

\( \tau_p \) Peak Shear Stress

\( \nu \) Poisson’s Ratio

\( \sigma_1, \sigma_2, \sigma_3 \) Principal Stresses

RD Roof Deflection

\( s \) Roof Displacement at the Center of the Entry

E\text{\scriptsize \( _{L} \)} Roof Layer’s Young’s Modulus

RSF Roof Safety Factor

RWPS Roof Weighted Plastic Strain

\( \tau \) Shear Stress

\( \tau_b \) Shear Stress on the Bedding Plane

E\text{\scriptsize \( _{t} \)} Tangent Young’s Modulus

E\text{\scriptsize \( _{50} \)} Tangent Young’s Modulus at the 50 % Stress Level of the Maximum Stress

\( W_l \) The Left Side Width of Feed Pressure Valley

R The Resistance between Drilling Bit Tip and the Free Face of the Voids When Feed Pressure Begins to Drop

F The Size of the Void
<table>
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<td>$t_1$</td>
<td>The Time When Drilling Starts</td>
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<tr>
<td>$h$</td>
<td>Thickness of the First Layer</td>
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<td>Time</td>
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<tr>
<td>$\sigma_c$</td>
<td>Uniaxial Compressive Strength</td>
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<tr>
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<td>Unit Weight</td>
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CHAPTER 1

EXECUTIVE SUMMARY

The objectives of this research are: (1) to develop a method for identifying the geological features in the mine roof strata based on the drilling parameters obtained during normal roof bolting operation, and (2) to develop roof bolting design criteria for modern roof bolting system.

1.1 Prediction of Roof Geology and Stability

From the results of a series of laboratory and underground tests, feed pressure is found to be a good indicator for identifying the voids/fractures and estimating the roof rock strength when both penetration rate and rotation rate were controlled or kept constant. Methods for determining quantitatively the location and the size of void/fracture and estimating the roof rock strength from the drilling parameters of roof bolter have been developed. In addition, characteristics of the machine, sensors, and drilling control algorithm, etc were also studied as to eliminate these effects from drilling data as much as possible. The following conclusions can be made:

Void/Fracture Prediction

- The feed pressure trends of dropping to the level of drilling in the air when a void / fracture in rock is encountered can be used to detect the voids/fractures. This is the main criteria of void/fracture prediction. In addition, in order to enhance the prediction accuracy, a supplemental prediction criterion is developed considering the shape of feed pressure valley.
- From the results of lab and field tests, the prediction results show that a very high prediction percentage have been achieved for the 1/8-in or larger voids. But, a void/fracture of 1/16-in or smaller are difficult to predict by the system developed.
- The width of plateau at the bottom of feed pressure valley is much closer to the size of the void/fracture. However, the accuracy of predicting the void / fracture size using drilling parameters measured by the current data collecting system is too low to be acceptable for voids/fractures smaller than 1/2-in.

Estimation of Roof Rock Strength

- Feed pressure is the most sensitive and reliable parameters when the strength of roof rock changes under the current system. In order to eliminate the machine effect, the net feed pressure is recommended for estimating rock strength instead of feed pressure.
- Both penetration rate and rotation rate have obvious impact on the magnitude of feed pressure.
- The strength of roof rock can be determined / classified based on the magnitude of feed pressure because it takes both the effects of penetration rate and rotation rate into account.
Development of Data Visualization and Database Software (MRGIS)

- A new software package, MRGIS (Mine Roof Geology Information System), has been developed to allow mine engineers to make use of the large amount of roof drilling parameters for roof support design. MRGIS consists of four modules: data importing and cleaning, data management, data interpolation, and data visualization.

1.2 Bolting Design Criteria

Numerical simulations were performed to investigate the mechanisms of modern roof bolting systems: tension and fully grouted bolts. Parameters to be studied are: bolt size/strength, bolt length, bolt spacing, grout annulus and length, in-situ horizontal stress, overburden, and roof geology (massive strata, fractured, laminated or thinly-bedded, locations of bedding planes, and strata sequence from the strength points of view). From the results of those analyses, the following roof bolting criteria has been developed:

Tensioned Roof Bolting

- Based on the analysis of the mechanisms of tensioned bolts and the failure modes of tensioned bolted strata, a design procedure for tensioned bolting was proposed. The model can be used for tensioned bolting design, i.e. to determine the bolt length, optimum pre-tension, bolt diameter and bolt spacing.
- According to bedding plane location, the roof was classified into 4 types. Based on the geological condition of Pittsburgh seam, some design guidelines were developed.
- Computer program was developed for tensioned bolting design using the data from numerical modeling. The results such as yield zone, roof deformation, bolt load increase, and the stresses around the entry from numerical modeling are built into a database. The program first gets the geological inputs from users, searches the design information in the database, performs the analysis using the design criteria, and finally displays the design by 2-D/3-D views.

Fully Grouted Bolting

- Different failure modes of the fully grouted bolts were defined.
- In order to evaluate the bolt and roof stability, bolt ad roof stability measures were presented.
- According to the plastic distribution for different geological conditions, four modes of failure were discussed. The immediate roof was classified into four types based on different strata sequences and different modes of failure.
- Based on the results of numerical analysis, four regression equations were developed for each type of roof to estimate the roof and bolt stability.
- The estimated roof and bolt stability measures are compared with corresponding critical values and changes in bolting parameters made, if appropriate. Then the roof and bolt stability measures are reevaluated with the new values of bolt length and diameter. Finally a safe roof and bolt design can be obtained.
CHAPTER 2

INTRODUCTION

Undoubtedly, recent coal mining is being conducted in more geologically disadvantaged coal reserves where the roof is either weak or contains geological anomalies with rapid changes of geological features such as rock type, slickensides, voids/fractures, etc. Thus the underground mine roof is more difficult to predict and control. Since all underground coal mine entries are supported by roof bolts and the design and selection of roof bolts are based on the knowledge of roof geology, it is of utmost importance that the roof geology and its variation over the immediate operation areas be known in advance so that a proper roof bolting system can be designed and/or selected.

The current method of using surface borehole loggings, which normally spaced more than 1,000 ft apart, to determine the immediate roof is awfully inadequate. Roof falls that caused injuries/fatalities and/or production delays are mostly localized even though some massive roof falls have been reported. For localized roof falls, the major reason is change in geology. Obviously a selected roof bolting system must match a certain geological features (rock type and stratigraphic sequence). But when this geological feature change and differ considerably from the original one, the selected roof bolting system may not work and roof falls occur.

How can a roof control engineer know the geological features change at certain location? In fact in order to prevent roof falls the roof geology within the bolted horizon must be known in advance from bolt row to bolt row. Only with this knowledge, a roof control engineer can determine if a change in the current roof bolting system is needed. In order to achieve this objective, a detailed roof geology map depicting geological changes from bolt row to bolt row must be available. In this respect, if a roof bolter’s drilling parameters can be monitored and correlated with the geological features, all changes in geological features can be mapped from bolthole to bolthole when the bolt holes are being drilled for roof bolts installation and the roof bolting system will stay compatible with the roof geological features. This system is called MRGIS (Mine Roof Geology Information System).

In this project a patented drill control unit (DCU) installed in the J.H. Fletcher & Co.’s roof bolter was used to record the drilling parameter for experiments conducted in mines and laboratory. Finally, more than 1,000 roof bolt holes have been drilled in and drilling parameters recorded from 10 concrete and simulated blocks in the laboratory and 8 underground mines\(^{(1)}\) - \(^{(4)}\).

As mentioned above, roof falls still occur frequently in the roof bolted entries. This is due to not only the lack of information of roof geology but also the lack of roof bolting design criteria for modern roof bolting system. Therefore, another task of this project is to develop roof bolting design criteria for modern roof bolting systems.
Roof bolting is a common practice in supporting entries in underground coal mines. U.S. coal mines have been consuming approximately 100 million pieces of roof bolts annually for the past twenty years (5). Six types of roof bolts are currently used in the U.S. coal industry for reinforcing the entries in underground coal mines. However, in terms of the mechanisms employed to reinforce the roof strata, there are only two basic types: tensioned bolt and fully grouted resin bolt. Tensioned bolt is primarily mechanically anchored, but in order to increase the anchorage capacity and installed bolt pre-tension, various variations of tensioned bolts have been developed including torque tension bolt, mechanically anchored resin-assisted bolt, combination bolt and resin-anchored cable bolt. It was estimated that 20% of bolts used in U.S. underground coal mines are tensioned bolt and another 80% are resin bolt (6).

The tensioned bolt is considered an active support system that can create a compression zone in the roof between the anchor and the bearing plate immediately after installation. Whereas the fully grouted bolt is generally considered a passive support system which means that its load is generated due to the movements of surrounding strata (1-7). Tensioned bolt and fully grouted resin bolt are believed to work in different mechanisms. In order to develop the criterion for roof bolting design, it is necessary to understand the mechanisms of tensioned and fully grouted bolts and analyze the factors that affect the stability of the bolted roof strata.

Several researches have done works on the mechanisms of both roof bolting systems and support designs using various approaches such as physical modeling, analytical method, field measurement, statistical method, and numerical modeling (7)-(11). The results of these researches help to certain extent explain how the tensioned bolts work, but difficulties exist in incorporating these mechanisms into practical bolting design because of the complicated interactions between the bolts and host rock, as well as complex roof behavior. As a result, roof bolting design is also still largely based on practical experience.

The advance of numerical software makes it possible to model the interaction between the bolts and roof, the large plastic deformation of the roof strata usually encountered underground, and the bedding planes which is important in controlling roof stability. In order to quantitatively describe the interaction between the bolts and the rock, as well as the effect of bedding plane, in-situ horizontal stress, excavation sequence and pre-tension, hole roughness, finite element models using ABAQUAS (12) were used. Based on the numerical modeling, the mechanisms of tensioned and fully grouted bolting systems, potential failure modes of the bolted strata and considerations in the design of both roof bolting systems are analyzed. From this analysis a design approach for tensioned and fully grouted roof bolting was proposed.

Finally, a roof bolting criterion or programs were combined with the MRGIS for use in conjunction with roof bolt installation.
CHAPTER 3

DRILLING AND DATA COLLECTING SYSTEM

3.1 Drill System

The research was conducted using a series of laboratory and underground tests for developing the theoretical methodology for roof geology prediction. All laboratory and underground tests were conducted using a J.H. Fletcher & Co.’s model HDDR walk-thru type dual head roof bolter as shown in Figure 3.1. The drilling system consists of a set of sensors and a drill control unit (DCU) installed on the machine (Figure 3.2). One side of the machine has standard hydraulic controls while the other side is fitted with the patented Fletcher Feedback Control System (13). This system allows the operator to preset the penetration rate, rotation rate, and the maximum feed pressure (= thrust cap) of the machine. Once the parameters are set, the machine drills without additional operator input. A data logger allows drilling data to be monitored and analyzed. Note the roof bolter is a mast feed type consisting of two stages: carriage and mast.

Figure 3.1 J.H. Fletcher & Co.’s HDDR dual head roof bolter
3.2 Drilling Algorithm (WVU Control Mode)

The drilling algorithms used in this project allow us to drill at both a pre-set penetration rate and a pre-set rotation rate. This control mode was designed for this project and called “WVU Control Mode”. Using this mode, the effect of different drilling setting (penetration rate and rotation rate) on drilling parameters and drilling conditions can be determined. In other words, this control mode helps us develop the methodology for predicting roof geology that work at any penetration rate and rotation rate and determine the optimum drilling setting.

When the WVU control mode is set on, the penetration and rotation rates are controlled separately by the closed loop feedback system. When a specific penetration rate is keyed in before drilling, this value becomes the drill feed up target. The system analyzes only the current velocity and adjusts the electrical solenoid either up or down to achieve the preset target penetration rate. It does this without considering what other drilling parameters are. To drill the same velocity, the feed pressure will have to be higher while drilling through a harder material than a softer material. Likewise, the preset rotation rate is controlled based on the current rpm. System does this without considering other drilling parameters. At a constant drill penetration and rotation rates, the feed and rotation pressures should be higher when drilling through a harder material.

Penetration rate and rotation rate are controlled independently by adjusting two 4-way valves through pulse width modulation (PWM) control.

In a series of underground tests, the feedback control mode in which drilling is controlled based on both horsepower curve and bite curves selected was also tested.
3.3 Data Collection System

The drilling parameters collecting system was originally designed for controlling roof bolter automatically so that overall drilling and bolting consistency can be improved. Drilling parameters are recorded every 100 milliseconds so a 54-in long hole will have 250 to 850 records, depending on the penetration rate and the condition of roof geology.

The drilling parameters collecting system is designed to collect 17 (Dec. 2000 - Aug. 2003) or 15 (May, 2004 - present) drilling parameters as shown in Table 3.1. The feed pressure measures the hydraulic pressure inside the cylinders applying the axial load. The rotation pressure records the hydraulic pressure in the hydraulic motor that provides rotational force. RPM-counts is measured using an electronic tachometer attached directly to the drill mast and can be converted into rotational rate.

These drilling parameters are collected in terms of sensor output in voltages and then converted to dimensionless numbers ranging from 0 to 4095 for feed and rotation flow, and from 0 to 255 for others since the resolutions of A/D converter are 12 bits and 8 bits, respectively. Note before the underground test on May 2004, channel 3 and 5 (feed and rotation flows) are the upper 8 bits of the measurement of the feed/rotation flow transducers and the lower 4 bits of them are put into channel 14 and 15 as feed/rot flow LSB’s. So, the data recorded before and after 2004 have 17 and 15 columns (channels), respectively. These dimensionless numbers are referred to as machine data and can be converted to engineering units based on the conversion factors if necessary. In the current experimental set up, all drilling parameters are collected and stored by a notebook PC.

<table>
<thead>
<tr>
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<td>Data 12</td>
<td>Spare Channel</td>
</tr>
<tr>
<td>Data 13</td>
<td>Message Counter</td>
</tr>
<tr>
<td>Data 14*</td>
<td>Feed Flow LSB’s*</td>
</tr>
<tr>
<td>Data 15*</td>
<td>Rot Flow LSB’s*</td>
</tr>
<tr>
<td>Data 16</td>
<td>Feed PWM CMD</td>
</tr>
<tr>
<td>Data 17</td>
<td>Flow PWM CMD</td>
</tr>
</tbody>
</table>

Table 3.1 Drilling parameters


(b) May 2004 - Present

<table>
<thead>
<tr>
<th>Column</th>
<th>Drilling Parameter</th>
</tr>
</thead>
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<tr>
<td>Data 1</td>
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<td>Feed Pressure</td>
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<td>Feed Flow</td>
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<td>Data 4</td>
<td>Rotation Pressure</td>
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<td>Data 5</td>
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<td>Rotation Torque</td>
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<td>Data 9</td>
<td>Carriage Position</td>
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<td>Data 10</td>
<td>Vacuum</td>
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<tr>
<td>Data 11</td>
<td>Temperature</td>
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<tr>
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<td>Data 13</td>
<td>Message Counter</td>
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<tr>
<td>Data 14*</td>
<td>Feed PWM CMD</td>
</tr>
<tr>
<td>Data 15*</td>
<td>Flow PWM CMD</td>
</tr>
</tbody>
</table>
into ASCII files. The ASCII file containing machine data is referred to as machine data file (Figure 3.3). A machine data file has 17 or 15 columns (channels) data, with different columns representing different drilling parameters. A software for displaying all the drilling parameters was developed as shown in Figure 3.4, so the drilling data and machine conditions including control condition can be checked at the test site immediately.
In this research, all drilling parameters except the data of two mast feed position sensors and rotation count are not converted from dimensionless machine data to engineering units for the following reasons:

- The intention of this research is to develop a set of computational rules for predicting roof geology based on the collected drilling parameters and implement the rules in MRGIS. What the end users of MRGIS are interested in is the roof geological properties determined by using drilling parameters. If the rules for predicting roof geology are derived directly from machine data and implemented in MRGIS without conversation, a lot of computational time will be saved when displaying prediction results in MRGIS. This is especially meaningful when multiple holes are displayed at the same time.

- It is easier to plot and display all drilling parameters together for comparing their roles in reflecting the variation in roof geology properties.

- A lot of time is saved when analyzing drilling parameters.
CHAPTER 4

EXPERIMENT

In this project, a series of laboratory and underground tests were conducted to develop the theoretical methodology for predicting roof geology based on the drilling parameters of roof bolter. A patented drill control unit (DCU) installed in the J.H. Fletcher & Co.'s HDDR dual head roof bolter was used to control drilling and collect the drilling parameters for all experiments. More than 1,000 roof bolt holes have been drilled in, and drilling parameters recorded from 13 concrete and simulated blocks in the laboratory and 8 underground coal mines\(^{(1)-(4)}\).

As mentioned in Chapter 3, the drill control unit (DCU) allows the parameters of penetration rate, rotation rate and maximum feed pressure (thrust cap) to be set by the operators. Several drilling settings were tested in order to determine the effect of both penetration rate and rotation rate on drilling parameters and/or drilling conditions. In this series of tests, the role of thrust cap was only for safety. So, the thrust cap was basically set at the maximum value (\(= 1,000 \text{ psi}\)) in order to eliminate its effect on drilling test as much as possible. In some holes, thrust cap was set at lower level as to observe the behavior of the drilling parameters and the machine including the response of DCU when feed pressure rises beyond the level of thrust cap.

All the holes were drilled using a standard roof bolt drilling bit designed for underground coal mines (Kennametal’s Dust Hog design bit). Figure 4.1 shows the bits used for this series of tests. The bits are fairly inexpensive carbide insert drag bits that designed primarily for coal-bearing sequence rocks, and the sizes and designs were selected because of their wide-spread use in the underground coal mining industry. The majority of the holes were drilled using the 1-3/8 in diameter bits but several holes were drilled using the smaller size bit (1-1/32 in) which the mining industry uses for specialized bolting applications. A new bit was used for every hole so as to minimize the effect of bit wear on drilling parameters and conditions.

The compensation run, i.e. drilling in the air without steel rod and bit, was conducted before every drilling to check the impact of drilling settings and machine conditions on drilling parameters for running the machine itself. When comparing it with drilling data, the amount of feed pressure and rotation pressure consumed for drilling...
rock can be determined. It can also be distinguished whether the change of drilling parameters is caused by geology change or by the machine characteristics including control algorithm.

4.1 Laboratory Tests Using Manufactured Roof Rock Blocks

A series of manufactured rock blocks were constructed for the purpose of simulating various roof rock lithologies. 4 blocks were designed to simulate massive rock units, 6 blocks were designed to simulate a sequence of rock layers of various physical properties, three blocks were designed to simulate bedding separations of varying dimensions and orientations, and one block was constructed to simulate different sizes of fractures. All were designed for laboratory testing using different combinations of drilling parameters. Figure 4.2 shows the test scene of laboratory test.

Figure 4.2 Test scene of laboratory test (J.H. Fletcher & Co.’s facility, Huntington, WV)
Each of the rock layers and the concrete used for constructing the manufactured rock blocks were tested to determine the physical properties. Each type was tested to measure unconfined compressive strength, Brazilian tensile strength, shear strength, Young’s modulus and unit weight. Depending on the size of samples available, the number of tests per rock varied. The test procedures adhered to ASTM standards and were conducted on cores of 2- and 1-in in diameter. Basically, 2-in cores were used when available.

4.1.1 Manufactured Blocks

4.1.1.1 Solid Concrete Block

As to check the consistency of the drilling parameters with a single rock type and the impact of different strengths of roof rocks on drilling parameters, three solid concrete blocks were constructed (Figures 4.3 and 4.4). Their dimensions were approximately 3 ft × 4 ft × 5 ft. The blocks were constructed using the concretes with an unconfined compressive strength of 4,000 psi, 8,000 psi, and 12,000 psi, respectively (designed). Table 4.1 shows their physical properties.

<Drilling Settings>
- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 0.4, 0.6, 0.8, 1.1, 1.3, 1.5, 1.7, 1.9 and 2.1 in/sec
- Rotation rate: 300, 350, 400, 450, 500, 600, 650 rpm and free*
- Thrust cap: 1,000 psi
- Bit size: 1-1/32 and 1-3/8 in
- Number of drill holes: 150 holes
* Free means the maximum allowable value on the DCU.

![Figure 4.3 Schematic of solid concrete block](image1)

![Figure 4.4 Solid concrete block (UCS = 4,000 psi)](image2)
Table 4.1 Physical properties of solid concrete blocks

<table>
<thead>
<tr>
<th>Concrete Block (designed)</th>
<th>UCS (psi)</th>
<th>BTS (psi)</th>
<th>E$_{50}$ (× 10$^6$ psi)</th>
<th>Unit Weight (lbs/ft$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>The 4,000 psi block</td>
<td>5,710</td>
<td>391.6</td>
<td>1.643</td>
<td>137.6</td>
</tr>
<tr>
<td>The 8,000 psi block</td>
<td>6,070</td>
<td>492.7</td>
<td>1.550</td>
<td>137.4</td>
</tr>
<tr>
<td>The 12,000 psi block</td>
<td>12,340</td>
<td>430.0</td>
<td>2.070</td>
<td>159.0</td>
</tr>
</tbody>
</table>

Where, UCS = standardized unconfined compressive strength, BTS = Brazilian tensile strength, E$_{50}$ = tangent Young’s modulus at the 50% stress level of the maximum stress.

4.1.1.2 Layered Block

Six different layered blocks were designed and constructed using a variety of quarried rocks that were embedded in poured concrete structures. The schematics of the blocks are shown in Figures 4.5 - 4.10. Their dimensions were 2 ft × 2 ft × 6-1/2 ft and each had four separate rock layers embedded. The rock units were cut to a finished size as indicated in the figures and in general were approximately 1 ft × 1 ft × 1 ft. The rocks were secured in place using steel banding attached to 3/4-in rebar, then encased with a standard concrete mix. Including the concrete that was poured to encase the rocks, each block had nine different layers (four rock layers and five concrete layers). The drilling tests were conducted along the long axis of the blocks which provided a maximum drilling length of 6-1/2 ft.

Figure 4.5 Schematic of layered block #1
Figure 4.6  Schematic of layered block #2

Figure 4.7  Schematic of layered block #3
Figure 4.8 Schematic of layered block #4

Figure 4.9 Schematic of layered block #5
The physical properties of the rock units, including the concrete used in the manufactured roof blocks are shown in Table 4.2. The average unconfined compressive strengths of the rocks varied from about 7,000 to 27,000 psi. The three kinds of sandstone, designated red, brown and light brown, were very consistent with no laminations or apparent bedding planes. All three sandstones are fine to medium-grained. The white marble is vuggy with voids and discontinuities throughout. The argillite, weakly metamorphosed shale, has zones of healed and open fractures and is generally

Table 4.2  Physical properties of the embedded rocks and the concrete used in the layered blocks

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>UCS (psi)</th>
<th>BTS (psi)</th>
<th>Shear Strength (psi)</th>
<th>$E_{50}$ ($\times 10^6$ psi)</th>
<th>Unit Weight (lbs/ft$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Sandstone</td>
<td>6,986</td>
<td>1,053</td>
<td>N/A</td>
<td>2.77</td>
<td>149</td>
</tr>
<tr>
<td>Brown Sandstone</td>
<td>17,104</td>
<td>930</td>
<td>N/A</td>
<td>1.94</td>
<td>160</td>
</tr>
<tr>
<td>Lt Brown Sandstone</td>
<td>27,359</td>
<td>1,934</td>
<td>2,846</td>
<td>2.34</td>
<td>155</td>
</tr>
<tr>
<td>White Marble</td>
<td>17,418</td>
<td>1,371</td>
<td>1,010</td>
<td>2.48</td>
<td>166</td>
</tr>
<tr>
<td>Argillite</td>
<td>20,473</td>
<td>1,044</td>
<td>1,395</td>
<td>4.24</td>
<td>168</td>
</tr>
<tr>
<td>Soft Concrete</td>
<td>2,864</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Embedding Concrete</td>
<td>4,021</td>
<td>474</td>
<td>N/A</td>
<td>0.65</td>
<td>132</td>
</tr>
<tr>
<td>High-strength Concrete</td>
<td>12,340</td>
<td>430</td>
<td>N/A</td>
<td>2.07</td>
<td>159</td>
</tr>
</tbody>
</table>
discontinuous. The high-strength concrete that was embedded as separate layers in layered blocks #2-#4 had a compressive strength less than 3,000 psi. While initially intended to be a high-strength concrete, these layers are the lowest strength units in the manufactured blocks. Samples of this concrete layer crumbled easily when handled probably due to an improper mix. The concrete used for embedding the rock layers had a compressive strength of 4,020 psi. Overall, the rock units and concrete layers in provided a series of drilling encounters with transition zones from weak-to-strong and strong-to-weak rocks. The drilling settings and number of drill holes for each layered block were as follows;

- **Layered Block #1**
  <Drilling Settings>
  - DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
  - Penetration rate: 0.6 and 1.1 in/sec
  - Rotation rate: 150, 200, 300, 400 and 500 rpm
  - Thrust cap: 1,000 psi
  - Bit size: 1-1/32 and 1-3/8 in
  - Number of drill holes: 8 holes

- **Layered Block #2**
  <Drilling Settings>
  - DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
  - Penetration rate: 0.6, 1.1 and 1.5 in/sec
  - Rotation rate: 150, 200, 300, 400 and 500 rpm
  - Thrust cap: 1,000 psi
  - Bit size: 1-1/32 and 1-3/8 in
  - Number of drill holes: 11 holes

- **Layered Block #3**
  <Drilling Settings>
  - DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
  - Penetration rate: 0.6, 1.1 and 1.5 in/sec
  - Rotation rate: 150, 200, 300, 400, 500 rpm and free
  - Thrust cap: 1,000 psi
  - Bit size: 1-1/32 and 1-3/8 in
  - Number of drill holes: 10 holes

- **Layered Block #4**
  <Drilling Settings>
  - DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
  - Penetration rate: 1.1 and 1.5 in/sec
• Rotation rate: 300 rpm
• Thrust cap: 1,000 psi
• Bit size: 1-1/32 and 1-3/8 in
• Number of drill holes: 8 holes

Layered Block #5
<Drilling Settings>
• DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
• Penetration rate: 0.4, 0.6, 1.1, 1.5 in/sec and free
• Rotation rate: 150, 400, 500 rpm and free
• Thrust cap: 1,000 psi
• Bit size: 1-1/32 and 1-3/8 in
• Number of drill holes: 15 holes

Layered Block #6
<Drilling Settings>
• DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
• Penetration rate: 0.6, 1.1, 1.5 in/sec and free
• Rotation rate: 300, 400, 500 rpm and free
• Thrust cap: 750 and 1,000 psi
• Bit size: 1-1/32 and 1-3/8 in
• Number of drill holes: 19 holes

4.1.2.3 Fractured Block

This block was constructed to simulate small voids/fractures (Figures 4.11). This concrete block was also constructed of high strength concrete mix with an unconfined compressive strength of about 12,300 psi on average. The block for this test consists of four individual layers of concrete which were 15-in thick and constructed parallel faces. In between each concrete layers, a void was formed by inserting a narrow steel plate around the block perimeters as shown in Figures 4.12 – 4.14. The thickness of 3 steel plates were 1/16-, 1/8-, and 3/8-in, respectively to simulate voids/fractures with different sizes. The layers were bolted together and in a steel frame so the block would act as one single unit. The final block dimensions were approximately 3 ft × 4 ft × 5ft. The 1/16-, 1/8-, and 3/8-in voids were located at 15, 30 and 45 in, respectively.

<Drilling Settings>
• DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
• Penetration rate: 0.4, 0.8, 1.1 and 1.5 in/sec
• Rotation rate: 300, 400, 500 rpm and free
• Thrust cap: 600, 700, 850, and 1,000 psi
• Bit size: 1-1/32 and 1-3/8 in
• Number of drill holes: 37 holes
Figure 4.11 Schematic of fractured block

Figure 4.12 Assembling fractured block
4.1.2.4 Bedding Separations Block

Three blocks were designed and constructed to simulate bedding separations in rock layers. Different block designs were constructed to simulate different types of bedding separations that could be encountered in roof rock sequences. The designs included different sizes of horizontal bedding separations and angled separations. The drilling settings and number of drill holes for each layered block were as follows;

- **Large-size Bedding Separations Block**
  This block was constructed using embedded foam sheets to simulate large-size bedding separations (Figure 4.13). Its dimensions were 2 ft × 2 ft × 6-1/2 ft. These were four separation layers composed of embedded foam sheets. The foam layers were 1-, 2-, 3-, and 4-in in thickness, respectively. The concrete for embedding the foam layers was a standard mix with an unconfined compressive strength of 4,020 psi.
  
  <Drilling Settings>
  - DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
  - Penetration rate: 0.6 and 1.1 in/sec
  - Rotation rate: 150, 400 and 500 rpm
  - Thrust cap: 1,000 psi
  - Bit size: 1-1/32 and 1-3/8 in
  - Number of drill holes: 5 holes

![Figure 4.13 Schematic of manufactured block with large bedding separations](image-url)
Small-size Bedding Plane Separations Block

This block was constructed using horizontal cardboard inserts to simulate smaller bedding separations (Figure 4.14). This block had three separate cardboard layer inserts constructed of 1/8-in sheets of poster board. The thickness of cardboard layers were 1/8-, 1/4- and 3/8-in, respectively. The concern to embed the cardboard layers was a special high-strength mix with an unconfined compressive strength of about 12,000 psi. Its dimensions were 3 ft × 4 ft × 5 ft.

<Drilling Settings>
- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 0.6 and 1.1 in/sec
- Rotation rate: 300, 400 and 500 rpm
- Thrust cap: 1,000 psi
- Bit size: 1-3/8 in
- Number of drill holes: 5 holes

Angled Bedding Separations Block

This block was designed and constructed to simulate bedding separations that are intercepted by a drill bit at an angle as opposed to perpendicular (Figure 4.15). This block was constructed using the 1/8-in sheets of poster board embedded in the high strength concrete. The angled layers permitted a variety of drilling opportunities as the block was rotated so the layers could be intercepted at angles of 15, 30, 60 and 75 degree. Its dimensions were 3 ft × 4 ft × 5 ft.

Figure 4.14 Schematic of manufactured block to simulate with horizontal cardboard
<Drilling Settings>
- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 0.6, 1.1 and 1.5 in/sec
- Rotation rate: 200, 300, 400 and 500 rpm
- Thrust cap: 1,000 psi
- Bit size: 1-3/8 in
- Number of drill holes: 14 holes

Figure 4.15 Schematic of manufactured block with angled cardboard lavers

4.2 Underground Tests

4.2.1 Design of Underground Tests

In order to observe the behaviors of drilling parameters when drilling in different strengths of rocks and develop the criteria for estimating the strength of roof rock, underground tests were conducted in different coal mines. The criteria for void/fracture prediction developed from the results of laboratory tests were also checked and verified. In addition, bit wear tests were conducted so as to observe the impact of rock strength on the magnitude of bit wear and its impact on drilling parameters. Core sampling and borehole scoping were always conducted at each test site to verify the roof geology and determine the physical properties of each roof rock.
4.2.1.1 Core Sampling
A number of core samples were collected at each test site. These cores were used to determine the roof geology of their test area and to measure the physical properties of roof rocks. Cores were collected using a core barrel with diamond bit as shown in Figure 4.16. Water was used to flush cuttings from the core barrel and to cool the drill bit. The diameter of cores obtained from the barrel was 2 in.

4.2.1.2 Borehole Scoping
Although the cores can be used to define the roof geology of test sites, geology can often change place to place. Moreover, it is difficult to determine the actual size and condition of void/fracture. Thus, borehole scoping was also conducted at each test site to verify the roof geology. Figure 4.17 shows the borehole scope system. The probe of the borehole scope contains a camera, light and mirror system. The digital camcorder records the image data to a micro tape.

4.2.2 Overview of Underground Tests
The roof geology and drilling settings of each test site are mentioned as follows:

4.2.2.1 Mine A Underground Test
Mine A was located in western Colorado. The roof of this test site consisted of mostly shale and mudstone. There was one small layer of competent sandstone that was around 40 in deep from the roof line. Figure 4.18 shows the core log.
<Drilling Settings>
- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 0.4, 0.6, 1.1, 1.3 and 1.5 in/sec
- Rotation rate: 200, 300, 400 and 500 rpm
- Thrust cap: 1,000 psi
- Bit size: 1-1/32 and 1-3/8 in
- Number of drill holes: 34 holes

4.2.2.2 Mine B Underground Test
Mine B was located in eastern Kentucky. The roof of this mine consisted of approximately 20-30-in of shale overlain by sandstone member. Figure 4.19 shows the result of borehole scoping. However, as the core drilling apparatus was broken, the core sample could not be obtained.

<Drilling Settings>
- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 0.8, 1.1 in/sec and Free
- Rotation rate: 300, 400, 500 rpm and Free
- Thrust cap: 400, 650 and 800 psi
- Bit size: 1-1/32 and 1-3/8 in
- Number of drill holes: 53 holes

Figure 4.18 Core log (Mine A)  
Figure 4.19 Result of borehole scoping (Mine B)
4.2.2.3 Mine C Underground Test

Mine C was located in southern West Virginia. A diagram of the test area and the core log are shown in Figures 4.20 and 4.21, respectively. As shown in Figure 4.21, the roof of the test area consists of about 35 in shale overlain by sandstone layer that extends at least 35 in above the shale.

<Drilling Settings>
- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 0.8, 1.1, 1.5, 5.1 in/sec and free
- Rotation rate: 400, 500 rpm and free
- Thrust cap: 200, 400, 650 psi
- Bit size: 1-1/32 and 1-3/8 in
- Number of drill holes: 36 holes

4.2.2.4 Mine D Underground Test

Mine D was located in southern West Virginia. A sandstone channel was observed. Drilling tests were focused in and around this sandstone channel. Because the edges of the channel had a steep slope, data could be collected from three different geologic conditions within close proximity. Drilling occurred in the sandstone channel yielding drilling data that intercepted only sandstone. Drilling was also conducted outside the sandstone channel, but near its edge, yielding drilling data the intercepted shale and sandstone, respectively. Finally, drilling was conducted at some distance from the channel, yielding drilling data that only intercepted shale. Figures 4.22 and 4.23 show a diagram of the test area and core logs, respectively.
<Drilling Settings>

- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 0.8, 1.1, 1.5 and 1.7 in/sec
- Rotation rate: free
- Thrust cap: 650 and 800 psi
- Bit size: 1-3/8 in
- Number of drill holes: 27 holes

Figure 4.22  Diagram of test site (Mine D)

Figure 4.23  Core logs (Mine D)
4.2.2.5 Mine E Underground Test

Mine E was located in southern West Virginia. The roof at test site was a consistent sandstone layer within the range of drilling of the roof bolter. Vertical holes were drilling into the sandstone roof. Despite the consistent roof strata, three different geological conditions were drilled using the angle drilling features of the roof bolter (Figure 4.24). The angle drilling feature of the roof bolter allows holes to be drilled at any angle from the vertical up to 90 degrees. Two geologic features were intercepted using this feature. A high angle (approximately 75 degrees from horizontal) mining-induced fracture was present in the roof. 4 holes were drilled to intercept this feature using the angle drilling feature of the roof bolter. The angle drilling feature was also used to begin drilling in the coal rib of the mine. As drilling proceeded, the drill bit intercepted the sandstone roof some distance into the hole. This procedure enabled the data collection system to monitor drilling through two distinct layers of strata in one hole. Figures 4.25 and 4.26 show a diagram of the test area and core logs, respectively.

<Drilling Settings>

- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 1.5 in/sec
- Rotation rate: free
- Thrust cap: 800, 1,000 and 1,100 psi
- Bit size: 1-3/8 in
- Number of drill holes: 18 holes

Figure 4.24  Angle drilling with roof bolter (Mine E)
Figure 4.25  Diagram of test site (Mine E)

Figure 4.26  Core logs (Mine E)
4.2.2.6 Mine F Underground Test

Mine F was located in southern West Virginia. In the testing site, soft shale strata were in the roof bolting horizon. Figure 4.27 shows a diagram of the test site. Figure 4.28 shows the core and core log. Bit wear test was also conducted during this test. As the rock sample was too soft to make the specimen, point load test was conducted to estimate the strength of the roof rock. The unconfined compressive strength of it was estimated at 3,400 psi or less.

<Drilling Settings>
- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 0.4, 0.8, 1.1, 1.3, 1.5, 1.7 and 2.1 in/sec
- Rotation rate: 400, 500, 600, 650 and 700 rpm
- Thrust cap: 1,000 psi
- Bit size: 1-3/8 in
- Number of drill holes: 103 holes

Figure 4.27  Diagram of test site (Mine F)
4.2.2.7 Mine G Underground Test

Mine G was located in southern West Virginia. In the testing site, hard sandstone roof strata were in the roof bolting horizon. Figure 4.29 show a diagram of the test site. Figure 4.30 shows the core and core log. Bit wear test was also conducted. Besides, some holes were drilled by feedback system mode (horsepower curve and bite curve setting). From the results of rock property test, the unconfined compressive strength of roof rock is about 9,400 psi on average, ranging from 8,200 to 10,400 psi. Table 4.3 shows the result of rock property test.

<Drilling Settings>
- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant, and feedback system mode = penetration rate and rotation rate were controlled according to both horsepower and bite curves selected.
- Drilling parameters
  - WVU mode
    - Penetration rate: 0.4, 0.6, 0.8, 0.9, 1.1, 1.3, 1.5, 1.7 and 2.1 in/sec
    - Rotation rate: 400, 500, 600, 650 rpm and free
(b) Feedback system mode
   - H.P. = 0.18, 0.5, 1.01, 1.1, 1.5, 2, 2.24, 2.5, 3 and 4
   - Bite curve: 0.063, 0.074, 0.101 and 0.125

(c) Bit wear test
   - Penetration rate: 1.1 in/sec
   - Rotation rate: 400 and 650 rpm
   - Number of drill holes with one bit: 3, 5 and 7** holes
     ** After the 7th hole was completed, the bit broke.

- Thrust cap: 1,000 psi
- Bit size: 1-3/8 in
- Number of drill holes: 268 holes

Table 4.3  Physical properties of core samples (Mine G)

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>UCS (psi)</th>
<th>BTS (psi)</th>
<th>$E_{50}$ ($\times 10^6$ psi)</th>
<th>Unit Weight (lbs/ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>9,392</td>
<td>589.6</td>
<td>2.082</td>
<td>150.9</td>
</tr>
</tbody>
</table>

Figure 4.29  Diagram of test site (Mine G)
4.2.2.8 Mine H Underground Test

Mine H was located in southern West Virginia. In the testing site, hard sandstone roof strata were in the roof bolting horizon. Figure 4.31 shows a diagram of the test site. Figure 4.32 shows the core and core log. From the results of rock property test, the unconfined compressive strength of roof rock is about 9,700 psi on average, ranging from 8,900 to 10,300 psi. Table 4.4 shows the result of rock property test.

<Drilling Settings>
- DCU control mode: WVU mode = both penetration rate and rotation rate were controlled or kept constant.
- Penetration rate: 0.4, 0.8, 1.1, 1.3, 1.5, 1.7 and 2.1 in/sec
- Rotation rate: 400, 500, 600 rpm and free
- Thrust cap: 1,000 psi
- Bit size: 1-3/8 in
- Number of drill holes: 56 holes

Table 4.4 Physical properties of core samples (Mine H)

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>UCS (psi)</th>
<th>BTS (psi)</th>
<th>( E_{50} ) ( \times 10^6 ) psi</th>
<th>Unit Weight (lbs/ft(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>9,770</td>
<td>521.3</td>
<td>1.649</td>
<td>151.9</td>
</tr>
</tbody>
</table>
Figure 4.31  Diagram of test site (Mine H)

Figure 4.32  Core and core log (Mine H)
CHAPTER 5

DEVELOPMENT OF DATA INTERPRETATION METHODOLOGY

In designing a proper roof support system, one must know the features of roof geology in advance of mining. These geological features include: rock type, rock strength, rock layer interfaces, voids, cracks, and bed separations. The drilling parameters obtained during normal roof bolt installation cycle can provide a large amount of information on roof geology when properly interpreted. It is also an economic way for identifying the geological features of the roof. One of the main tasks of this project is to develop a method for predicting roof geology based on the drilling parameters obtained during normal roof bolting operation. In this project, experimental studies were conducted to determine the relationship between the drilling parameters and geological features based on the results of a series of laboratory and underground tests as mentioned in Chapter 4. Study about the characteristics of the machine and control system used in this project was also conducted to eliminate their effects on drilling parameters. Because, as the drilling was controlled by DCU in this test (WVU control mode), it is necessary to distinguish the changes of drilling parameters that was caused by geological change and that by the machine characteristics/control algorithm. Based on the above results, an empirical model for roof geology prediction has been developed.

This chapter describes the results including the methods for determining quantitatively the location and size of voids/fractures and estimation of roof rock strength from the recorded drilling parameters of roof bolter.

5.1 Concept of Data Analysis and Rule Generation

The basic assumptions for this research are as follows:

- Variation in geological properties of roof strata will result in the change of some of the measured drilling parameters. Those drilling parameters that obviously reflect the change of roof geological properties are referred as relevant drilling parameters.

- Different roof geological properties are statistically independent. This independence could be a result of their different physical properties, for example, their different compressive strengths and shear strengths.

- Each relevant drilling parameter has a Gaussian or normal distribution for each roof geological property, but with different means and variances.

These assumptions provide the foundation for using a statistical model called finite mixtures in this research. In this case, a mixture is a set of k probability distributions, representing k geological properties, that governs the drilling parameter...
values for members of that geological property. In other words, each distribution gives the probability that a particular data point would have a certain set of drilling parameters values if it were known to be a member of that geology property. Each geological property has a different distribution. Any particular data point really belongs to one and only one of the geological properties. Roof geological properties are not likely to be equal: there is some probability distribution that reflects their relative populations.

Based on the above mentioned assumptions, the following procedures are taken for generating the rules of predicting the voids in roof rock by using the roof drilling parameters as shown in Figure 5.1.

5.2 Effect of Drilling Settings and Machine/Drilling Conditions on Drilling Parameters for Compensation Runs

As the data collection system used in this series of research was designed not for prediction of roof geology but for control drilling, the drilling parameters which can be measured are the parameters in the hydraulic system. This means that the drilling parameters contain not only for drilling rock but also for running machine itself. First of all, one needs to know how much the drilling parameters, especially feed pressure / rotation pressure, consumed for running the machine itself and how much impact different drilling settings and machine conditions (i.e. oil temperature in the hydraulic system) have on them. So, the first set of experiments was performed by drilling nothing to determine the consistency of the drilling parameters in the air. These data collected when drilling in the air are referred as compensation run data. Besides, compensation run was always conducted before every drilling to determine the machine and control conditions.

5.2.1 Effect on Feed Pressure

Figure 5.2 shows the feed pressure, carriage position and mast position curves for compensation run. It can be seen that the magnitude of feed pressure consumed for running the machine itself changes. This is caused by the change from carriage stage to mast stage. This machine can drill a 52-54 in deep hole in one cycle by two stages as shown in Figure 5.3. So, it was considered that feed pressure consumed for running the machine depends on the net bit position not on the bit position from the roof line. Here, the difference between bit position and net bit position is shown as follows;
Figure 5.2  Relationships between feed pressure /carriage position/mast position and elapsed time (P.R. = 0.4 in/sec, R.R. = 600 rpm)

Figure 5.3  Carriage and mast stages

\[
\text{BitPosition}(t) \text{(in)} = 0.1485 \times \text{MastPosition}(t) + 0.1593 \times \text{CarriagePosition}(t) - \text{BitPosition}(t_1) \\
= 0.1485 \times \text{MastPosition}(t) + 0.1593 \times \text{CarriagePosition}(t) \\
- 0.1485 \times \text{MastPosition}(t_1) - 0.1593 \times \text{CarriagePosition}(t_1)
\]  

(5.1)

\[
\text{NetBitPosition}(t) \text{(in)} = 0.1485 \times \text{MastPosition}(t) + 0.1593 \times \text{CarriagePosition}(t)
\]  

(5.2)

where, \( t_1 \) = the time when drilling starts, mast position \( t = \) output count of mast position sensor and carriage position \( t = \) output count of carriage position sensor. The conversion factors of mast and carriage position sensors are 0.1593 in/count and 0.1485 in/count, respectively. Note bit position refers to the distance from roof line including initial startup adjustments while net bit position refers to the actual bit drilling distance.

There is about 7-10 output units difference between feed pressure in carriage stage and that in mast stage. This means the magnitude of feed pressure is different with the different stages even drilling in a homogeneous rock. As shown in Figure 5.4, this feed pressure change was caused by the machine itself.
Figures 5.5 and 5.6 show the relationship between feed pressure and net bit position for compensation runs under different controlled settings and oil temperatures, respectively. It can be seen that drilling settings and oil temperature have no obvious impact on both the magnitude and trend of feed pressure-net bit position curve for compensation run. Hence, it was concluded that feed pressure for compensation run can be defined as a function of net bit position only in this machine and the machine effect on the feed pressure can be eliminated from the drilling data once feed pressure-bit position curve for compensation run has been recorded at any settings. From the compensation run
data obtained so far, the approximate curve for the relationship between feed pressure and net bit position is defined by the following equation (Figure 5.7);

\[
FP_c(\text{BP}_{\text{net}}(t_n)) = -5.49 \times 10^2 + 1.21 \times 10^3 \times \text{BP}_{\text{net}}(t_n) - 1.08 \times 10^6 \times \{\text{BP}_{\text{net}}(t_n)\}^2 + 5.08 \times 10^4 \times \{\text{BP}_{\text{net}}(t_n)\}^3 \\
- 1.37 \times 10^2 \times \{\text{BP}_{\text{net}}(t_n)\}^4 + 2.14 \times 10^4 \times \{\text{BP}_{\text{net}}(t_n)\}^5 - 1.78 \times 10^6 \times \{\text{BP}_{\text{net}}(t_n)\}^6 \\
+ 6.20 \times 10^9 \times \{\text{BP}_{\text{net}}(t_n)\}^7
\]  
(5.3)

(Number of data points = 27,626, variance = 0.7163, correlation coefficient = 0.945)

where, \(\text{BP}_{\text{net}}(t_n)\) = net bit position and \(t_n\) = elapsed time after drilling starts

As the feed pressure for compensation runs defined by equation (5.3) is subtracted from the feed pressure when drilling rock, the machine effect can be eliminated and how much feed pressure consumed for drilling rock can be determined. This value is called the “net feed pressure”. The net feed pressure is defined by the following equation;

\[
\text{Net Feed Pressure}(\text{BP}_{\text{net}}(t_n)) = FP_{DR}(\text{BP}_{\text{net}}(t_n)) - FP_C(\text{BP}_{\text{net}}(t_n))
\]  
(5.4)

where, \(FP_{DR}\) = feed pressure when drilling in rock.

Most likely, as mentioned above, different drilling settings and machine conditions (i.e. oil temperature) have no obvious impact on feed pressure-bit position curve for compensation run. However, as shown in Figure 5.8, the magnitude of feed pressure is always slightly higher at the beginning of test. This point should be taken into account when the feed pressure-bit position curve for compensation runs is approximated. Moreover, from Figure 5.9, it can also be seen the difference among the drilling data for different date, it seems that the machine condition also has a small impact on the
magnitude of feed pressure-bit position curve for compensation run. From these results, it was suggested that compensation runs should be conducted at the beginning and the end of each shift to check the machine condition and minimize the errors when calculating the net feed pressure. Besides, the rule for determining the approximate curve should be developed in order to apply it for practical use.

5.2.2 Effect on Rotation Pressure

Figure 5.10 shows the relationship between rotation pressure and net bit position for compensation runs under different controlled settings. It can be seen that different rotation rates have obvious impact on the magnitude of rotation pressure. This is because of the increasing friction from bearings, gears and oil flow in the pipe. Figure 5.11 shows the relationship between rotation pressure and bit position for compensation runs and drilling rock under different oil temperatures. From this figure, it can be recognized that different oil temperature also have obvious impact on the magnitude of rotation pressure and the impact of oil temperature when drilling rock is the same as that for compensation run. Since the locations of both drill holes are close to each other, the change of rotation pressure is not caused by the change of roof geology but by the change of oil temperature. This phenomenon may have been caused by the change of viscosity of hydraulic oil. Generally speaking, the viscosity of hydraulic fluid is altered by its temperature; the higher the oil temperature is the lower the viscosity of the hydraulic fluid is. Especially, it changes dramatically at low oil temperature. Accordingly, one question arises: why the effect of oil temperature was recognized only in rotation pressure but not in feed pressure? One conceivable reason is the difference between the magnitudes of oil flows. The feed flow varied from 0.857 to 4.820 gpm when penetration rate was set from 0.4 to 2.1 in/sec. On the other hand, rotation flow varied from 21.5 to 30.8 gpm when rotation rate was set from 400 to 600 rpm. But this point has not been confirmed yet. Besides, it can also be seen from Figure 5.11 that the effects of rotation rate and oil temperature on rotation pressure are too large to be ignored comparing with that in drilling rock. Hence, in order to estimate roof rock strength based on the magnitude of rotation pressure, a model or formula for eliminating these effects needs to be developed. On the other hand, in order to develop a model or formula to eliminate these effects, there are still many unknown factors. In addition, their effects seem to be complicated. For example, even if the same hydraulic oil is used, its characteristics change with conditions (new or old, polluted, etc).

5.2.3 Effect of Drill Rod

Basically, compensation runs were conducted without a drill rod and bit. So, in order to check the effect of drill rod on drilling parameters, compensation runs with drill rod were also conducted. Drilled holes were used for this test. As shown in Figure 5.12 (a) and (b), even though there was some frictional resistance between bit/rod and the hole surface, the existence of drill rod has no obvious impact on drilling parameters. The effect of drill rod on drilling parameters is so small in comparison with the feed pressure and rotation pressure consumed for running the machine and drilling rock. It is therefore concluded that it is not necessary to take into account the effect of drill rod on drilling parameters.
Figure 5.10  Effect of drilling settings on the magnitude of rotation pressure for compensation run

Figure 5.11  Effect of oil temperature on rotation pressure for compensation run and drilling rock (P.R. = 1.1 in/sec & R.R. = 600 rpm)
5.3 Void / Fracture Prediction

5.3.1 Determination of Relevant Drilling Parameters for the Voids/Fractures

The first step of data analysis is to find the most appropriate drilling parameters for void/fracture prediction. So, all the measured drilling data for a hole are plotted on a figure for comparison. Examples of the plotted data for holes drilled in the air, solid concrete and fractured blocks are shown in Figures 5.13 (a)-(c), respectively. All data are expressed in output counts of sensors. The basic principle for the drilling parameter selection is to look for highly discriminative drilling parameters regarding the rock voids and keep the number of selected drilling parameters as small as possible. Comparing these three figures, it can be seen that the feed pressure changes dramatically to form valleys around the locations of 15, 30 and 45 in where the voids are located. It is also observed that rotation pressure has some kind of change around the locations of voids but the magnitude of the change is much smaller than that of feed pressure. The other drilling parameters do not change at the presence of voids. Therefore, it can be concluded that feed pressure is the most relevant drilling parameter for the existence of voids/fractures.

5.3.2 Mechanism of Rock Fragmentation When a Void/Fracture is Encountered

The phenomenon that a void in rock induces a valley in feed pressure can be explained by the mechanism of rock fragmentation. Feed pressure which to some extent represent the thrust force that the roof bolter provides for drilling in rock. According to the theory of rock fragmentation due to cutting, cracks will be created and propagate under a thrust force applied to by a drilling bit\(^{(14),(15)}\). Once the cracks reach the free face of rock void, the whole pieces of the remaining rocks between drilling bit and free face of rock void are completely broken off.
rock void will begin to break into small chips as shown in Figure 5.14. As the drill bit keeps advancing, some of these chips will further be broken into even smaller chips by the rotation of the drill bit and sucked out by the dry dust collector through the drill rod while others will be pushed into the void. Air will fill the space left by the removed rock chips. Therefore, the resistance that the drill bit needs to overcome will rapidly drop. Consequently, this result in a rapid drop of the force needed to keep the drill bit advancing and rotating because the penetration rate is maintained at a pre-set level for each hole. Since the void is filled with the air, the feed pressure should drop to the level when drilling in the air. Once drill bit encounters rock again after going through the void, the feed pressure will rapidly climb back to the level when drilling in rock. This is how
Figure 5.14 Rock fragmentation when drilling close to a rock void

Figure 5.15 Conditions of both surfaces of void.
the valleys in feed pressure curve are formed. If the size of the void is large enough, there should be a plateau at the valley bottom of feed pressure curve as the drill bit penetrates through the void. It seems that the width of the plateau can be used to estimate the size of void. Figure 5.15 shows the concrete blocks with a simulated void that drilling test was conducted. From the chipping off nature of the void edge, the mechanism of rock fragmentation that mentioned above can be verified.

Although the feed pressure tends to reduce the level when drilling in the air if a void is encountered, it is observed that the valley bottoms of all feed pressure curves do not reach the level when drilling in the air. Besides measurement errors, the size of the void plays the most important role in this phenomenon. The smaller the size of void is, the less possible it is for the feed pressure to fall into the range when drilling in the air (Table 5.1). Two possible reasons can be considered for this trend: One is that rough surfaces of concrete layers may make the actual void size smaller than the designed one, or close to zero. Another reason is the small void does not provide enough space for broken rock chips to move in although cracks have already propagated to the void. Consequently, these rock chips are still confined to their original locations before they are further broken into smaller chips and removed by the dust collector, as normal drilling does. In this situation, the magnitude of feed pressure is much higher than that for drilling in the air. Therefore, the valley bottoms of feed pressure will be very shallow or there is no valley at all.

Table 5.1 Statistics of valley bottoms of feed pressure

<table>
<thead>
<tr>
<th>Void Size (in)</th>
<th>Number of holes with available data</th>
<th>Number of valley bottoms of feed pressure reaching the level in the air</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/16</td>
<td>22</td>
<td>5</td>
</tr>
<tr>
<td>1/8</td>
<td>22</td>
<td>19</td>
</tr>
<tr>
<td>3/8</td>
<td>19</td>
<td>15</td>
</tr>
</tbody>
</table>

5.3.3 Criterion for Void/Fracture Prediction

The major criterion for void prediction is developed based on the fact that feed pressure should drop to the level of drilling in the air when a void in rock is encountered. Here, as mentioned in 5.2.1, different drilling settings and machine conditions have no obvious impact on both the magnitude and trend of feed pressure-bit position curve when drilling in the air. So, this criterion is not affected by any drilling setting and machine conditions. Hence, once the compensation run is conducted and the feed pressure-bit position curve is recorded, this major criterion can be determined. Moreover, to help increase the correct prediction ratio, especially at a narrow void/fracture, a set of supplementary prediction rules are also developed considering not only the magnitude of feed pressure but also the shape of feed pressure valley.

In this research, it is assumed that the change in roof geological properties results in the change in roof drilling parameters. But this does not mean that all the changes in
the collected drilling parameters are caused by roof geology change. Those that are not caused by roof geology change are referred to as noise data. These noise data may come from drilling system, data collecting system or some other unknown sources. In order to distinguish the noise data from the normal ones, it needs in-depth information about roof bolter, data collecting devices and roof bolter operating procedures. Therefore, a set of rules for cleaning up the noise data are also created based on current knowledge about the machine used in this research in order to eliminate the effect of noise data on prediction as much as possible.

5.3.4 Prediction Results

The measured drilling parameters of 22 holes drilled in the fractured block were used to check the criteria for void/fracture prediction. The results are shown in Table 5.2. The prediction results show that a very high prediction percentage have been achieved for the 1/8 in and 3/8 in voids. But the 1/16 in void does not cause an obvious change in not only feed pressure but also all other drilling parameters. It seems that there is a limitation of the void size that can be detected by the current system.

<table>
<thead>
<tr>
<th>Void Size (in)</th>
<th>Actual Location (in)</th>
<th>Number of holes with available data</th>
<th>Number of correct prediction</th>
<th>Percentage of correct prediction (%)</th>
<th>Average predicted location (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/16</td>
<td>15</td>
<td>22</td>
<td>13</td>
<td>59.09</td>
<td>14.036</td>
</tr>
<tr>
<td>1/8</td>
<td>30</td>
<td>22</td>
<td>22</td>
<td>100</td>
<td>29.222</td>
</tr>
<tr>
<td>3/8</td>
<td>45</td>
<td>19</td>
<td>18</td>
<td>94.73</td>
<td>44.494</td>
</tr>
</tbody>
</table>

5.3.5 Prediction of the Size of Void/Fracture

Mining Engineers are interested in not only the void location but also its size in the roof strata when designing roof supports. The relationship between feed pressure valley and size of void is modeled as shown in Figure 5.16. The width of the left side feed pressure valley can be expressed as follows (See Figures 5.14 and 5.16);

\[
W = \begin{cases} 
R + F & \text{If there is no plateau at the valley bottom} \\
R & \text{If there is a plateau at the valley bottom} 
\end{cases}
\]  

(5.5)

where \(W\) = the left side width of feed pressure valley, \(R\) = the distance between drilling bit tip and the free face of the void when feed pressure begins to drop and \(F\) = the size of the void.

On the other hand, the right side of a feed pressure valley has nothing to do with the size of void. It mainly depends on the strength of rock since it represents the distance for a drill bit to start drilling in rock until the feed pressure reaches the level in rock.
Hence, it can be said that the width of the plateau at the bottom of feed pressure valley is much closer to the size of the void. Table 5.3 shows that the relationship between the width of plateau of feed pressure valley and the size of the void. It can be said that the larger the size of the void/fracture is, the more possible it is to form a plateau at their bottom of feed pressure valley and the more precisely it can be predicted. On the other hand, it seems to be difficult to determine the size of a small void by using the current data collecting system. This is because of the resolution of bit position sensors. The conversion factors for mast and carriage position sensors are 0.1485 in and 0.1593 in, respectively. This means that the minimum measurable plateau width is 0.1485 in, this value is much larger than 1/16-in. Considering measurement errors, the error rate of prediction will be considerable. Moreover, the plateau may begin to form even before a drill bit reaches the void if broken rock chips are removed fast. This can explain why the maximum plateau size is almost three times of a 3/8-in void.

### 5.3.6 Field Verification of Void/Fracture Prediction

Figures 5.17 and 5.18 show the feed pressure curve and the result of borehole scoping in Mine F (Roof Rock: Shale) and Mine G (Roof Rock: Sandstone), respectively. It can be seen that the locations of feed pressure valleys and their bottom that drops down

<table>
<thead>
<tr>
<th>Void Size (in)</th>
<th>Number of holes with available data</th>
<th>Number of holes with plateau</th>
<th>Max. plateau width (in)</th>
<th>Min. plateau width (in)</th>
<th>Ave. plateau width (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/16</td>
<td>22</td>
<td>5</td>
<td>0.3186</td>
<td>0.1593</td>
<td>0.223</td>
</tr>
<tr>
<td>1/8</td>
<td>22</td>
<td>13</td>
<td>0.5940</td>
<td>0.1485</td>
<td>0.3084</td>
</tr>
<tr>
<td>3/8</td>
<td>19</td>
<td>13</td>
<td>0.9958</td>
<td>0.1593</td>
<td>0.4043</td>
</tr>
</tbody>
</table>
Figure 5.17  Feed pressure curves and results of borehole scoping
(Mine F, roof rock: shale)

Figure 5.18  Feed pressure curves and results of borehole scoping
(Mine G, roof rock: sandstone)
Figure 5.19 Drilling parameters and results of borehole scoping (Mine G)
to the level of compensation run match well with the actual void locations. On the other hand, Figure 5.18 shows the trend that feed pressure did not drop to the level of that obtained in compensation run under the high penetration rate drill settings. It seems that the higher the penetration rate is the higher the bottom level of the feed pressure valley is. This may be because the drill bit has already reached the other free face of the void before the feed pressure drops down to the level of drilling in the air. This result also indicates that supplemental prediction criteria considered the shape of feed pressure valley is needed to enhance the prediction accuracy. From the results of a series of underground tests, the criterion for void/fracture prediction developed from the results of laboratory tests was verified.

On the other hand, the following two trends can also be observed. One is that the actual void location from the roof line was a little deeper than the location of feed pressure valley on the drilling data. This is because of the following drill procedures. At first, the drill bit was penetrated a few inches deep manually. And then, drilling controlled by DCU & data recording start. As for roof mapping, to record the relative locations of voids is considered to be one of the important factor in the near future. Another is that even two voids exist in close proximity, only a single wide valley is observed in the feed pressure curve (Figure 5.19). This trend seems to depend on conditions such as intervals between voids, rock strength, existence of filling materials, etc.

5.4 Estimation of the Rock Strength

The same way as the development of void/fracture prediction methodology, the most appropriate drilling parameters for estimating rock strength and developing the criterion for rock classification can be found based on the behaviors of drilling parameters when drilling in different strengths of rocks. Note the parameters which can be measured are the parameters in hydraulic system. So, the drilling parameters contain not only for drilling rock but also for running machine itself. Therefore, the results that are indicated in Section 5.2 have to be taken into account for discussion.

5.4.1 Impact of Different Rock Strength on Feed Pressure and Rotation Pressure

Figures 5.20 (a) and (b) show the impact of rock strength on feed pressure and rotation pressure at P.R. = 1.5 in/sec and R.R. = 600 rpm. It can be seen obviously that the harder the roof rock is the larger the feed pressure is. On the other hand, even though the impact of rock strength on the magnitude of rotation pressure can be recognized, it is not so clear cut. Figure 5.21 shows the same trend within one hole. Besides, the trend and magnitude of feed pressure for compensation run is not affected by any drilling settings and machine conditions (i.e. oil temperature) as mentioned in Section 5.2, the difference between their magnitudes of feed pressure represents the difference between their consumption for drilling rocks of different strength directly. On the other hand, the different rotation rate and machine conditions (i.e. oil temperature) have obvious impact on the rotation pressure for running the machine itself, and the magnitude of their impacts
are too large to be ignored comparing with that for drilling rock. It also seems to be difficult to develop a model or formula to eliminate these effects on rotation pressure. Hence, it would be true that it is difficult to evaluate the roof rock strength by using the magnitude of rotation pressure under the current system.

Therefore, it can be concluded that feed pressure is the most sensitive and reliable parameter for estimating the strength of roof rock under the current system.
5.4.2 Effect of Penetration Rate on Feed Pressure

Figures 5.22 (a) and (b) show the relationship between feed pressure and bit position under different penetration rates in shale and sandstone, respectively. It can be seen obviously that different penetration rates have obvious impact on the magnitude of feed pressure. The higher the penetration rate is the larger the magnitude of feed pressure is. Moreover, it was also observed that the harder the roof rock is the larger the impact of penetration rate on the trend of feed pressure-penetration rate curve is. So, it seems that not only the magnitude of feed pressure but also the slope of feed pressure-penetration rate curve is related to rock strength. Accordingly, the relationship between feed pressure and penetration rate under different rock strengths is analyzed and discussed. Here, following assumptions were made for simplification.

a. Drilling data for location within 5 in from the beginning and the end of drilling were not used.
b. Each roof rock was assumed to be almost homogeneous.
c. Actual rotation rate can be kept constant as a target speed.
d. The effect of bit wear on feed pressure is not considered.
e. Compressibility of the hydraulic oil is not considered.

First of all, the penetration rate was calculated by using the heavily filtered method. This method was applied for penetration rate calculation in DCU. By using this value, the DCU attempts to control the feed flow as to make the penetration rate close to the target speed. This method is shown as follows;

![Figure 5.22 Effect of penetration rate on feed pressure (R.R. = 600 rpm)](image-url)
• Heavily Filtered Method

\[
PenetrationRate(t_n) = \frac{1}{\Delta t} \{\text{BitPosition}(t_n) - \text{BitPosition}(t_n - \Delta t)\} 
\]

\[
PenetrationRate_{HF}(t_n) = \frac{1}{4} \{PenetrationRate(t_n) \times 2 + PenetrationRate_{HF}(t_{n-1}) + PenetrationRate(t_{n-2})\} 
\]

Where, \( t = \) time, \( \Delta t = 100 \) msec

Figure 5.23 shows the relationship between feed pressure and penetration rate for different strength of rocks. The blue and red data points indicate drilling data for Mine F where the roof is shale and those for Mine G where the roof is sandstone, respectively. Two trends can be recognized from this figure. One is that the harder the roof rock is, the higher the magnitude of feed pressure is while the other is that the slope of feed pressure-penetration rate curve is different for different strengths of rocks. The harder the roof rock is the steeper the slope of feed pressure-penetration rate curve is. However, as distributions of both sets of data points vary widely, their trends and the boundary between them can not be clearly defined even if there is a difference in UCS of more than 5,000 psi. In regard to this point, the causes for their wide variation may be attributed to:

(1) The roof rock was not homogeneous.
(2) The penetration rate calculated by the heavily filtered method varies widely because of the resolution of two position sensors as shown in Figure 5.23 (Carriage Position Sensor 0.1485 in/count and Mast Position Sensor 0.1593 in/count).
(3) Actual rotation rate is not kept constant as a target speed.
(4) The magnitude of feed pressure consumed for running the roof bolter itself changes with each stage of mast feed as mentioned in Section 5.2.
(5) The effect of bit wear

The items we can discuss and evaluate quantitatively are items (2), (3) and (4). Here, items (2) and (4) are discussed as to eliminate both effects on the relationship between penetration rate and feed pressure for different strengths of rocks.

5.4.2.1 Effect of the Method of Penetration Rate Calculation

As shown in Figure 5.24, the penetration rate calculated by the heavily filtered
method (penetration rate (HF)) has wide variances. However, this trend and value are not for the actual penetration rate. This trend is because of the insufficient resolution of two position sensors. In other words, this variance is caused by the calculation method for penetration rate. In order to make the difference clear in the trend and distribution of both sets of data points of different strengths of rocks, one of the important key is how to calculate the penetration rate more exactly under the current system. So, first of all, in order to eliminate the effect of resolution on penetration rate calculation, the penetration rate was calculated from 10 backward points average (= penetration rate (10BP)) instead of heavily filleted method. This formula is defined as follows;

\[
Penetration\ rate_{(10BP)}(t_n) = \frac{1}{10} \left[ \text{penetration rate}(t_n) + \text{penetration rate}(t_{n-1}) + \cdots + \text{penetration rate}(t_{n-9}) \right]
\] (5.8)

Here, it is noted that speed is controlled by the amount of flow in the hydraulic system. Besides, pressure is the resistance of oil flow in the hydraulic circuit and pressure is also induced by the flow. So, it can be assumed that there are strong correlation between the actual penetration rate/feed pressure and feed flow. Consequently, the penetration rate can be calculated from the feed flow (= penetration rate (FF)) by using the following equation;

\[
Penetration\ rate_{(FF)}(t_n) = \left\{ feed\ flow(t_n) - 819 \right\} \times 0.00305 \times 231 / 7.07 / 60
\] (5.9)

where the conversion factor from output unit to actual hydraulic volume is 0.00305, 1 gallon = 231 in\(^3\), cross section of the cylinder = 7.07 in\(^2\) and both cylinder diameters in each stage are assumed to be the same.

Figure 5.25 shows the correlation between two penetration rates calculated by different methods and feed pressure. It can be seen that the penetration rate curve calculated from the 10 backward points average still has a variance. This calculation method has weak points in that the current calculation speed still reflects the information of previous locations and, in the same way, a current change is not reflected on the current speed directly. So, there is still difference in the trend between the penetration rate curve calculated from the 10 backward points average and feed pressure curve. On the other hand, as comparing with the penetration rate curve calculated from the feed flow, it can be seen that the change of feed pressure curve coincides with that of the penetration rate curve calculated from the feed flow except the data around the void location. Both the feed pressure curve and the penetration rate curve change simultaneously. Moreover, it can be distinguished by using the penetration rate curve calculated from the feed flow whether the feed pressure change is caused by geology change or DCU. For example, the change of feed pressure around No.1 in Figure 5.25 was caused by geology because the feed flow did not change. In fact, there were voids around this location. On the other hand, the change of feed pressure around No.2 was caused by DCU because the feed flow itself changed and both curves changed simultaneously.
Figures 5.26 (a)-(c) show the relationships between feed pressure and penetration rates calculated by different methods. From these figures, by applying feed flow for penetration rate calculation, distribution of data points becomes narrower and its trend is made clearer. Therefore it can be concluded that feed flow should be applied for penetration rate calculation instead of the data of two position sensors in the current system. Hereafter, “penetration rate” means that calculated from the feed flow.
5.4.2.2 Elimination of the Effect of Oil Temperature and Cylinder Diameter on Penetration Rate Calculation

As mentioned above, from the hydraulic system and resolutions of two position sensors points of view, the penetration rate calculated from the feed flow is more reliable for calculating the penetration rate more exactly and determining the impact of rock strength on the relationship between feed pressure and the penetration rate under the current system. However, generally speaking, the oil leaks in the hydraulic circuit and

Figure 5.26 Relationship between penetration rates and feed pressure (R.R. = 600 rpm, sandstone)
the lower the oil temperature is, the more the oil leaks. This is because the elasticity of sealing materials in the hydraulic circuit. Therefore it can also be guessed that different oil temperatures have impact on the penetration rate calculation. Hence, in order to check this point and modify equation (5.9) if necessary, actual bit position calculated from the data of two position sensors and the bit position calculated from the feed flow are compared. Here, bit position was calculated from the feed flow by using formulas (5.10) and (5.11);

\[
\text{Penetration rate}(t) (\text{in/sec}) = \left\{ \text{feed flow}(t) - 819 \right\} \times 0.00305 \times 231 / 7.07 / 60
\]

\[
\text{Bit position}(t) (\text{in}) = \text{bit position}(t-1) + \text{penetration rate}(t-1) \times 0.1
\]

where, \( t = \text{time (sec)} \)

Figures 5.27 (a) and (b) show the relationships between elapsed time from the start of drilling and bit position calculated from the data of two position sensors (= actual bit position)/feed flow under the different oil temperatures. It can be seen that the lower the oil temperature is, the larger the difference between actual bit position and the bit position calculated from feed flow is. This difference may be caused by oil leakage in the hydraulic circuits. The lower the oil temperature is, the more the oil leaks. This result indicates that oil leakage has to be taken into account when penetration rate is calculated from feed flow, especially when oil temperature is low. Figure 5.28 (a) and (b) show the relationship between the actual bit position/the bit position calculated from feed flow and elapsed time under different penetration rate settings. It seems that the difference between both bit positions is dependent on the time and there is no effect of penetration rate

![Graphs showing bit position vs. elapsed time for different oil temperatures](image)

(a) Oil Temp = 108° output units  
(b) Oil Temp = 130° output units

Figure 5.27 Relationship between the bit positions calculated from the data of two position sensors/feed flow and elapsed time (P.R. = 0.4 in/sec, R.R. = 500 rpm)
setting on the magnitude of their difference. In other words, the amount of oil leakage seems to be dependent on the oil temperature only and penetration rate setting has no obvious impact on it. Besides, from Figure 5.29, even though there is no obvious change about the trend of bit position curve, the magnitude of feed flow was different with different stages. It seems that the conversion factor (cylinder diameter, etc) and/or the amount of oil leakage at the mast stage are also different with those at the carriage stage. From these points of view, an empirical formula for the oil leakage factor was developed in order to calculate more exactly the penetration rate from the feed flow.

Here, the following assumptions are used for trials:

a. Drilling setting has no obvious impact on oil leakage
b. The amount of oil leakage is only dependent on the oil temperature.
c. The magnitudes of oil leakage in both mast and carriage extension stage are the same.
d. Effective cylinder diameters of mast and carriage stage are different.
e. Compressibility of the oil fluid is not taken into account.

Based on the above assumptions, the calculation formula for penetration rate at each stage can be defined as follows;

\[
Penetration\ rate_{C}(t) = \left\{ feed\ flow(t) + F_{L}(T) - 819 \right\} \times 0.00305 \times 231/7.07/60 \quad (5.12)
\]

\[
Penetration\ rate_{M}(t) = a \times \left\{ feed\ flow(t) + F_{L}(T) - 819 \right\} \times 0.00305 \times 231/7.07/60 \quad (5.13)
\]

where, penetration rate \( C \) (t) = penetration rate at the carriage stage penetration rate \( M \) (t) = penetration rate at the mast stage, \( F_{L}(T) \) = oil leakage factor, \( T \) = oil temperature and \( a \) = constant.

At first, an attempt was made to determine the oil leakage factor from compensation run data at the carriage stage. Figure 5.30 shows the relationship between oil leakage factor and oil temperature at the carriage stage. It can be seen that the effect of oil leakage decreases with increase of oil temperature and the effect of oil leakage can be ignored in penetration rate calculation when oil temperature is larger than about 135 in output units. Then, an approximation curve, which represents the relationship between oil leakage factor and oil temperature, is determined. The oil leakage factor is defined as follows;

\[
F_{L}(t) = -5.34 \times T(t) + 718 \quad (5.14)
\]

Next, ‘a’ in equation (5.13) is determined by the data at the mast stage as follows;

\[
a = 1.09 \quad (5.15)
\]

From the equations (5.12)-(5.15), the empirical formula for penetration rate calculation by the feed flow can be defined as follows;
i) Oil Temperature < 135 output units

Carriage stage

\[ \text{Penetration rate}_c(t) = \{\text{feed flow}(t) - 5.34 \times T(t) - 101\} \times 1.66 \times 10^{-3} \]  \hspace{1cm} (5.16)

Mast stage

\[ \text{Penetration rate}_m(t) = \{\text{feed flow}(t) - 5.34 \times T(t) - 101\} \times 1.83 \times 10^{-3} \]  \hspace{1cm} (5.17)

ii) Oil Temperature ≥ 135 output units

Carriage stage

\[ \text{Penetration rate}_c(t) = \{\text{feed flow}(t) - 819\} \times 1.66 \times 10^{-3} \]  \hspace{1cm} (5.18)

Mast stage

\[ \text{Penetration rate}_m(t) = \{\text{feed flow}(t) - 819\} \times 1.83 \times 10^{-3} \]  \hspace{1cm} (5.19)

Figure 5.31 (a) and (b) show the examples of the revised results. The bit positions calculated by equations (5.16)-(5.19) fit the actual bit position well. Moreover, in order to verify these equations, they are applied to another set of independent data. From Figure

![Graph](attachment:image.png)

(a) P.R. = 0.4 in/sec, Oil Temp = 108 output units  
(b) P.R. = 2.1 in/sec, Oil Temp = 123 output unit

Figure 5.31  Relationship between the bit positions calculated by different methods and elapsed time
5.32, it can also be seen that bit position calculated by equation (5.16)-(5.18) fit the actual bit position well. From these results, it was concluded that the effect of oil temperature can be eliminated by applying equations (5.16)-(5.18).

Figure 5.33 shows the relationship between feed pressure and penetration rates calculated by the feed flow (= penetration rate \(FF\)). It can be seen that the revision of oil leakage effect makes the data at low temperature shift to the right and the variance of distribution of data points narrower.

Figure 5.34 (a) and (b) show the relationships between penetration rate calculated by different methods and feed pressure for different strengths of rocks. From these figures, by applying feed flow for penetration rate calculation, distribution of both sets of data points become narrower, and differences in both trends and the boundary of both distributions were made clear.
5.4.2.2 Elimination of Machine Effect on Feed Pressure

As mentioned in Section 5.2, the magnitude of feed pressure consumed for running the roof bolter itself changes and there is about 7-10 output units difference between feed pressure in carriage stage and that in mast stage. This means the magnitude of feed pressure is different with different stage even drilling in a homogeneous rock. This effect causes 7-10 output units variance in the distribution of feed pressure-penetration rate data points. On the other hand, drilling settings and machine conditions have no obvious impact on both the magnitude and trend of feed pressure-net bit position curve. Besides, the error for all the compensation run data is less than 3 output units. Therefore, the effect of machine itself on drilling parameters when drilling rock can be eliminated once the compensation run is conducted under any settings. For this reason, the net feed pressure is used for rock strength estimation instead of feed pressure. The net feed pressure was calculated by equation (5.3) and (5.4).

Figure 5.34 show the relationship between penetration rates calculated by different methods and feed pressure (R.R. = 600 rpm)

Figure 5.35 show the relationship between feed pressure/net feed pressure and penetration rate. Comparing with
5.4.3 Effect of Rotation Rate on Feed Pressure

Figures 5.36 (a) and (b) show the relationship between feed pressure and bit position under different rotation rates when drilling in shale and sandstone, respectively. It can be seen that different rotation rates also have obvious impact on the magnitude of feed pressure. The magnitude of feed pressure decreases with the increasing rotation rate. This result indicates that the rotation rate helps feed pressure in the drilling process. Besides, it was also observed that the harder the roof rock is the larger the impact of rotation rate on the magnitude of feed pressure is. Next, the effect of rotation rate on the trend of net feed pressure-penetration rate curve is determined. The approximate curves for each rotation rate setting vs. the strength of rock are defined as linear functions for
simplification. Figure 5.37 shows the approximate curves under different strengths of rocks and rotation rates. It can be seen that the higher the rotation rate is the more gentle the slope of net feed pressure-penetration rate curve is. Moreover, the harder the roof rock is the larger the impact of rotation rate on the slope of net feed pressure-penetration rate is. Therefore, it is concluded that the impact of rotation rate on the magnitude of net feed pressure is also related to rock strength.

5.4.4 Determination of the Boundary Planes for Estimating Roof Rock Strength

The results obtained so far clearly show that the magnitude of net feed pressure correlates well with the rock strength and both penetration rate and rotation rate have obvious impact on the magnitude of net feed pressure. Therefore both parameters have to be considered when it comes to roof geology prediction using the magnitude of net feed pressure. From Figure 5.37, the relationship among net feed pressure, penetration rate and rotation rate can be represented by the following equation;

\[ \text{Net Feed Pressure}(t_n) = F_{RS}(\text{Rotation Rate}(t_n)) \times \text{Penetration Rate}(t_n) + C_0 \]  

(5.20)

where, \( F_{RS} \) = function of rotation rate for each strength of rocks, \( C_0 \) = constant, and \( t_n \) = elapsed time after drilling starts.

Figure 5.38 shows the distributions of data points in Mine F and Mine G on the net feed pressure-penetration rate-rotation rate system. The roof rock in Mine F is shale (UCS < 3,500 psi) and that in Mine G is sandstone ranging from 8,400 to 10,400 psi with an average UCS of 9,500 psi. In order to classify the roof rock from the strength point of view, the upper and lower boundaries of each data set distribution are determined. Based on the trend of data set distributions and for simplicity, \( F_{RS} \) is assumed to be a linear function of rotation rate. Each boundary plane shown in Figure 5.39 is defined as;

Sandstone (UCS ≈ Ave. 9,400 psi)

- The upper boundary plane (UCS ≈ 10,500 psi)

\[ \text{Net feed pressure}(t_n) = \left\{ 35.84 - 0.02853 \times \text{rotation rate}(t_n) \right\} \times \text{penetration rate}(t_n) + 9 \]  

(5.21)

- The lower boundary plane (UCS ≈ 8,500 psi)

\[ \text{Net feed pressure}(t_n) = \left\{ 32.84 - 0.02753 \times \text{rotation rate}(t_n) \right\} \times \text{penetration rate}(t_n) - 1 \]  

(5.22)

Shale (UCS < 3,500 psi)

- The upper boundary plane (UCS ≈ 3,500 psi)

\[ \text{Net feed pressure}(t_n) = \left\{ 14 - 0.01 \times \text{rotation rate}(t_n) \right\} \times \text{penetration rate}(t_n) + 5 \]  

(5.23)
Mine G Sandstone (8,400-10,400 psi)

Mine F Shale (≤ 3,400 psi)

Figure 5.38 Distributions of both sets of data points (Mine F and Mine G)

Mine G Sandstone (8,400-10,400 psi)

Mine F Shale (≤ 3,400 psi)

Figure 5.39 Upper and lower boundary planes for distributions of both data points (Mine F and Mine G)
The lower boundary plane

\[
Net \text{ feed pressure}(t_n) = \left[12 - 0.01 \times \text{rotation rate}(t_n)\right] \times \text{penetration rate}(t_n) - 2 \quad (5.24)
\]

Where, \(t_n\) = elapsed time after drilling start

The 5,500 psi boundary plane is also determined similarly using the set of drilling data in concrete block. This boundary is represented by the following equation;

\[
Net \text{ feed pressure}(t_n) = \left[24.50 - 0.01833 \times \text{rotation rate}(t_n)\right] \times \text{penetration rate}(t_n) - 2 \quad (5.25)
\]

The boundary planes defined so far are shown in Figure 5.40.

![Figure 5.40 Boundary planes for estimating rock strength](image)

5.4.5 Verification of the Boundary Planes for Estimating Rock Strength

In order to verify the above boundaries, another set of independent data has been selected and plot them on Figure 5.40. From the results of the lab tests, the data from two kinds of concrete blocks were used; one is 12,340 psi high strength concrete block and the other is 4,000 psi concrete. These two different strengths of concretes data plotted on
the area of the approximate boundary planes are shown in Figure 5.41. It can be seen that the drilling data for 12,340 psi concrete block is distributed above the 10,500 psi boundary plane. On the other hand, those of the 4,000 psi concrete block are distributed around the 3,500 psi boundary plane. These results indicate that roof rock can be classified by these boundary planes defined in terms of rock strength.

Moreover, the strength of roof rock in Mine H (UCS Ave. ≈ 9,700 psi, ranging from 9,000 to 10,300 psi) is almost the same as that in Mine G (Ave. UCS ≈ 9,500 psi). So, the data in Mine H was also plotted on the net feed pressure-penetration rate-rotation rate system (Figure 5.40). It can be seen that the distribution of drilling data in Mine H is distributed between the 8,500 psi and 10,500 psi boundary planes. This result verifies that if the strengths of roof rocks are almost the same, the distributions of their drilling data in net feed pressure-penetration rate-rotation rate system are also the same.

From the above results, it can be seen that roof rock can be classified based on the magnitude of net feed pressure because it takes both the effects of penetration rate and rotation rate into account. In other words, the relationship among net feed pressure, penetration rate and rotation rate is a good indicator for estimating the strength of roof rock. The strength of roof rock can be determined and/or classified based on the location of data point in the net feed pressure-penetration rate-rotation rate system.

![Figure 5.41 Boundary planes for estimating rock strength and distributions of three sets of data points](image-url)
5.5 Bit Wear

One of the important factors that would affect the drilling performance is bit wear. So, bit wear tests were also conducted in a series of underground tests in order to observe the impact of rock strength on the magnitude of bit wear and its impact on drilling parameters.

5.5.1 Characteristics and Effect of Bit Wear on Drilling Parameters

5.5.1.1 Drilling in Soft Rock (Mine F, UCS < 3,500 psi)

Figure 5.42 shows a new bit and a drill bit after 17 full-length holes were drilled under several drilling settings. From the observation of old and new bits, bit wear was not recognizable. Figure 5.43 shows the relationship between feed pressure/rotation pressure

Figure 5.42 Drill bit (left, new bit: right, after having drilled 17 holes, shale)
and bit position before and after replacing a drill bit. Before replacing a bit, 17 full-length holes were drilled by using the same drill bit. By comparing the two data, it shows that there was also no change in both feed pressure and rotation pressure curves. Therefore, it was concluded that bit wear does not need to be taken into account in analyzing drilling data when drilling in soft rock. Incidentally, each drill bit was replaced by a new bit after 50 drilled holes at Mine F. By extrapolation one may further states that the magnitude of bit wear is not so big even after 50 holes drilled in the same soft roof rock.

5.5.1.2 Drilling in Hard Rock (Mine G, UCS ≈ 9,500 psi)

At first, we attempted to drill 10 holes with one bit under the controlled setting of P.R. = 1.1 in/sec and R.R. = 650 rpm. However, after completing the 7th hole, the drill bit broke as shown in Figure 5.44 and the magnitude of bit wear couldn’t be measured. So, the maximum number of drill holes with one bit was fixed at 5 and two different settings of controlled parameters were tested; P.R. = 1.1 in/sec & R.R. = 400 rpm and P.R. = 1.1 in/sec & R.R. = 650 rpm in order to study the effect of rotation rate on bit wear. Figure 5.45 shows a drill bit after 5 holes were drilled with different rotation rates (P.R. = 1.1 in/sec, R.R. = 400 and 650 rpm). The relationship between feed pressure/rotation pressure and bit position are shown in Figures 5.46 and 5.47. Both the magnitude of feed pressure and rotation pressure increase with increase of bit wear and the higher the rotation rate is the more the bit wear is. As an example, comparing the drilling data for the 1st hole to that of the 3rd hole, the difference at R.R. = 650 rpm is about 20 output units in feed pressure and about 10 output units in rotation pressure. On the other hand, the difference at R.R. = 400 rpm is about 15 output units in feed pressure and about 7 output units in rotation pressure.
Next, the effect of penetration rate on the magnitude of bit wear was analyzed. Figure 5.48 shows a drill bit after 3 holes were drilled under different penetration rates (P.R. = 0.4 in/sec and 1.5 in/sec, R.R. = 650 rpm). Figures 5.49 and 5.50 show the relationship between feed pressure/rotation pressure and bit position under different penetration rates. From Figure 5.48, it is observed that the magnitude of bit wear for P.R. = 0.4 in/sec is larger than that for P.R. = 1.5 in/sec. From Figures 5.49 and 5.50, even both the magnitudes of feed pressure and rotation pressure increase with increase of bit wear, there is no big difference between the magnitude of feed pressure/rotation pressure for the 1st hole and those for the 3rd hole. Compared with the results of different rotation rates, bit wear seems to be more affected by rotation rate than by penetration rate.

Figure 5.44 Drill bit (left, new bit & right, after 7 holes drilled) (sandstone, P.R. = 1.1 in/sec and R.R. = 650 rpm)
Figure 5.45  Drill bit (left, new: center, R.R. = 400 rpm: right, R.R. = 650 rpm) (sandstone, P.R. = 1.1 in/sec, number of holes drilled = 5)

Figure 5.46  Magnitude of feed pressure/rotation pressure for each drill hole (bit wear test, sandstone, P.R. = 1.1 in/sec and R.R. = 400 rpm)
Figure 5.47  Magnitude of feed pressure/rotation pressure for each drill hole (bit wear test, sandstone, P.R. = 1.1 in/sec and R.R. = 650 rpm)

Figure 5.48  Drill bit (left, new: P.R. = 0.4 in/sec; center, P.R. = 0.4 in/sec; right, P.R. = 1.5 in/sec) (sandstone, R.R. = 650 rpm, number of holes drilled = 3)
Incidentally, from the observation of drill bits, it was noticed that the magnitude of wear on the front tip was different from the other parts. Moreover, wear of bit was seen clearly not only on the front tip but also on the sides of it. These points are illustrated in Figures 5.45 and 5.48.
5.5.2 Effect of Bit Wear on Roof Geology Prediction

5.5.2.1 Effect on Void/Fracture Prediction

Figure 5.51 show the impact of bit wear on the relationship between feed pressure and bit position under different penetration rates. A fracture existed around 6-7 in deep from the roof line in the drilling area. From these figures, it can be seen that even though the magnitude of feed pressure increases with increase of bit wear, bit wear has no obvious impact on the trend and the level of the bottom of feed pressure valley. Therefore, it was concluded that bit wear does not affect the void/fracture prediction.

![Figure 5.51 Magnitude of feed pressure for each drill hole and the result of borehole scoping (bit wear test, sandstone, P.R. = 1.1 in/sec and R.R. = 650 rpm)](image)

5.5.2.2 Effect on Estimation of Rock Strength

From the above results, as bit wear and its effect on feed pressure and rotation pressure was not recognizable in soft rock, the effect of bit wear may not be considered when drilling in soft rock. However, when drilling in hard rock, bit wear was recognizable clearly and had obvious impact on the magnitude of feed pressure and rotation pressure. Moreover, different controlled settings of drilling parameters, especially different rotation rates, have obvious impact on the magnitude of bit wear. Therefore, when applying the methodology for estimating rock strength developed so far for production, the effect of bit wear on the magnitude of feed pressure and rotation pressure should be taken into account. This point needs to be explored further in order to apply to this system for normal operation.
5.6 Other Factors Considered

5.6.1 Repeatability

Repeatability means that drilling parameters are in the same magnitude and trend if drilling is conducted under the same drilling setting and roof geology. In this test, two holes were drilled close each other at each setting. Figure 5.52 (a) and (b) show the relationship between feed pressure/rotation pressure and bit position. It can be seen that the drilling data for 1st and 2nd holes are in good agreement with each other. It was therefore confirmed that the data are repeatable.

![Graphs showing feed pressure and rotation pressure](image)

Figure 5.52 Confirmation of repeatability (P.R. = 1.1 in/sec, R.R. = 600 rpm)

5.6.2 Effect of Control Mode (Feedback System Mode)

As mentioned above, drilling was not conducted manually but controlled by DCU in this research. One of the main keywords is “automatically”. WVU control mode, in which both penetration rate and rotation rate are controlled or kept constant, was mainly used in a series of underground and laboratory tests in order to determine the effect of penetration rate and rotation rate settings on drilling conditions and drilling parameters. Another control mode, the feedback system mode was also tested. Feedback system mode was developed and suggested by J.H. Fletcher & Co to be applied for normal drilling operation. In this mode, drilling is controlled by both horsepower curve and bite curve selected by the operator. Depend on the two curves and current condition (feed pressure), penetration rate and rotation rate are changed and controlled. It can be estimated that different control modes have different drilling behaviors and responses include drilling parameters. So it is necessary to determine, based on the drilling data, whether or not the interpretation methodology for roof geology prediction developed so far (based on the
data for drilling under the WVU control mode) can be applied for the interpretation of the drilling data controlled by feedback system mode.

A drilling test controlled by feedback system mode was conducted in Mine G where the roof is sandstone ranging from 8,400 to 10,400 psi with an average of 9,400 psi in UCS. Figure 5.53 shows the distribution of drilling data points and the boundary planes defined so far. From this figure, it can be seen that the drilling data in which drilling was conducted by feedback system mode are also distributed between the 8,500 psi and 10,500 psi boundary planes. It seems that the boundary planes can also be applied for interpretation of drilling data controlled by feedback system mode. However, the number of drill holes drilled by feedback system mode was only 12 holes and all of them were in only one type of roof rock. In addition, as there were no obvious voids/fractures in and around the drilled holes, it can not verify the void/fracture prediction criteria. Hence, in order to verify the applicability of the data interpretation methodology developed so far for the drilling data controlled by feedback system mode, more drilling test have to be conducted under several drilling settings and in different strengths of roof rocks with voids/fractures.

Figure 5.53  Distribution of drilling data points and boundary planes for estimating rock strength (Mine G, feedback system mode drilling)
5.6.3 Effect of Thrust Cap

The thrust cap can be implemented by two ways in the machine used for this project. One is by setting the relieve valve, while another is by setting the DCU. If the setting of DCU is smaller than that of relieve valve, the behavior of the feed pressure will be controlled by the algorithm of DCU once the feed pressure reaches beyond the preset value in DCU. In such a case, the feed pressure curve will be highly fluctuated because DCU tries to adjust the feed flow to prevent the feed pressure from exceeding the preset thrust cap (Figure 5.54). It can be seen from the feed PWM curve. So, this change is not caused by the change of roof geology but caused by DCU (control algorithm). On the other hand, if the setting of DCU is larger than that of the relieve valve, the behavior of the feed pressure will be controlled by the relieve valve when the feed pressure reaches beyond the preset value in the relieve valve. In this case, feed pressure curve is so smooth. In addition, as the DCU does not realized that the valve is relieved, the feed PWM curve keeps increasing to reach the target speed (Figure 5.55).

From these results, it can be concluded that the drilling behavior when the feed pressure reaches beyond the preset level of thrust cap is different from that when the feed pressure is below it. Besides, the drilling behavior when the feed pressure reaches beyond the level of thrust cap is different for different thrust caps. Hence, the methodology for prediction of roof geology when the feed pressure reaches beyond the level of thrust cap also needs to be explored further.

Figure 5.54 Drilling parameters for drilling in layered block # 5 (P.R. = 0.4 in/sec, R.R. = 500)
Figures 5.56 (a) and (b) show the relationship between net feed pressure and penetration rate under R.R. = 400 rpm and 600 rpm, respectively. From these figures, as mentioned above, it can be seen that different rotation rate have obvious impact on the slope of the net feed pressure-penetration rate curve and the higher the rotation rate is the lower the magnitude of net feed pressure is. Moreover, it was recognized that the slope of net feed pressure-penetration rate curve also changes under high penetration rates and low rotation rates, i.e. large bite depth. For example, when the rotation rate was set at 400 rpm, the top of the bit body was compared. As an example, when the rotation rate was set at 400 rpm, from Figure 5.56 (a), the slope of the net feed pressure-penetration rate curve becomes steep when penetration rate is larger than about 1.5 in/sec.

As for this trend, there are two possible reasons. One is the capacity of vacuum pump while the other is the bit geometry. As shown in Figure 5.57, the height of the front tip of the bit is 0.16 in. Once the bite depth is larger than this value, the drilling mechanism between bit and rock seems to have changed. Once there is an interaction between bit body and rock, changes in drilling parameters occur and the coats of paint on the top of the bit body may be wiped out entirely. As every bit was used to drill only one hole, the slope of net feed pressure-penetration rate curve and the condition of paint on the top of the bit body was compared. As an example, when the rotation rate was set at 400 rpm, from Figure 5.56 (a), the slope of the net feed pressure-penetration rate curve would...
Penetration Rate (in/sec)

(a) R.R. = 400 rpm

(b) R.R. = 600 rpm

Figure 5.56 Relationship between net feed pressure and penetration rate

Figure 5.57 Bit geometry (new bit)

seems to have changed and become steeper when penetration rate is larger than 1.3 in/sec. On the other hand, from Figure 5.58, it can be seen clearly the paint on the top of bit body was wiped off when penetration rate was larger than 1.3 in/sec. Similarly, when rotation rate was set at 600 rpm, changes in slope and condition of the pant were both observed when the penetration rate was around 2.1 in/sec (Figures 5.59). From these results, it seems that drilling mechanism may change when the bite depth is larger than 0.20 in/rev. which is larger than 0.16 in/rev. This means that the roof rock is broken as the bit is penetrated.

Therefore, it can be concluded that bit geometry and/or the capacity of vacuum pump affects drilling parameters and drilling mechanism may change when the bite depth (penetration rate/rotation rate) is large. Hence, the model or equations developed so far may need to be modified at high penetration rates & low rotation rates. This point also needs to be explored further.
Figure 5.58  Drill bit (R.R. = 400 rpm, number of holes drilled = 1)
Figure 5.59  Drill bit (R.R. = 600 rpm, number of holes drilled = 1)
CHAPTER 6
DATA VISUALIZATION AND DATA BASE SOFTWARE

6.1 Requirements and Capabilities for MRGIS

In a production environment, manual interpretation and management of the collected drilling parameters is not practical since hundreds of holes will be drilled a production day. A new software package, MRGIS (Mine Roof Geology Information System), has been developed to allow mine engineers to make use of the large amount of roof drilling parameters for roof support design.

The basic requirements and goal for MRGIS include:

- A stand alone software
- Run on PCs under Windows 9.x, Me and XP
- Flexible development environments
- Inexpensive distribution cost
- Important machine data from an ASCII file which contains drilling parameters recorded during drilling a roof hole
- Conduct data clearing for removal of possible errors and useless data when importing machine data into MRGIS automatically
- Store cleaned machine data in a database
- Allow the user to manage the information about drilling holes
- Allow the user to enter or select conversion factors
- Allow the user to enter bit parameters
- Allow the user to enter or derive the location of the collar of a hole
- Import and display current CAD format mine map (.DXF format file)
- Match the drilling hole map to imported mine map
- Track mouse movement with northing and westing coordinate
- Provide a routine to identify roof fractures based on drilling parameters and their locations using the rules developed by WVU
- Provide map tools (Zoom in, Zoom out, Pan, and Full extent) to allow user to search the specific area
- Allow the user to add a new mine map and corresponding hole location map
- Allow the user to remove all layers
- Allow the user to select any drilling hole on map
- Display roof fractures and their locations

Considering the distribution cost for hundreds of potential users, Visual Basic, Map Objects and OpenGL are selected as the development environments. This choice provides a flexible and powerful prototype and development environment. MS Access is selected as database engine.
Figure 6.1  Structure of MRGIS

MRGIS consists of four modules developed for separate tasks (Figure 6.1):

- Data importing and cleaning
- Data management
- Data interpolation
- Data visualization

6.2 Data Import and Data Cleaning

When a J.H. Fletcher & Co.’s twin-boom roof bolter is drilling a hole into the roof, drilling parameters are collected into an ASCII file with a specialized microprocessor. Drilling parameters data recorded by roof bolter are referred to as machine data and the ASCII file containing machine data is referred to as machine data file. A machine data file has two headlines and 17 column data (before 2004) or one headline and 15 column data (after 2004) as shown in Figure 6.2. 8 columns represent feed pressure, feed flow, rotation pressure, rotation flow, rotation rate, mast position, carriage position, and oil temperature, respectively. Other columns represent the information such as vacuum pressure, control signal and status of the electronics that indicate control condition but are not used for the interpolation of roof geology directly in this research. The drilling parameters are recorded every 0.1 second so a 54 in long hole will have 250 to 850 records, depending on the penetration rate and roof geology. Every hole drilled by the roof bolter will have such a file.

Since 8 out of 17 or 15 columns in a machine data file are used for the identification of roof geology, there is no need to import all data recorded in a machine data file into MRGIS in order to save the space for storing the data. Additionally, noise data is produced at the beginning and end of the hole and when the bit is plugged. MRGIS contains a procedure to remove noise data automatically.
Only the records having non-zero value in their columns are imported into MRGIS. A non-zero value indicates the DCU is controlling the drill. Hole mouth position is calculated from the first record with non-zero value in the first column and saved in the database of MRGIS. Before a record is imported into MRGIS, its bit position is calculated and compared with the bit position of the previous record, this record will be filtered out to ensure that every record for the same hole has a unique bit position.

To prevent a redundant machine data file from being imported into MRGIS, a checking procedure is also provided to ensure there is a unique set of machine data for each hole in MRGIS.

When drilling a hole in the roof, the operator of the roof bolter may stop drilling before the bit reaches the designed hole depth. This may be due to bit jam, clog, or other reasons. The same hole may be drilled again until its completion. This will produce two sets of machine data files for the same hole with different file names, one with uncompleted machine data and another with completed machine data. In order to distinguish the completed machine data file from uncompleted one, a checking procedure is provided in MRGIS to check the value of final bit position in a machine data file. Before a machine data file is imported into MRGIS, the user is asked to enter or select the designed hole depth. The designed hole depth is used to compare with the final bit position. If the final bit position is less than 0.95 × designed hole depth, the machine data in the machine data file will be classified as incompletely and is removed from the
database. A message will pop up to remind the user that the machine data file being imported is invalid.

MRGIS is specifically designed for the purpose of identifying roof geology from roof drilling data recorded by J.H. Fletcher & Co.’s roof bolter including DCU. It is compatible for the format of the machine data files produced by J.H. Fletcher & Co.’s roof bolters and DCUs. If the user tries to import the data from a file with a format different from that of machine data file, an error will be produced and the importing process will be stopped. If one is not sure if a data file with correct format is being imported, the user can use preview to check the format of a data file manually although it may take longer time to import data (Figure 6.3).

When a machine data file is imported, its name, size, data and time created are also automatically imported into the database of MRGIS, which can be used as searching criteria. The data and time created of a machine data file can also be used as the data and the time of finishing drilling the hole.

Roof geology data is directly related to specific location in underground. Peoples who use geology data are interested in not only “What” but also “Where”. To make roof geology data meaningful and useful, spatial data is necessary. Distinguishing itself from traditional geographic data which usually employs 2 dimensional (X and Y) coordinate system, roof geology data carries information associated with Z to represent location. Unfortunately, a machine data file contains only Z data (bit position above the roofline) but not X and Y information of a bolt hole. Although it is not elevation which is
commonly used as Z value in Geographic Information System, bit position provides enough Z information from roof support design perspective because peoples are more interested in what geology it is at what location above the roof line when designing roof bolting. So there is no need to convert bit position into elevation. But without X and Y data, drilling parameters are still less useful for roof support designing since people often rely on the knowledge of the geology within a relatively larger area instead of at the point of interest or nearby. Manually calculating and entering X and Y data for every drilling hole is time-consuming and labor intensive since hundreds of bolting holes will be drilled each day in a mine. To solve this problem, a routine is being developed to automatically drive X and Y location information of the drilling holes based on the drilling time, drilling sequence, and entry orientation. It is indispensable to develop the system for measuring and recording the relative location of each hole automatically.

Conversion factors and information about the seam being mined also need to be entered or selected from database when importing a machine data file.

6.3 Data Base Design

One of the tasks of MRGIS is to store and manage drilling parameters. To accomplish this goal, a database is created in MRGIS.

A “database” is a collection of interrelated data specifically designed to be shared by multiple users. Data redundancy is controlled and a uniform approach is used for accessing and modifying data within a database. Using a database to store the data has many advantages. Most importantly, all of the data is stored together to prevent data files from being lost or being updated without authorization. Any required updating can be completed efficiently with data integrity being maintained.

In MRGIS, the database has been implemented as MS Access (.mdb) file. This approach has been taken in order to ensure that there are commercial tools to alter and access the contents of the database besides MRGIS. Also we have implemented the functionality and the User Interface for the database by means of a form for editing.

Design philosophy of the database in MRGIS:

- Only the information about drilling holes, such as X and Y, conversion factors, bit parameters, and seam name in which the hole is drilled, are stored in database in order to make the required storage volumes smaller and make data structure simple.

- Vector data model and 2D point data structure are selected to represent the location (X and Y) of the collar of a drilling hole. Bit position (Z) data are stored together with other machine data in a separate table from the table containing the location data of hole collars. This approach can avoid redundancy and make it easier to maintain the database.
• Only machine data are stored in database. The data derived or interpolated from machine data are calculated and stored in temporary tables only when being queried and will be deleted from the database once query is finished in order to save storage space and provide more flexibility for different sets of conversion factors and interpolation rules.

• Only the information about the collar of a drilling hole can be edited to keep data integrity.

• Database access is accomplished though ADO (ActiveX Data Object). ADO is the high-level data access architecture and the standard data access object model across Microsoft tools. MapObjects now supports ADO as a source of data for Table objects so a MapObjects Table object can be populated with data from a MS Access database (using the MS Jet 4.0 OLE DB Provider) and set up an AddRelate to join this information to a shapefile.

6.4 Data Interpolation

The core of MRGIS is its capability to allow the users to make use of roof drilling parameters to understand roof geology. What a user of MRGIS is interested in is the roof geology features interpolated from drilling parameters rather than drilling parameters themselves. So a procedure has been built in MRGIS to interpolate the roof drilling parameters into roof geology features automatically based on interpolation rules.

As mentioned above, a number of researchers have developed the rules for interpolating the roof geology features based on roof drilling parameters. Each set of rules have their advantages and limitations. The methodology developed in this research is used to estimate roof geology such as the existence and location of void/fracture and the roof rock strength based on drilling parameters in MRGIS.

6.5 Roof Geology Visualization

In production environment, hundreds of bolting holes will be drilled everyday. Providing an easy and convenient method to query and display roof geology information resulted from the interpolation of roof drilling data is another key task of MRGIS. The best way to accomplish this task is to visualize it by using maps and figures. A number of researchers have reported their studies on visualization of geological data \(^{(16)}\) - \(^{(21)}\). But, they did not provide an efficient querying method.

In MRGIS, two roof geology visualization methods are provided. The first is 2D+1D as shown in Figure 6.4. The interface is divided into 2 parts. The left parts is 2D map which is used to display the location (X and Y) of drilling hole collar. The right part
is 1D figure which is used to display geological features along Z direction of a drilling hole.

The following map tools are provided to help the user zoom in an area of interests:

- Zoom In
- Zoom Out
- Pan
- Full Extend

Also a ruler tool is provided to enable the user to measure the distance in map units on the 2D map.

To facilitate finding a specific area, MRGIS supports AutoCad DXF file format so a DXF formatted mine map can be imported into MRGIS. This DXF file is used as a means to relate the location of drilling holes to the mine map.

Before an AutoCad mine map file is imported, some preparation work may be necessary. An AutoCad mine map file may contain multiple layers which represent multiple seams and the surface. Such a file needs to be divided into multiple files so that each file contains the information for only one seam.
When being imported into MRGIS, only line and area objects in an AutoCad mine map file are imported and point objects are filtered out so that all point objects in 2D map only represent the locations of drilling holes.

Correspondingly, the locations of drilling hole collars also relate to a certain seam. Considering the fact that a large number of machine data files will be generated and imported in MRGIS every day, a routine is provided to automatically produce drilling hole map for each seam when importing machine data files is finished. This approach can keep drilling hole map up to date. The user can also create a specific drilling hole map by using different searching criteria such as drilling date or hole length.

To find a drilling hole, the user must add at least one drilling hole map into 2D map. Adding a DXF mine map is not a must but can make it much easier to determine the location of a drilling hole.

The user can use the mouse to select a hole on 2D map. The selected hole will flash and then be highlighted. The interpolation results of drilling parameters of the selected hole will be displayed in 1D figure. A message box will pop up to display geological features at a selected position by clicking on 1D figure. Meanwhile a form will pop up to display the information about the collar of selected hole.

The second visualization method is 2D + 2D. It allows the user to select multiple holes at the same time and display the results of the interpolation (Figure 6.5). Similar to 2D + 1D, the interface is also divided into two parts. The left one is also used to display 2D map but uses a smaller display area. The user can use the mouse to draw a polyline on 2D map and the geological features of all hole on or close to the polyline will be displayed on the right side if the interface.

The third visualization method is 2D + 3D. This feature can display a 2D mine map and a 3D figure that represents the results of the interpolation of roof drill holes at the same time (Figure 6.6). It allows the users to do a “virtual walk” through the underground spaces to check out roof geological properties at different locations.

6.6 Hardware

Since large datasets are processed in MRGIS, a high performance computer is needed in order to make fast operations.

MRGIS was actually test on a PC with the following hardware:

- Pentium 4 processor (2.0 GHz)
- RAM 512 MB
- 17’ Monitor, 1,152 × 864 pixels and true color were used as video settings
Figure 6.5  Roof geology visualization: 2D + 2D

Figure 6.6  Roof geology visualization: 2D + 3D
CHAPTER 7

MECHANISM AND DESIGN FOR TENSIONED BOLTING SYSTEM

7.1 Introduction

One of the main tasks of this project is to develop bolting design methodology based on the information of the in-situ roof geology predicted by MRGIS. Six types of roof bolts are currently used in the U.S. coal industry for reinforcing the entries in underground coal mines. However, in terms of mechanisms employed to reinforce the roof strata, there are only two basic types: tensioned bolt and fully grouted resin bolt.

The tensioned bolts work in different mechanisms from the fully grouted resin bolts and are believed to be effective for certain types of roof strata. A 3-D finite element model using ABAQUS has been developed for the tensioned bolting design. In this model, the physical process of the tensioned bolting including entry excavation sequence, roof bolting components, bolt installation procedure and pre-tension is modeled realistically. Bedding planes and in-situ horizontal stresses are also considered in the model.

Using the established model, the bedding plane study is conducted in terms of the mechanical properties and behavior as well as the effect of bedding plane location, number of bedding planes in the immediate roof, and strength sequence of immediate roof on roof stability and tensioned bolting.

Factors such as in-situ horizontal stress, overburden depth are studied to determine on how do they affect the roof stress distribution, roof yielding, roof deformation, and tensioned bolting?

Based on the analysis of mechanisms of tensioned bolts and the failure modes of tensioned bolted strata, a design procedure for tensioned bolting is proposed. The developed model can be used for tensioned bolting design to determine the bolt length, optimum pre-tension, bolt diameter, and bolt spacing.

According to the bedding plane locations, the roof is classified into 4 types. Based on the geological condition of the Pittsburgh seam, some design guidelines are given. Finally, a computer program is developed for tensioned bolting design using the data from numerical modeling. The results such as yield zone, roof deformation, bolt load increase and stresses around the entry from numerical modeling are built into a database. The program first gets the geological inputs from users, searches the design information in the database, performs the analysis using the design criteria, and finally displays the design by 2-D/3-D views.
7.2 Fundamental Study

The modeling of roof bolting is more complicated than that of other geomechanical problems, especially when bedding planes and non-linearity are considered. First of all, in order to determine how to model the tensioned bolts installed in layered roof and the effect of several geological features and factors, the basic model was developed and analyzed using ABAQUS.

7.2.1 Numerical Modeling

7.2.1.1 Global Model and Sub-model

The finite element models for underground structures are often huge in size in order to include the gravitational and tectonic loading to reduce the boundary condition effects. Although computers with high speed and large memory are readily available now, the model size is still a concern when dealing with coal mine structures. ABAQUS uses a sub-model linked with a global model by displacement to reduce the model size for the analysis of local response. In this study, a global model is used to analyze the overall response of rock mass while a sub-model is to analyze the local response around the entry. The size of sub-model is so chosen that it is beyond the influence of the stress concentration zone around the entry. Symmetrical model with half of the entry is also used to reduce the model size.

Since this study focuses on the mechanisms of tensioned bolt installed in the layered roof, a single entry during development and a typical geological column (Figure 7.1) for the U.S. Eastern coal field are used for basic analysis. A global model with the dimension of $200 \times 50 \times 150$ ft is set up to simulate the stress and displacement field around the entry as shown in Figure 7.2 (a). The sub-model is taken from the central area of the global model with the dimension of $40 \times 4 \times 75$ ft as shown in Figure 7.2 (b). The global model is elastic while the sub-models are elastic-plastic.

The rock and bolt mechanical properties used in the modeling are listed in Table 7.1. The rock strength used in the modeling is one fourth of the laboratory-determined uniaxial compressive strength. Four sub-models from the global model are set up to simulate the bedding plane and bolt effects as shown in Table 7.2

7.2.1.2 Modeling of Bedding Plane

The mechanical properties of bedding planes that are important to roof stability are the low or zero

![Figure 7.1 Geological column used in the models](image-url)
Figure 7.2  Global model and sub-model

Table 7.1 Mechanical properties for rock and bolt

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Young’s Modulus ($\times 10^6$ psi)</th>
<th>Poisson’s Ratio</th>
<th>Uniaxial Compressive Strength (psi)</th>
<th>Strength Used in Models (psi)</th>
<th>Unit Weight (lb/ft$^3$)</th>
<th>Internal Friction Angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>2.95</td>
<td>0.10</td>
<td>13,000</td>
<td>3,250</td>
<td>165</td>
<td>30</td>
</tr>
<tr>
<td>Claystone</td>
<td>1.52</td>
<td>0.27</td>
<td>2,400</td>
<td>600</td>
<td>165</td>
<td>28</td>
</tr>
<tr>
<td>Shale</td>
<td>0.71</td>
<td>0.34</td>
<td>4,800</td>
<td>1,200</td>
<td>162</td>
<td>24</td>
</tr>
<tr>
<td>Grayshale</td>
<td>0.71</td>
<td>0.34</td>
<td>4,800</td>
<td>1,200</td>
<td>162</td>
<td>24</td>
</tr>
<tr>
<td>Siltyshale</td>
<td>0.71</td>
<td>0.34</td>
<td>4,800</td>
<td>1,200</td>
<td>162</td>
<td>24</td>
</tr>
<tr>
<td>Coal</td>
<td>0.22</td>
<td>0.25</td>
<td>3,600</td>
<td>900</td>
<td>87.6</td>
<td>28</td>
</tr>
<tr>
<td>Steel</td>
<td>29</td>
<td>0.30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 7.2 Factors considered in the four models

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Bolt Installed</th>
<th>Bedding Planes Activated</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>2</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>3</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>4</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>
tensile strength in the direction normal to the bedding planes and the relatively low shear strength of the surfaces. Sliding and separation are the two phenomena caused by the bedding planes.

The bedding plane can be simulated by a contact interface in ABAQUS. The interaction between two opposite surfaces of a bedding plane consists of two components (Figure 7.3 (a)): normal force and shear force. The normal force acts on the bedding planes when they are in contact with each other. The shear force is the frictional force created to resist sliding between the two opposite surfaces of a bedding plane. Separation occurs when the normal force becomes zero or tensile and the constraint is removed. The Coulomb friction model is used to describe the interaction between the opposite surfaces of a bedding plane (Figure 7.3 (b)). The model characterizes the frictional behavior in the bedding planes using a coefficient of friction, $\mu$. The product $\mu p$, where $p$ is the normal force between the two opposite surfaces of the bedding plane, defines the limiting frictional shear stress for the bedding plane. The two opposite surfaces of the bedding plane will not slide on each other until the shear force across their interface equals the limiting frictional shear force, $\mu p$. Two bedding planes are considered in the modeling with the coefficient of friction in the bedding plane being 0.3 as shown in Figure 7.2 (b).

7.2.1.3 Modeling of Bolt

The bolt is modeled by a three-dimensional beam with 1.5 ft long resin anchor on the upper end and a $6 \times 6 \times 1/4$-in bearing plate on the lower end. The resin anchor is tied to the 1-3/8-in borehole so
that the anchor and the surrounding rock deform together without slippage, and the bearing plate is in contact with the roof surface by a contact surface. The bolts are 8-ft long and 3/4-in in diameter. The pre-tension is simulated by an assembly load so that the bolt will act like a fastener. As shown in Figure 7.4, a pre-tension section is defined on a cross section of the bolt and a pre-tension node defined as a reference to adjust the length of the bolt to achieve the prescribed amount of pre-tension. In the subsequent steps during the modeling process, further change in the length is prevented so that the bolt acts as a standard, deformable beam responding to the load by roof displacement.

7.2.1.4 Horizontal Stress
The horizontal stress is more important for controlling roof stability than the vertical stress because the vertical stress is born by the pillar while the roof must bear nearly all the horizontal stress. In the U.S., the horizontal stress is larger than the vertical stress with the ratio of horizontal to vertical stress ranging from one to three, and the maximum horizontal stress is about 40% greater than the minimum one (Mark and Barczak, 2000)\(^{(22)}\). It is not possible in numerical modeling for this ratio to reach to one only by the Poisson’s effect of the gravity load. So an initial horizontal stress is applied to the model to meet the desired in-situ horizontal stress level.

7.2.1.5 Roof Failure Criterion
The roof is assumed to behave elastically before yielding and plastically after yielding as defined by the Mohr-Coulomb failure criterion. The Mohr-Coulomb failure criterion assumes that the rock mass strength is defined by the cohesive strength and the internal angle of friction. As shown in Figure 7.5, the linear relationship between the major and minor principal stresses for the Mohr-Coulomb criterion is

\[
\frac{\sigma_1}{2c \cos \phi} - \frac{\sigma_3}{2c \cos \phi} = 1
\]  

(7.1)

![Figure 7.5 Mohr-Coulomb failure criterion](image)
where, $\sigma_1, \sigma_3 = \text{major and minor principal stress, respectively, } \phi = \text{internal angle of friction, } c = \text{cohesion of the material, which can be determined by the uniaxial compressive strength.}$

The cohesion of the material $c$ can be determined by the uniaxial compressive strength and using the following equation:

$$
\sigma_c = \frac{2c \cos\phi}{1 - \sin\phi} 
$$

(7.2)

### 7.2.2 Results and Discussion

#### 7.2.2.1 Stress Distribution around the Entry for the Four Models

Four sub-models as shown in Table 7.2 were run using ABAQUS and the stress distributions around the entry were obtained for analysis of the effects of bedding planes and roof bolting.

Figure 7.6 shows the vertical stress distribution for the four models. It can be seen that the installation of bolts only causes a local change in the vertical stress around the bolt in that compressive zones are generated at the two ends of the bolt. The existence of bedding planes has significant effect on the vertical stress distribution over the central area of the roof. Because of the sliding and separation, the vertical stress is almost zero over an arch shaped zone above the center of the entry, i.e. roof layers in this area do not take load from the overlying strata.

Figure 7.7 shows the horizontal stress distribution for the four models when the ratio of horizontal to vertical stress is one. It is obvious that the horizontal stress with bedding planes is greatly different from that without bedding planes. The horizontal stress redistributed extensively due to sliding along the bedding planes. The whole first layer of the roof is subjected to large horizontal stress concentrations although the horizontal stress near the entry corner is larger. If the first layer is not strong enough, it may result in compressive failure. Again the effect of roof bolting is restricted to around the anchors at both ends.

Figure 7.8 shows the shear stress distribution for the four models. The shear stress around the entry corners decreases due to sliding in, and separation of the bedding planes. This decrease in shear stress reduces the risk of shear failure around the entry corners.

#### 7.2.2.2 Roof Deflection

Figure 7.9 shows the roof deflection for the four models. Deflections for model 1 and model 2 are almost the same, but the roof deflections are larger when the effects of the bedding planes are considered. The installation of bolts in model 4 reduces some amount of roof deflection around the entry center by increasing the stiffness of the bolted strata.
Figure 7.6  Vertical stress distribution over the entry
Figure 7.7  Horizontal stress distribution over the entry

(a) Model 1  
(b) Model 2

(c) Model 3  
(d) Model 4
Figure 7.8  Shear stress distribution over the entry
7.2.2.3 Failure Modes of the Bolted Roof.

Figure 7.10 shows the yield zones for different thickness of the first roof layer for model 4 when the ratio of horizontal to vertical stress is one. When there are no bedding planes in the roof, the yield zone is only developed around the entry corner (Figure 7.10 (a)). If there are bedding planes in the roof, the range of yield zones goes deeper and wider depending on the location of the bedding planes. If the roof layer near the entry is thin, the whole layer yields (Figure 7.10 (b)). When the thickness of the first layer reaches to that shown in Figure 7.10 (d), yielding extends to around the entry corners and the upper portion of the layer. Figure 7.11 shows the yield zone over the roof when the ratio of horizontal to vertical stress increases to two and three. The yield zones go deeper and wider into the roof when the horizontal stress level is larger. From these results, it can be concluded that the failure zone in the roof depends on the locations of bedding planes, horizontal stress level and roof strength, and that three possible failure modes for the tensioned bolted roof can be generalized in Figure 7.12. It is very important that the possible failure modes for a given bolted roof strata must be known before a proper roof bolt design can be made.

7.2.2.4 Sliding and Separation

Figure 7.13 shows the bedding plane sliding and separation for model 3 and model 4. Sliding occurs along all the bedding planes in the models. Separation occurs only in the lower bedding plane in model 3 whereas only the upper bedding plane shows separation in model 4 because the tensioned bolts have clamped the lower bedded layers together. It can be seen that sliding extends into the area above the pillar while bed separation only occurs above the center of the entry. Both sliding and separation have negative effect on the roof stability. Sliding can change the stress distribution around the
Figure 7.10  Yield zone for different thickness of the first roof layer for model 4

(a) no bedding plane
(b) First layer = 1 ft
(c) First layer = 2 ft
(d) First layer = 4 ft
Figure 7.11  Yield zone when horizontal stress ratio is three and two for model 4

Figure 7.12  Possible failure modes of tensioned bolted roof
entry and reduce the roof stiffness. Sliding also causes the roof layer near the entry to be subjected to more horizontal stress which may result in failure. When separation occurs in a bedding plane the mechanical interaction between the opposite side of the bedding plane has been lost, and the lower layer needs only to support its own weight, not the weight of the overlying layers. The sliding zone over the entry can be determined by the shear strength of the bedding plane:

\[ \tau > \sigma \tan \phi \]  

(7.3)

where, \( \tau \) = shear stress on the bedding plane, \( \sigma \) = normal stress on the bedding plane, \( \phi \) = angle of friction on the bedding plane.

Using this criterion, the sliding zones under different overburden depth are defined in Figure 7.14 and the sliding zones under different horizontal stress ratio when overburden depth is 500 ft is shown in Figure 7.15. If there are bedding planes within the sliding zone, sliding along these planes will occur. But outside the sliding zone, no sliding will occur even if bedding planes exist. It is interesting to note that regardless of the magnitude of horizontal stress and overburden depth, the range of sliding zone is only within about 11 ft above the entry.

7.2.2.5 Compressive Zone by Bolt Tension

Figure 7.16 shows the compressive stress zones under different pre-tension. Near the two ends of the bolt, there are large compressive stresses, but they decrease rapidly away from the ends. A small amount of compressive stress (about 10 psi) remains to maintain friction in the bedding planes and add vertical confinement to the bolted strata. It should be noted that a small tension zone is formed above the top anchor when the pre-tension is applied. It is possible that tension cracks may be generated in this area.
The stress distribution created by the pre-tension of the bolts does not cover the whole bolted horizon; there is a triangle zone between adjacent bolts near the roofline that is beyond the effect by the bolt pre-tension. This area is subjected to compressive failure by horizontal stress if the roof layer near the entry is weak.

7.2.2.6 Effectiveness of Tensioned Bolt Support for the Layered Roof

From the above simulation results and analysis, the effect of the tensioned bolt support can be analyzed as follows:
When an entry is developed, the stress around the entry is redistributed. The bedding planes cause the horizontal stress to concentrate more in the first layer of the roof and to form an arched zone over the entry with little vertical stress due to sliding in, and separation of, the bedding planes. A yield zone is developed in the roof due to high horizontal compression but low vertical confinement. Because the rock in the yield zone is in its post failure region, further deflection of the roof will make the roof even weaker and easy to fall. The tensioned bolt is to provide vertical confinement to the yield zone. This confinement has three effects: to reduce the deflection of the roof by increasing the friction in bedding planes; to increase the residual strength of the failed roof; and to maintain the horizontal stress in the first layer for the reason that if the horizontal stress in the first layer decreases by roof deflection, the horizontal stress will be transferred to the upper layer causing more roof to fail.

In order to provide a good confinement, the bolt should cover the yield zone as far as possible and an effective connected compressive zone should be formed over the roof by proper pre-tension and bolt spacing. The longer the bolt, the larger the pre-tension should be applied in order to create an effective compression zone in the roof because the weight of the roof needs to be overcome before the bolt pre-tension becomes effective. The tensioned bolts can provide little compression to the triangle zones between two adjacent bolts at and near the roofline level and the two entry corners no matter how high the pre-tension is. Therefore, if the first roof layer near the entry is weak and the horizontal stress is large, roof failure in these triangle areas may occur.

The tensioned bolt is more effective when the immediate roof layers are weak but the horizontal stress level is low.

7.2.3 Effect of Bedding Plane

7.2.3.1 Single Bedding Plane

In order to see the bedding plane effect, a single bedding plane over the immediate roof is considered. The models with 400 ft overburden depth and horizontal-to-vertical stress ratio of three were run with bedding plane location from 2 ft to 14 ft above the roof line.

Figure 7.17 shows the vertical stress distribution over the entry for different bedding plane location. It can be seen that the vertical stress reduces to low values over the entry due to the bedding plane effect. However, the low vertical stress zone over the entry has to do with bedding plane location. It is shown that if 50 psi is chosen as a boundary, the low vertical stress zone is within 8 ft from the roofline when the bedding plane is less than 5 ft from the roofline. However, it reduces to about 4 ft from the roofline when the bedding plane is more than 5 ft from the roofline. Therefore, the vertical stress in the immediate roof reduces to low values when there is a bedding plane near the roofline. This vertical stress release is caused by bedding plane sliding and separation. The low vertical stress in the immediate roof means that the roof will be easier to yield under horizontal stress as a result of lower vertical confinement.
Figure 7.17  Vertical stress distribution for different bedding plane locations
Figure 7.18 shows the horizontal stress distribution for different bedding plane locations. It is shown that the existence of the bedding plane in the immediate roof causes the horizontal stress to concentrate on the first roof layer when the bedding plane is within 4 ft above the roofline (Figure 7.18 (a)-(c)). However, the horizontal stress is more concentrated on the mid-upper portion of the first roof layer when the bedding plane is 5-8 ft above the roofline (Figure 7.18 (d) and (e)). When the bedding plane is more than 8 ft above the roofline, the horizontal stress is almost not affected (Figure 7.18 (f) and (g)). Moreover, the stress concentration in the immediate roof is largely affected by the bedding plane location. It is seen that the stress concentration within 2-3 ft of the immediate roof decreases from 2,200 psi to 1,400 psi as the bedding plane locations change from 2 ft to 14 ft above the roofline. Since the immediate roof is mainly subject to horizontal load, the bedding plane location is important in determining the loading condition of the immediate roof.

Figure 7.19 shows the shear stress distribution for different bedding plane locations. It can be seen that the bedding plane causes the shear stress to distribute more extensively over the entry. But the shear stress zone is reduces as the bedding plane goes higher above the roofline. When the bedding plane location is larger than 8 ft from the roofline, there is no effect of the bedding plane on the shear stress distribution. The shear stress is reduced along the bedding plane due to shear sliding, making the shear stress discontinuous across the bedding plane.

In order to see the details of the stress concentration in the immediate roof, the maximum values of the vertical, horizontal and shear stress together with the average horizontal stress in the immediate roof are taken from the models with different bedding plane locations as shown in Table 7.3.

The maximum vertical stress occurs on the entry corners over the pillar and increases as the distance of the bedding plane above the roofline increase. The vertical stress concentration in the roof over the pillar may not affect the roof stability if the pillar is stable under the high vertical load. The maximum shear stress occurs on the entry corners, which increases as the bedding plane is located higher above the roofline. The bedding plane sliding makes the shear stress distribute over a larger area and reduce its value on the entry corners. Therefore, if there are no bedding planes in the immediate roof, the entry corners are more likely to have shear type failure if the immediate roof has lower shear strength. The maximum horizontal stress also occurs on the entry corners but decreases as the bedding plane horizon increases. The average horizontal stress within 3 ft of the immediate roof decreases with the increase of the bedding plane horizon. So, if there is a bedding plane less than about 3 ft from the roofline, the first roof layer will be subjected to large horizontal stress concentration, which can affect the roof stability.

Figure 7.20 shows the sliding of bedding plane under different locations. It can be seen that the sliding dramatically decreases as the bedding plane is more than 5 ft above the roofline. When the bedding plane is more than 8 ft above the roofline, the sliding becomes very small. That is why a bedding plane close to the roofline has a larger effect on the stress distribution over the immediate roof than those higher above the roofline.
Figure 7.18  Horizontal stress distribution for different bedding plane locations
Figure 7.19  Shear stress distribution for different bedding plane locations
Table 7.3  Stress vs. bedding plane location

<table>
<thead>
<tr>
<th>Bedding plane distance from the roofline (ft)</th>
<th>Max. vertical stress (psi)</th>
<th>Max. shear stress (psi)</th>
<th>Max. horizontal stress (psi)</th>
<th>Average horizontal stress within 3 ft of the immediate roof</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>614</td>
<td>594</td>
<td>2,947</td>
<td>2,200</td>
</tr>
<tr>
<td>3</td>
<td>682</td>
<td>657</td>
<td>2,991</td>
<td>2,00</td>
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<td>4</td>
<td>751</td>
<td>707</td>
<td>2,982</td>
<td>1,800</td>
</tr>
<tr>
<td>5</td>
<td>817</td>
<td>744</td>
<td>2,904</td>
<td>1,600</td>
</tr>
<tr>
<td>8</td>
<td>910</td>
<td>762</td>
<td>2,754</td>
<td>1,500</td>
</tr>
<tr>
<td>10</td>
<td>913</td>
<td>760</td>
<td>2,731</td>
<td>1,500</td>
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<tr>
<td>14</td>
<td>912</td>
<td>757</td>
<td>2,716</td>
<td>1,500</td>
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</table>

Figure 7.20  Sliding of bedding plane under different locations

Figure 7.21  Bedding plane location vs. bolt load change
Figure 7.21 shows the bolt load changes after installation as the bedding plane location changes. It can be seen that the bolt load change is larger when the bedding plane is close to the roofline than when the bedding plane is about 3-4 ft above the roofline. The change in bolt load is smaller when the bedding plane is less than 3 ft or more than 4-5 ft above the roofline. It is also seen that the bolt load changes on the side bolt has the same trend as on the center bolt, but the center bolt is subjected to more load than the side bolt.

7.2.3.2 Multiple Bedding Planes

Figure 7.22 shows the stress distribution over the entry for the model with two bedding planes: one is 2 ft and the other is 5 ft above the roofline. Figure 7.22 (a) shows a significant vertical stress reduction over the middle of the entry. The height of the vertical stress relief zone goes up to 10 ft above the roofline if 50 psi is taken as the boundary. Compared with Figure 7.17 (a), the model with two bedding planes has a 2 ft increase of the vertical stress relief zone caused by the second bedding plane. Figure 7.22 (b) shows that the horizontal stress is concentrated in the first layer of the immediate roof, while the horizontal stress concentration in the second layer is smaller than the first layer. Compared with Figure 7.18 (a), the second bedding plane almost has no effect on changing the horizontal stress concentration in the first layer. The shear stress is concentrated around the entry corner and more extensively distributed over the roof above the bedding planes (Figure 7.22 (c)).

Figure 7.23 shows the stress distribution over the entry for the model with three bedding planes. With the increase of the number of the bedding planes, the vertical stress relief zone is increased to within 11 ft above the roofline (Figure 7.23 (a)). The horizontal stress almost has no change on the first layer but a little change on the upper roof layers (Figure 7.23 (b)). The shear stress has a little change due to the addition of one more bedding plane (Figure 7.23 (c)).

7.2.3.3 Thinly Laminated Planes

The thinly laminated roof is modeled as multiple bedding planes within one roof layer. Figure 7.24 shows the stress distribution for the model with a 2-ft thinly laminated roof layer 3 ft above the roofline. The thickness of the laminations in the model is 0.5 ft. The vertical stress (Figure 7.24 (a)) shows a stress relief zone which is about 11 ft above the roofline. The horizontal stress (Figure 7.24 (b)) is concentrated in the first roof layer but there is no horizontal stress concentration within the thinly laminated layer. The shear stress (Figure 7.24 (c)) is concentrates on the entry corner, reduced the thinly laminated layer. Because the lamination has the same behavior as bedding plane such as sliding and separation, its effect is to reduce the vertical stress and make the horizontal stress concentrated in the immediate roof. Comparing with Figure 7.23, both stress distributions are similar. Therefore, if the thinly laminated layer is not very thick (around 3 ft), it can be modeled by two bedding planes at the upper and lower boundaries of the laminations.

In summary, bedding planes change the stress distribution significantly, especially when there is a bedding plane located near the roofline. The effect of bedding plane decreases as they are located higher above the roofline. The bedding plane 2-3 ft above
Figure 7.22  Stress distributions for the model of two bedding planes

Figure 7.23  Stress distributions for the model of three bedding planes

Figure 7.24  Stress distributions for the model of thinly laminated layer
the roofline has the largest effect on the stress distribution over the entry and the load in the bolt. From a stress distribution point of view, the model with two or three bedding planes can roughly represent the case with multiple bedding planes. The thinly laminated layer in the roof can be modeled by two bedding planes located on the lower and upper boundary of the laminations without significant change in the stress distribution of the lamination.

7.2.4 Effect of the Strata Strength Sequence

Figure 7.25 (a) and (b) show the vertical and horizontal stress distribution when the strength of the first layer is 1,800 psi and the strength of the second layer is 750 psi. It can be seen that the vertical stress is not reduced to low values around the first bedding plane because the second layer lays on the first layer. Figure 7.25 (c) and (d) show the vertical and horizontal stress distribution when the strength of the first layer is 1,200 psi and the strength of the second layer is 750 psi. It can be seen that the vertical stress is reduced to low values around the first bedding plane and the pressure applied by the second layer to the first layer is small. The plastic deformation of the immediate roof, which is determined by its strength and the bedding plane separation in the immediate roof when a weak layer lays above a strong layer in the immediate roof. Hence, the stress distribution over the entry and bedding plane separation is largely affected by the strata strength sequence and the plastic deformation of the immediate roof.

7.3 Tensioned Bolting Design Using Numerical Modeling

The weak planes in the immediate roof near the roofline are important to the roof stability because their sliding and separation can significantly affect the stress distribution in the immediate roof. The analysis results of previous section show that if bedding planes exist within 3 ft of the immediate roof, the first layer is important to the roof stability and the effect of tensioned bolt. In this section, the behavior of the immediate roof with the bedding plane both within 3 ft of the immediate roof and 3 ft above the roofline is focused in regarding to the roof stability and the effect of the pre-tension as to develop the guideline for tensioned bolting design. If there are no bedding planes in the immediate roof, the entry corners are critical to the roof stability and they are also analyzed.

7.3.1 Numerical Modeling

A typical geological condition in the Pittsburgh seam with overburden depth of 600 ft is used to model the effect of tensioned bolts (Figure 7.26). The mechanical properties of rock, resin grout, and steel used in the model are shown in Table 7.4. In order to simulate the effects of entry development and longwall mining, the global model consists of a 3-entry system and the longwall panels on both sides of the 3-entry system. The size of the global model is $1,228 \times 1,214 \times 630$ ft with 18 ft wide entries and 60 $\times$ 120 ft chain pillars. The excavation sequence in the global model is simulated in three steps: solid panel without excavation in the 1st step (Figure 7.27 (a)), entries developed
Figure 7.25  Stress distribution when the first layer is stronger than the second layer of the roof
Figure 7.26 Geological column and bolt layout

Figure 7.27 Global model and excavation sequence for tensioned bolt system

Table 7.4 Mechanical properties for rock, resin grout and bolt

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Young’s Modulus ($\times 10^6$ psi)</th>
<th>Poisson’s Ratio</th>
<th>Uniaxial Compressive Strength (psi)</th>
<th>Strength Used in Models (psi)</th>
<th>Unit Weight (lb/ft$^3$)</th>
<th>Internal Friction Angle (°)</th>
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</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>2.95</td>
<td>0.15</td>
<td>13,000</td>
<td>3,250</td>
<td>167.0</td>
<td>35</td>
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<tr>
<td>Sandyshale</td>
<td>1.00</td>
<td>0.20</td>
<td>10,000</td>
<td>2,500</td>
<td>160.0</td>
<td>33</td>
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<tr>
<td>Claystone</td>
<td>0.80</td>
<td>0.27</td>
<td>2,400</td>
<td>600</td>
<td>155.0</td>
<td>28</td>
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<tr>
<td>Blackshale</td>
<td>0.30</td>
<td>0.27</td>
<td>3,000</td>
<td>750</td>
<td>150.0</td>
<td>29</td>
</tr>
<tr>
<td>Grayshale</td>
<td>0.68</td>
<td>0.27</td>
<td>4,800</td>
<td>1,200</td>
<td>150.0</td>
<td>29</td>
</tr>
<tr>
<td>Siltshale</td>
<td>0.68</td>
<td>0.27</td>
<td>4,800</td>
<td>1,200</td>
<td>150.0</td>
<td>29</td>
</tr>
<tr>
<td>Coal</td>
<td>0.36</td>
<td>0.34</td>
<td>3,600</td>
<td>900</td>
<td>87.6</td>
<td>28</td>
</tr>
<tr>
<td>Limestone</td>
<td>2.50</td>
<td>0.30</td>
<td>10,000</td>
<td>2,500</td>
<td>165.0</td>
<td>31</td>
</tr>
<tr>
<td>Resin Grout</td>
<td>0.435</td>
<td>0.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>29.0</td>
<td>0.30</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>
and bolts installed with the face 24 ft in by the bolt location in the 2nd step (Figure 7.27 (b)), and entries fully developed in the 3rd step (Figure 7.27 (c)). The sub-model is used to simulate the details around the entries. The sub-model for the center entry is taken from the middle of the global model following the corresponding excavation sequence in the global model (Figure 7.28). The size of the sub-model is 98 × 4 × 75 ft with two bedding planes.

Since in practice the bolts are installed after the entry is driven forward for a distance of 10 to 40 ft from the face, the time lapse between roof exposure upon undermining and bolt installation in the model is important in order to obtain the true effect of the tensioned bolts. Figure 7.29 shows the maximum roof displacement developed in the middle of the entry as the face advances. It is shown that more than 80% of the roof deformation has already developed when the bolts are installed 24 ft out by the face. If no time dependent deformation is involved, only a small amount of load will be
induced in the bolts by subsequent roof deformation and the bolt loads will remain constant during the development stage. In this model, the bolts are installed 24 ft outby the face to simulate the standard 20-ft cut.

The horizontal-to-vertical-stress ratios are 3 and 2.41 across and along the entry direction, respectively. Since almost all the roof strata in coal mines are in layers, all the models studied have bedding planes. Model with and without bolts are used to study the effect of tensioned bolts.

7.3.2 Mechanism of Tensioned Bolt

7.3.2.1 Stress Distribution around the Entry after Excavation and Bolt Installation

When an entry is developed, the stresses around the entry are redistributed. The installation of tensioned bolts causes local stress concentrations around the two ends of the bolts. The characteristics of stress distribution around the entry can be summarized as follows:

The vertical stress over the entry is reduced to very low values due to bedding plane sliding and separation. The pre-tension produces a vertical compressive zone within 2-3 ft above the bearing plate. The horizontal stress is concentrated on the first layer of the roof while the shear stresses around the entry corners. But the shear stress distributions across the bedding planes are discontinuous.

Figure 7.30 (a) shows that small vertical tensile stresses are induced in the first roof layer by the horizontal stress. After the bolts with 10-ton pre-tension are installed, the vertical tensile stress changes to compressive stress by the pre-tension (Figure 7.30 (b)). If the immediate roof is weak and thinly-laminated the vertical tensile stress prior to bolt installation may cause the immediate roof to become unstable.
7.3.2.2 Effect of Pre-tension

Figures 7.31 and 7.32 show the vertical stress distributions caused by pre-tension using the resin and mechanical anchors, respectively. In order to see the details of the stress distributions induced solely by the pre-tension, the effects of overburden and horizontal stress are not included. It can be seen that for both types of anchors the compressive zones near the bearing plate is about 2 ft from the roof line while the compressive zones near the top anchor is about half of that near the bearing plate. A tensile stress zone is also developed at the upper end of, and above, the anchor. The compressive and tensile stress zones increase with the increase of the applied pre-tension.

Figure 7.31 Vertical stress distribution by pre-tension for mechanical anchor

Figure 7.32 Vertical stress distribution by pre-tension for resin anchor
When the pre-tension is increased from 5-ton to 10-ton the compressive zones near the bearing plates are increased from 1.5 ft to 2.5 ft from the roof line if 10 psi is used as the cut-off stress. It should be pointed out that the compression induced by the pre-tension near the top anchor is so small as compared to the vertical pressure due purely to rock weight in this area that it plays little role in affecting roof stability. The difference between the resin anchor and mechanical anchor is that the tension zone around the resin anchor is distributed all along its length, while in the mechanical anchor it concentrates on its top and consequently its magnitude is much larger. If the rock in the anchorage horizon is not sufficiently strong tension cracks could be generated on the top of the mechanical anchor. Figure 7.33 shows the shear stress distribution along the interface of a 2-ft resin anchor subject to different applied pre-tensions. The maximum shear stress occurs near the anchor end where the pre-tension is applied and the shear stress reduces rapidly towards the other end of the anchor. Most of the shear stress distributed around the first 1-foot of the anchor, beyond which the shear stress is very small. Therefore, the resin anchor should be more than 1-foot long depending on the strength of the rock in which the bolts are anchored.

The pre-tension can close a bedding plane separation near the roof line. Figure 7.34 (a) shows a bedding plane separation located at 1-ft above the roof line. When a 5-ton pre-tension is applied the separation is partially closed (Figure 7.34 (b)). The separation is completely closed when a 10-ton pre-tension is applied (Figure 7.34 (c)).

Closing a separation has a significant impact on roof stability. First of all, closing a separation means the roof layers on two sides of the bedding plane are in contact, which will generate a frictional force to enhance the beam building effect. Most importantly, when a separation is closed any subsequent vertical roof displacement is resisted by the load which is at least as much as the amount of pre-tension.

Whether a separation can be closed or not depends on its location and magnitude, the horizontal stress level surrounding it, roof strength and applied pre-tension. The
(a) Bedding plane separation without bolts

(b) Partially closed bedding plane separation by pre-tension

(c) Closed bedding plane separation by pre-tension

Figure 7.34  Separation closure by pre-tension

Figure 7.35  Diagram to calculate the minimum required pre-tension to close a separation
minimum pre-tension required to close a separation near the roof line can be estimated by a simplified beam with the loading conditions taken from the model (Figure 7.35). The first layer of the immediate roof is separated from the upper roof and subjected to both horizontal stress and gravity load. In order to hold this layer up by pre-tension, the rotation moment by the pre-tension should be greater than that by the horizontal stress and gravity load. The following equation was derived to estimate the minimum required pre-tension:

\[ (a_1 + a_2)T > \frac{w}{4} \left( \frac{w}{2} h \rho r + \sigma_h s \right) hr \]  

(7.4)

where, \( a_1 = \) distance from the rib to side bolt (ft), \( a_2 = \) distance from the rib to center bolt (ft), \( T = \) pre-tension (lbs), \( w = \) entry width (ft), \( h = \) thickness of the first layer (ft), \( \rho = \) rock density (lb/ft\(^3\)), \( r = \) bolt row spacing (ft), \( s = \) roof displacement at the center of the entry (ft), and \( \sigma_h = \) horizontal stress on the first layer (lbs/ft\(^2\))

Equation (7.4) can be rewritten as:

\[ T > \frac{\left( \frac{w^2}{8} \rho + \sigma_h s \right) hr}{a_1 + a_2} \]  

(7.5)

Since the maximum roof displacement and horizontal stress on the first roof layer can be obtained by the results of modeling, this formula can be used to estimate the minimum required pre-tension to close a separation near the roof line.

### 7.3.3.2 Mechanisms of Tensioned Bolts

After an entry is excavated the stresses are redistributed and concentrated around the entry corners and first layer of the roof strata. As a result, yielding zones are developed over the entry depending on rock strength, stress level and number and location of bedding planes. Since the vertical stress in the immediate roof has been largely released over the entry, the horizontal and shear stresses are the two factors that affect the development of yielding zone, and thus roof stability. The yielding zone is in the post failure region and its residual strength is dependent on roof displacement and confinement. The role of the bolts is to increase the self-supporting capacity of the yielding zone so that the rock in the yielding zone will have the ability to take the load from, and limit the deformation, of the upper strata. Fractures, cracks and separations may exist in the yielding zone. The residual strength of the yielding zone depends on the friction along these failure planes, which is determined by the contact surface area and the normal pressure on the surface. Therefore in order to increase the residual strength of the yielding zone, the fractures and separations within it must be made to contact each other as much as possible.

The tensioned bolts are subjected mainly to the axial load. Figure 7.36 shows the amount of sliding along the bedding plane 3 ft from the roof line for side and center bolts under different roof displacements at the center of the entry. Obviously, the bedding
plane sliding at the side bolt is larger than that at the center bolt. When the vertical roof displacement is larger than 0.75-in, the sliding at the side bolt location is larger than 0.5-in, making the bolts subject to shear load if the diameter of the bolt is larger than 7/8-in in a 1-3/8-in borehole. However, it should be realized that the center bolts are already in the yielding state after 0.75-in of vertical roof displacement. Since the tensioned bolts are designed not to go to the yielding state, the amount of bedding plane sliding induced through vertical displacement alone is too small for the bolts to be subjected to the shear load. The vertical confinement provided by the tensioned bolts is the result of the suspension effect. However, the bolts not only suspend the gravity load but also those caused by rock expansion and bending of the roof layers under the horizontal stress. Figure 7.37 shows the increased load in the center bolt after the entry is fully developed under different horizontal stress levels. Apparently, the load developed in the bolts after installation is largely controlled by the horizontal stress. This is evident in the findings by Signer et al.(1993) (23) that the measured loads on the roof bolts are often twice as much as that predicted by the dead weight load.
The tensioned bolts provide the compressive zones in the immediate roof within 2 ft of the roof line. These vertical compressive zones can generate frictional forces along the fractures and bedding planes near the bearing plates providing shear resistance for the rock in this area. This effect can be called beam building, but it is only in effect within about 2 ft of the immediate roof. Although the vertical compression near the bearing plate is very small this low confining pressure can generate significant friction according to the findings by Barton (1973) \(^{(24)}\) that the friction angle along the joints increases up to 70° when the normal pressure on the joints is very small.

The compression by the pre-tension also increases greatly the residual strength of the rock in the compressive zone. Gale (1986) \(^{(25)}\) showed that the confining pressure can increase the residual strength of the failed rock markedly, up to 20-30 times of the average confining pressure.

The pre-tension can close the cracks and separations in the immediate roof, which not only increases the strength of the affected roof layers but also the stiffness of the whole bolted strata because any subsequent vertical displacement in the bolted strata is resisted by at least the same amount of load as the pre-tension.

Bolt load is generated mainly by the vertical roof displacement which in turn is caused mainly by the horizontal stress or other time-dependent loading. The load in the bolted strata is transferred from a weak layer to a strong layer through roof displacement. If the weak roof layer is immediately above the strong layer, it tends to have more deformation and transfers its horizontal load to the lower layer by applying vertical pressure on it and taking less of the horizontal stress, i.e. the lower layer will take more of the horizontal stress. On the other hand, if the weak layer is immediately below the strong layer, it tends to separate from the upper layer and leaves the upper layer to take more of the horizontal stress. All the roof layers in the bolted strata interact each other so that finally the load is transferred from the first roof layer to the bearing plate which in turn transfers it to the top anchor through the steel rod. Because all the bolt loads are taken by the bolt anchors, it is important to anchor the bolts in a stable roof layer.

In summary, the tensioned bolts support the bolted strata by reinforcing the 2-3 ft of the immediate weak roof and provide a high resistance to subsequent roof displacement through the pre-tension.

7.4 Failure Analysis of Tensioned Bolted Strata and Design Considerations

7.4.1 Failure Modes from Field Observation

From field observation, the failure modes of the bolted strata can be classified into three groups according to the location and extent of the failure: failure around the middle of the entry (Figures 7.38 (a) and (b)), failure in the whole bolted strata (Figure 7.38 (c)), and failure around the entry corners (Figure 7.38 (d)).
Figure 7.38  Failure modes of bolted roof

Figure 7.39  Potential failure modes of bolted roof
However, according to the causes and mechanisms of failures, the modes of failures can further be classified into five groups: skin failure (Figure 7.38 (a)), buckling failure (Figure 7.38 (b)), shear failure (Figure 7.38 (c)), and cutter roof failure (Figure 7.38 (d)). These failure modes can be summarized as shown in Figure 7.39.

7.4.2 Skin Failure

The tensioned bolts produce compressive zones near the bearing plate but a triangular zone between the bolts and some areas around the entry corners are not affected (Zhang and Peng, 2001) (26). If the immediate roof is very weak, the skin failure in these unaffected areas may occur due to compression and/or weathering (Figure 7.39 (a)). In this case, the tensioned bolts should be used in conjunction with straps or wire meshes.

7.4.3 Buckling Failure

The buckling failure occurs when the roof lose stability with the load not exceeding its strength. It is caused by horizontal compression and is likely to occur between bolts around the middle of the entry when the immediate roof is thinly laminated and sliding occurs between laminations. It can be analyzed by taking the laminations in the immediate roof as beams with fixed ends. The beam length is the spacing between the bolts across the entry. The criterion for buckling failure can be derived as

$$\sigma_h = \frac{\pi^2 E J}{b h (a/2)^2} \quad (7.6)$$

Where, $\sigma_h$ = horizontal stress in the immediate roof (psi), $E$ = Young’s modulus of the immediate roof (psi), $J$ = Moment of inertia of the lamination beam, $b$ = width of lamination (in), $h$ = thickness of lamination (in), $a$ = beam length which is the spacing between the bolts across the entry (in). Besides, $J$ is expressed as following equation:

$$J = \frac{b h^3}{12} \quad (7.7)$$

Equation (7.7) can be rewritten as:

$$h = \frac{a}{\pi} \sqrt{\frac{3\sigma}{E}} \quad (7.8)$$

Figure 7.40 shows the thickness of laminations below which it would cause the buckling failure under different horizontal stress levels and bolt spacing. It can be seen that the range of the thickness of laminations causing the bucking failure decreases with the reduction of bolt spacing.
7.4.4 Shear Failure

If the whole bolted strata are weak, it is likely to be sheared off along both ribs and fall down (Figure 7.39 (c)). In this case, the tensioned bolts should be designed to suspend the gravity load of the bolted strata.

7.4.5 Cutter Roof Failure

As both the shear stress and horizontal stresses are concentrated around the entry corners and shear sliding along bedding planes or laminations occurs above the entry corners, most roof failures in coal mines are cutter roof failure initiated at the entry corners (Figure 7.39 (d)). Figure 7.41 shows the stress state around the entry corners. The rock is subjected to the shear and horizontal stresses. The angle between the horizontal axis and major principal stress is

![Figure 7.40 Buckling failure criterion for laminated immediate roof](image)

![Figure 7.41 Entry corner failure](image)
\[ \theta = \frac{1}{2} \arctan \frac{\tau_{xy}}{\sigma_x} \]  

Equation (7.9)

According to the Mohr Coulomb failure criterion, the failure angle between the major principal stress and failure plane is

\[ \pi - \frac{\phi}{2} \]  

Equation (7.10)

Where, \( \phi \) = the internal frictional angle.

Hence the failure angle between the roof line and potential failure plane is

\[ \alpha = \frac{1}{2} \arctan \frac{\tau_{xy}}{\sigma_x} + \pi - \frac{\phi}{2} \]  

Equation (7.11)

According to this equation, the failure angle will develop, due to both shear and horizontal stresses, towards the center of the entry following approximately the magnitude defined by Equation 7.11. In this case, the bolt should be designed to extend beyond the failure plane to limit the roof displacement in case the failure occurs.

7.5 Proposed Design Procedures for Tensioned Bolting

Based on the above analysis, a new design approach is proposed for the tensioned roof bolting design using ABAQUS. The procedures are:

7.5.1 Determination of Bolt Length and Minimum Required Pre-tension by Running the Model without Bolts

The first step is to determine the bolt length and minimum required pre-tension. By running the model without bolts, yielding zones and maximum roof displacement are obtained. The maximum height of the yielding zone is used to determine the bolt length which is the height of the yielding zone plus anchor length. The minimum required pre-tension is estimated by Equation 7.5 using the roof displacement and horizontal stress in the first roof layer obtained from the results of modeling. If the thickness of the first roof layer is larger than 3 ft, 3-ft is used to estimate the minimum pre-tension, because the influence zone of pre-tension commonly applied is less than 3-ft above the roof line.

The bolt length is determined such that its anchor is located in an area where the rock strata are stable. The shear strength of rock in the anchorage horizon should be larger than the maximum shear stress around the anchor using the bolt’s yield strength as the maximum allowable bolt load. The tensile strength of the rock in the anchorage horizon should be larger than the maximum tensile stress around the anchor so that tension cracks around the anchor would not occur.
7.5.2 Determination of the Optimum Pre-tension by Running the Model with Bolts

The second step is to run the model with standard layout of bolts and the estimated pre-tension obtained in step 1 to check if any separations within 3 ft of the immediate roof are closed by checking if there is a normal pressure on the bedding planes. If not, a new pre-tension with 2-ton increment will be used to run the model with bolts again. This process is repeated until the bed separations are closed. The final pre-tension is the optimum pre-tension that will be used in the design. If the bedding plane is 3 ft above the roof line or no bedding planes over the bolted horizon, the pre-tension should produce a 2-ft compressive zone above the bearing plate.

7.5.3 Determination of Bolt Density by Bolt Loads

The loads in the bolts obtained from the model can be used to determine the bolt density. The load in the bolts is the combined result of the pre-tension and roof deformation, which is related to roof strength and horizontal stress level. Since the tensioned bolt is designed to work within its capacity, the designed bolt load should not exceed 75% of its capacity. If the designed bolt load is beyond this limit, the bolt density should be increased to reduce the load in the bolts.

7.5.4 Prediction of the Potential Failure Modes and Adjust the Bolt Spacing to Prevent Failures

The shape of the yielding zone and plastic strain pattern above the entry are used to predict the potential failure modes. Figure 7.42 shows the plastic strain pattern for the four failure modes. For the first failure mode, straps or wire meshes are used in conjunction with bolts. For the 2nd failure mode, Equation 7.8 is used to check if the buckling failure is possible. For the 3rd failure mode, the numbers of bolts are checked to see if they are sufficient to take the dead weight load. For the 4th failure mode, the side bolts should be adjusted to extend beyond the failure zone.

7.5.5 Verification of Proposed Design Procedures

In this section, a field case with well-instrumented sites was used to validate the newly-developed 3-D tensioned bolting design model using finite element method. The instrumentation data from an entry intersection were used to compare the difference between the results of modeling and field measurements. Finally, how the model can be used to do the tensioned bolting design is given.

7.5.5.1 Test Site Description

The instrumentation results obtained by Hanna et al (1991) (27) in a shallow underground coal mine in central Illinois were used for this study. The room-and-pillar method was used to mine a flat-lying 7-ft-thick seam at an overburden depth of 360 ft. The entries were 7 ft high by 20 ft wide, separated by 50 by 60 ft rectangular pillars.
Figure 7.42  Plastic strain and failure mode
In order to model the room-and-pillar mining system, as well as excavation sequence and bolt installation procedure, both global and submodel were used. The global model consisted of four entries simulating room-and-pillar mining as shown in Figure 7.46. The dimension of the global model was 460’×430’×180’. The global model was used three steps to simulate the excavation sequence. In the 1st step the global model was solid, i.e. before excavation begins. In the 2nd step cut 1 through 5 was made while in the 3rd step cuts 6 through 35 were made. The submodel was taken from the global model with one fourth of the cross-section include as shown in Figure 7.47. The size of the submodel was 30’×20’×86’ and two bedding planes were considered. One bedding plane was 2 ft from the roof line between the black shale and limestone and the other 8.5 ft

Figure 7.43  The location of test site and development sequence

Figure 7.43 shows the plan view of the face layout during development and the test site location. The test intersection was located in entry 2 of the eight-entry main being driven to the north. Because the objective was to investigate the stability of four-way intersections, the staggered pillar plan was modified to provide a four-way intersection for testing. As shown in Figure 7.43, the numbers 1 through 35 indicate the sequence of mining in and around the intersection. Cuts 1 through 5 were completed in one week, and mine development at test site was postponed for two weeks while most of the instruments were installed. After installation, cuts 6 through 35 took 6 days to complete and the remaining instruments were installed. The primary roof support at the instrument test site was 4 ft long mechanical anchor bolts, which had a 6 ton pre-tension applied.

The geological column of the test site is shown in Figure 7.44. There were four slip zones around the test area with one major slip across the intersection as shown in Figure 7.45. They affected the behavior of the roof strata above the instrumented intersection.
Figure 7.44 Geological column at test site (modified from Hanna, et al., 1991)

Figure 7.45 Location of slip zones at test site (modified from Hanna, et al., 1991)

Figure 7.46 Global model in plan view

Figure 7.47 Sub-model in 3-D view
from the roofline between limestone and limey shale. Twelve tensioned bolts were installed in the roof as shown in Figure 7.48. In the submodel, the bolt installation procedure was simulated by taking the boundary condition from the corresponding steps in the global model. Bolts were installed in the second step when the cuts from 1 through 5 were made so that bolt loads would be increased in the third step when the cuts from 6 through 35 were made. In addition, measured in-situ horizontal stresses, which are 1207 psi in N 65°E and 686 psi in N 25°W, were applied in both the global model and submodel as initial stresses. Each bedding plane was modeled by a contact surface with friction (Zhang and Peng, 2001)\(^{(26)}\).

The bolt was modeled physically by a three-dimensional beam with 6-in mechanical anchor on the upper end and a 6×6×1/4-in bearing plate on the lower end. The mechanical anchor was tied to the 1-3/8-in borehole so that the anchor and the surrounding rock deform together without slippage, and the bearing plate was in contact with the roof surface by a contact surface. The bolts were 4-ft long and 3/4-in in diameter. A 6 ton pre-tension was applied during bolt installation.

The Mohr-Coulomb failure criterion was used as the yielding criterion. The model was assumed to behave elastically before yielding and perfect-plastically after yielding. Cohesion and internal friction angle are the input parameters for rock strength. The rock properties used in the model is shown in Table 7.5.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Young’s Modulus (× 10^6 psi)</th>
<th>Poisson’s Ratio</th>
<th>Uniaxial Compressive Strength (psi)</th>
<th>Strength Used in Models (psi)</th>
<th>Unit Weight (lb/ft^3)</th>
<th>Internal Friction Angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>3.00</td>
<td>0.26</td>
<td>14,000</td>
<td>3,500</td>
<td>167.0</td>
<td>35</td>
</tr>
<tr>
<td>Siltstone</td>
<td>0.80</td>
<td>0.30</td>
<td>6,000</td>
<td>1,250</td>
<td>160.0</td>
<td>33</td>
</tr>
<tr>
<td>Mudstone</td>
<td>0.80</td>
<td>0.27</td>
<td>3,600</td>
<td>900</td>
<td>155.0</td>
<td>28</td>
</tr>
<tr>
<td>Blackshale</td>
<td>0.30</td>
<td>0.27</td>
<td>3,600</td>
<td>900</td>
<td>150.0</td>
<td>28</td>
</tr>
<tr>
<td>Grayshale</td>
<td>0.68</td>
<td>0.27</td>
<td>4,800</td>
<td>1,200</td>
<td>150.0</td>
<td>29</td>
</tr>
<tr>
<td>Coal</td>
<td>0.30</td>
<td>0.34</td>
<td>3,600</td>
<td>900</td>
<td>87.6</td>
<td>28</td>
</tr>
<tr>
<td>Limestone</td>
<td>4.00</td>
<td>0.28</td>
<td>20,000</td>
<td>5,000</td>
<td>165.0</td>
<td>38</td>
</tr>
<tr>
<td>Steel</td>
<td>29.0</td>
<td>0.30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
7.5.5.2 Comparison of Results Between Modeling and Field Measurement

1) Stress in Pillar

The vertical stress in the pillars were monitored using BPC before and after intersection widening, i.e. before and after cuts #6-#35 were made (see Figure 7.43). The absolute vertical stresses in the pillar before and after widening are shown in Figures 7.49 and 7.50, respectively. It can be seen that the vertical stress in the pillar from the modeling agrees well with the field measurement except the vertical stress in the pillar from the modeling before intersection widening is slightly lower than that from field measurements.

![Figure 7.49 Vertical stress in pillar before widening](image)

![Figure 7.50 Vertical stress in pillar after widening](image)

(2) Roof Horizontal Stress

Horizontal stress measurements were obtained at various horizons from 0 to 11 ft above the roofline. The horizontal stress determined by the borehole deformation gauge is used to compare with the horizontal stress from the modeling. The mean of the two components of the horizontal stresses is used for comparison since this value is independent of the orientation of the principal stress axes. Figure 7.51 shows the average
Horizontal roof stress near the entries at different horizons from both modeling and measurement. The horizontal stress from the modeling is close to the field measurements when the location is less than 6 ft above the roofline. The high horizontal stress concentration could be due to the sliding of the slips around the intersection, which is not simulated in the model. It should also be noted that the roof bolts employed were only 4-ft long and the measured and predicted stresses match well within this range of interest.

(3) Roof Bolt Loading

The bolt load changes were monitored as the intersection was widened for a period of time. Figure 7.52 shows the measured roof bolt load changes available shortly (24 days) after widening (Hanna, et al, 1991) (27). Notice that the bolt loads concentrated more on one corner of the intersection due to the effect of the slips. Since the corner around the pillar A was farthest from, and subject to the least influence by, the slips, the submodel, which consisted of a quadrant of the intersection, was chosen from this area to obtain the bolt changes. Figure 7.53 shows the load changes for the bolts around pillar A.

![Figure 7.51 Horizontal roof stress](image)

![Figure 7.52 Measured roof bolt load changes 24 days after widening (Hanna, et al, 1991)](image)

![Figure 7.53 Measured and practiced changes in bolt load after intersection widening, klbs](image)
from both the modeling and field measurements 24 days after the intersection widening. The bolt locations in the model are not exactly the same as those in the field, but the number of bolts installed is the same. The measurements showed that the bolt loads at the center of the intersection were larger and those around the entry were much larger than those around crosscut. The larger bolt loads around the entry area were attributable to both the existence of slips and major horizontal stress. The predicted load by the model is also the highest around the center of the intersection and larger around the entry than around the crosscut because the entry direction is almost perpendicular to the major horizontal stress. The predicted loads by the model are close to those by field measurements around the center of the intersection and crosscut but lower around the entry. This difference could be caused by the slip near the pillar D and the slip across the pillar A.

(4) Roof Displacement and Bed Separation

Roof displacement and bed separation were monitored by extensometers. Analysis of the data was based on the assumption that the top anchor remains in a fixed position over time. Movements of the other anchors, including the roof surface, were calculated relative to the position of the top anchor. At each station, the data were used to calculate the downward movement of each anchor and the change in distance between anchors. The change in distance between anchors corresponds to the opening and closing of bed separations that occur between the anchors. Figure 7.54 shows the extensometer locations around the intersection.

Figure 7.54 Extensometer locations

Figure 7.55 shows the displacement of the roof line at the locations shown in Figure 7.54 from the modeling and field measurements 6 days after the widening. It can
be seen that the measured roof line displacements were close to those from the modeling except at extensometer Nos 1, 5, 11 and 12 that were close to the slips. It can be concluded that the model can predict roof displacement fairly well when the effects of geological anomalies such as slips are excluded.

Figure 7.56 shows the downward movements of each anchor for three extensometer stations at points 7, 8 and 9 shown in Figure 7.54, which are away from the influence of the slips. The four anchor locations are at the depth of 0, 1.5, 2.7 and 4.5 ft, respectively. The extensometer data in Figure 7.57 were chosen from 6 days after

Figure 7.56  Measured roof displacement at different horizons

Figure 7.57  Bedding plane separation before bolt installation
widening when cut sequence #6-#35 were complete. It is shown that there was a differential displacement of .03 in. at extensometer 9 between 1.5 ft and 2.7 ft, which could be caused either by plastic deformation or bed separation. Therefore it may be concluded that no separations or only extremely small separations exist between roof surface and 2.7 ft horizon, especially at the interface of shale and limestone at 2 ft horizon.

The results from the modeling show that there was a separation along the bedding plane. 2 ft above the roofline before roof bolts were installed (see Figure 7.57). The separation at the center of the intersection was about twice as much as that at the center of the entry. But the separation was closed after installation of bolts with 6-ton pre-tension as shown in Figure 7.58. Therefore, the model can predict closure of separation in the immediate roof resulting from bolt installation.

(5) Roof Yielding

The extensometer data at the locations 7, 8 and 9 show that there were relatively large displacements six days after widening between roof surface and 1.5 ft horizon while the displacements at 2.7 ft horizon were zero or very small (see Figure 7.56), which showed the depth of roof yielding was about 2 ft. the results from the model also show that the yielding zone is within 2 ft from the roof line. Therefore, the yielding zone obtained from the model could be used to estimate the roof yielding in the field.
7.6 Design Guideline and Program Development

According to the above results, tensioned bolts play four roles in supporting the immediate roof: (1) reducing the deformation and increase the residual strength of the yielding zone in the immediate roof so that the bolted strata will have the stability not only to support itself but also take the load from, and limit the deformation of, upper strata; (2) closing the cracks and separations in the immediate roof so as to increase both the strength of the affected roof layers and the stiffness of the whole bolted strata such that any subsequent vertical displacement is resisted by the high installation load in the bolt due to pre-tension; (3) providing in the immediate roof within 2-3-ft of the roof line compressive zones which can generate frictional forces along the fractures and bedding planes near the bearing plates – beam building effect; (4) suspending the gravity load and those vertical loads caused by rock expansion and bending of the roof layers subjected to the horizontal stress. The study also showed that tensioned bolts are subjected mainly to the axial load, which is generated by the vertical roof displacement caused mainly by the horizontal stress. The compressive zones in the immediate roof caused by the bolt pre-tension are restricted to the areas near the bearing plates except the triangle zones between bolts, and between side bolts and roof corners.

Possible anchor slippage of the tensioned bolts is an important factor in determining that the tensioned bolts are suitable for weak roof under low stress level. Since the tensioned bolt transfers its load to the anchor, it is important that the bolt is anchored in a stable roof layer. However, a high stress in the roof is likely to cause either the rock in the anchorage horizon to yield or anchor slippage due to excessive loading by large roof displacement. Although the bolt pre-tension provides active support to the roof, it also reduces the amount of bolt load increase allowed as a result of subsequent roof displacement. Therefore, tensioned bolts are not suitable for roof which could have large deformation after bolt installation.

In order to quantitatively describe the interaction between the bolt and the rock, and the supporting effect of the tensioned bolts on the immediate roof, a 3-D numerical model was established based on a commercial finite element package, ABAQUS. The model considers the excavation sequence, bolt installation procedure, bolt pre-tension, bedding planes and in-situ horizontal stress. Furthermore, a design procedure for tensioned bolting using the model developed so far was also suggested and verified.
To make this design approach practical and easy to use, this study refines the proposed design procedure, gives some basic design guidelines based on geological condition in the Pittsburg seam and develops a computer program for tensioned roof bolting design for entry development using the data obtained from numerical models.

7.6.1 Design Guidelines

Generally, tensioned bolts are suitable for weak roof under low stress level, especially for the roof which has a stable layer into which the bolts can be anchored. Roof strength and stress level are the main factors for bolting design. Besides, since the roof strata in coal mines often occur in layers, the bedding planes are important in the roof bolting design. The study on bedding planes showed that the bedding plane locations in the immediate roof significantly affect the stress distribution over the entry, and roof failure modes. When there are no bedding planes in the immediate roof, more roof yielding develops at the entry corners. When the bedding planes are located within 3-ft of the roof line, this portion of the immediate roof is subject to horizontal stress concentration and more prone to yield. When the bedding planes are located 3-ft above the roofline, the horizontal stress in the immediate roof can be represented by those with two bedding planes: one located within 3-ft of the roof line, and another 3-ft above the roof line. Therefore, according to the number and locations of the bedding planes, the roof can be classified into four type: type I – without bedding planes; type II – bedding planes within 3-ft of the roofline; type III – bedding planes 3-ft above the roofline; type IV – bedding planes both within and above 3-ft of the roofline (Figure 7.59).

To obtain how the overburden depth, horizontal stress and roof type affect the tensioned roof bolting design, 36 models with overburden depth of 400, 800 and 1,200 ft, major horizontal-to-vertical stress ration of 1, 2 and 3 and different roof types were run using the geological condition of the Pittsburg seam. The results were built into a database for the development of a design program. The results in the database include depth of yield zone, maximum shear stress, maximum horizontal plastic strain, average plastic strain, maximum roof displacement, optimum pre-tension and bolt load increase.

By analyzing the results of these 36 models, the following guidelines for tensioned bolting design were obtained. Basically they are applicable to 18-ft-wide entries during development under the geological conditions similar to the Pittsburg seam. Nevertheless, the ideas are useful for understanding how overburden depth, horizontal stress level and roof type affect tensioned bolting.

7.6.1.1 Bolt Length

Figure 7.60 shows the proposed bolt length under different overburden depth and horizontal stress level for the four roof types. It can be seen that basically the bolt length increases as the overburden depth and horizontal stress level increase, and that the bolt length required by roof type I and III is shorter than that by roof type II and IV.
Figure 7.59  Roof classifies by bedding plane locations

Figure 7.60  Proposed bolt length for four roof types
7.6.1.2 Bolt Pre-tension

Figure 7.61 shows the optimum pre-tension applied in the bolts under different overburden depth and horizontal stress level for the four roof types. For roof type I and III, the pre-tension is to provide about 2 ft compressive zone in the immediate roof while for roof type II and IV, the pre-tension is to close the bedding plane separations within 3 ft of the immediate roof. It can be noted that the optimum pre-tension increases uniformly as the overburden depth and horizontal stress level increase.

![Figure 7.61 Optimun pre-tension for four roof types](image)

7.6.1.3 Bolt Spacing

Assuming a 4 ft row spacing along the entry direction is preferred, the bolt spacing design should be focused on the spacing across the entry. For roof type I and III, the spacing between the side bolts and entry corners should be shortened to 2 ft or 2.5 ft for an 18-ft entry to prevent shear failure around the entry corners. For roof type II, a 3 ft corner should be used to provide a uniform support for 7.62

Figure 7.62 Angle between possible failure plane and roof line for different types

(a) Roof type I and III
(b) Roof type II and IV

![Figure 7.62 Angle between possible failure plane and roof line for different types](image)
depth is 400 ft. It can be seen that the angles decrease as the horizontal stress level increases, which means the failure tends to occur around the entry corners when the horizontal stress is low. Therefore, more attention should be paid to the entry corners if the horizontal stress is low. Figure 7.62 also shows that the failure angles for roof type I and III are larger than for roof type II and IV, which demonstrates that entry corners for roof type I and III are more important than for roof type II and IV.

7.6.1.4 Bolt Diameter
Since the allowable range of change for the bolt row spacing and number of bolts per row are small, bolt diameter change should be used in areas where a higher bolt loading is required. If the roof is subjected to high horizontal stress and the roof has multiple bedding planes, large diameter bolts should be used to prevent either bolt yielding or bolt breaking. Figure 7.63 shows the bolt load increase under different horizontal stress level for the four roof types when the overburden depth is 1200 ft. It is obvious that the bolt load increases more under high horizontal stress.

7.6.2 Design Program Development
To make the tensioned roof bolting design more easily, the design procedure is carried out in a design program using Visual C++ and OpenGL.

Figure 7.64 shows the flow chart of the program design. First, the program asks for the user’s input; after the program obtains the input from the user, it searches for a closest case in the database; with the obtained data from the database, the program performs the design using the proposed criteria; and finally, the design is displayed by 2-D/3-D views.

In searching for the closest case in the database, a distance function is used to calculate the difference between the case in question and each of the case in the database. The case with the smallest distance is used for the design. The distance function is defined as:

$$d = \frac{|h-h_0|}{h_0} + \frac{|h_1-h_0|}{h_0} + \frac{|s_1-s_0|}{s_0} + \frac{|s_2-s_0|}{s_0}$$

where, \(d\) = distance between the case in question and those in the database, \(h\) = overburden depth of the cases in the database, \(h_0\) = overburden depth of the case in...
INPUT
Overburden depth, horizontal-vertical stress ration, entry width, rock properties, rock strength and bedding plane locations

SEARCH DESIGN INFORMATION
Search the database and look for the closest case

DESIGN
Determine bolt length, pre-tension, bolt spacing, bolt diameter by proposed criteria

DISPLAY
Display the design in 2-D and 3-D views

Figure 7.64 Flowchart for tensioned roof bolting design program

question, \( h_r = \) horizontal-to-vertical stress ratio of the cases in the database, \( h_{r0} = \) horizontal-to-vertical stress ratio of the case in question, \( s_1 = \) roof strength within the first 3 ft of the immediate roof for the cases in the database, \( s_{10} = \) roof strength within the first 3 ft of the immediate roof for the case in question, \( s_2 = \) average roof strength of the immediate roof for the cases in the database, and \( s_{20} = \) average roof strength of the immediate roof for the case in question.

The four terms in the distance function are the differences of overburden depth, horizontal stress level, roof strength within the first 3 ft of the immediate roof and average strength of the immediate roof between the case in the database and the case in question. All the terms are normalized to make each factor have the same influence on the distance.

Figure 7.65 shows the user input interface. Overburden depth, horizontal-to-vertical stress ratio, entry width and the information for each roof layer in the immediate roof are the geological input data required from the user. Figures 7.66 and 7.67 show the design with 2-D/3-D views of the bolt length, bolt spacing, bolt pre-tension and bolt diameter.
Figure 7.65 User input interface

Figure 7.66 2-D view of tensioned roof bolting pattern
Figure 7.67  3-D view of tensioned roof bolting pattern
CHAPTER 8

MECHANISMS AND DESIGN GUIDELINE FOR FULLY GROUTED BOLTING

8.1 Introduction

The fully grouted bolt is a passive support. As mentioned above, it accounts for the greatest portion of the annual roof bolt consumption in the U.S. There are three components of the fully grouted bolt; steel rebar (bolt), resin annulus, and grout/rock interaction. Fully grouted bolts are loaded in three ways: axial load, shear load and bending moment. Moreover, vertical shear stress is developed along the grout/rock interface.

Using ABAQUAS (12), finite element models were developed to simulate the components of the fully grouted bolt and its interaction with the surrounding roof strata. Several models were conducted in order to investigate the effect of the fully grouted bolt on roof stability, and the load transfer from the rock to the bolt and vise versa under different geological, mining conditions and bolt parameters. Different failure modes of the fully grouted bolts, such as bolt axial failure, bolt shear failure, and grout/rock interface failure, were defined. Bolt and roof stability measures were presented in order to evaluate the bolt and roof stability. The effect of bedding plane, overburden depth, horizontal stress, and bolt parameters were also considered.

Based on the results of numerical models and its analysis, a design guideline for fully grouted bolt system has been developed.

8.2 Finite Element Modeling of the Fully Grouted Bolt

First of all, in order to simulate the behavior of a fully grouted bolt and the roof, the model for the fully grouted bolts was developed. The components of a fully grouted bolt are: steel rebar (bolt), resin annulus, and rock/grout interaction.

8.2.1 Modeling of Steel Bolt

The steel rebar (bolt) is simulated by using the two-dimensional beam elements. A beam element is the most common type of structural components in engineering field. In general a beam is like a member designed to resist a combination of loading actions such as biaxial bending, transverse shears and axial stretching (Figure 8.1). The axial forces, transverse shear forces and loading moments will be monitored for the beam elements during the analysis.
8.2.2 Modeling of Resin Elements

Field observation show that failure rarely occurs at the bolt/grout interface, due to the strong interlock between the rebar on the steel bolt and the grout annulus (Figure 8.2). Accordingly, the bolt and the grout will be simulated as one unit. During the numerical simulation, the nodes of the beam elements will be tied to the nodes of the surrounding grout elements. The resin annulus is simulated by a 4-nodes plane strain element (Figure 8.3).
8.2.3 Modeling of Rock/Grout Interaction

The interaction between the grout and the rock is the most important parameter in the fully grouted bolts system. It is responsible for loads transferring from the surrounding rock to the fully grouted bolt and vice versa. The rock/grout interaction will be simulated by a contact surface in ABAQUS. Both the grout and rock surfaces along that interface will be “slave” or “master” depending on their Young’s modulus values. Shear stress and vertical frictional stress, will be the dominating factor that controls the deformation and failure along that interface. The shear stress along that interface, which is equal to the normal stress between the two surfaces multiply by the coefficient of friction along that interface, will resist the sliding between the two surfaces.

The bearing plate is a means to control the skin failure and roof spalling. It does not have a significant effort on the loading behavior of the fully grouted bolt system. To simulate the effect of a bearing plate, the bottom nodes of the grout element, at the bolt head, are tied to the surrounding rock element. The top nodes of the grout elements are connected to the surrounding rock nodes by gap elements (Figure 8.3). The gap element allows no interaction between the grout and the surrounding rock unless certain amount of clearance is reached.

8.3 Effects of Geological and Bolt Parameters on the Behavior and Failure Modes of the Fully Grouted Bolts

In order to make it clear the behavior of the fully grouted bolt under different factors and conditions, numerical analysis is conducted. Moreover, different failure modes of the fully grouted bolts, according to the loading conditions, is investigated.

8.3.1 Failure Modes of Fully Grouted Bolt

The fully grouted bolt system consists of three components, the bolt, the resin, and the host rock. Each of the three components has different material properties which will affect the final performance of the fully grouted bolt. Consequently, there are three types of failure modes that can occur for the fully grouted bolts. Failure can take place in the bolt, steel bolt axial or shear failure, or at the grout/rock interface failure.

8.3.1.1 Steel Bolt Axial Failure

The axial behavior of the fully grouted bolts is governed entirely by the steel bar, because it is much stiffer and stronger than the grout and rock. The most important advantages of the steel are its ductility and ability to sustain large loads and deformation (Pile, et al, 2003) (28). Axial failure will occur in the steel bar if it is subjected to an axial load which exceeds the ultimate capacity of the steel. The ultimate capacity of the steel depends on the steel grade and the bolt diameter.
8.3.1.2 Steel Bolt Shear Failure

If a section along the fully grouted bolt is subjected to a shear load that exceeds its shear strength, then shear failure could occur along that section of the bolt. The importance of the shear strength of the fully grouted bolts is that it can resist the horizontal movement that occurs as a result of bedding plane sliding. The shear strength of the steel bar is about 50% of its uniaxial tensile strength.

8.3.1.3 Grout/Rock Interface Shear Failure

The load transfer mechanism of the fully grouted bolts depends mainly on the grout/rock interface. The interaction at the grout/rock interface is a complicated problem; a lot of parameters affect its behavior. The response of the bolt to strata movement depends on the shear strength of that interface and shear stress generated on it. The shear strength of the grout/rock interface has three components: adhesion, mechanical interlock, and friction. Once relative displacement occurs along that interface, the adhesion and the mechanical interlock will be lost. The friction will be the dominating factor that controls the failure of that interface. The shear strength for that interface will be estimated as a ratio of the shear strength of the weakest rock composing the interface:

\[
\text{Interfacial shear strength} = \frac{\text{weakest rock shear strength}}{\sqrt{3}} \quad (8.1)
\]

8.3.2 Parameters that Affect Bolt Loads and Stresses on Grout/Rock Interface

8.3.2.1 Common Model and Parameters

Figure 8.4 shows a layout for the bolt spacing and numbers for the models. A detailed analysis of the shear load and shear stress at grout/rock interface were conducted for bolt #1 (left side bolt). In the fully grouted bolt system, the side bolt was subjected to more shear loads than the corner bolt because it was close to the entry center where maximum sliding at the bedding planes occur. Because it was close to the entry center...
where maximum separation at the bedding planes occurs, a detailed analysis of the axial load was conducted for bolt #2 (left center bolt). A bolt of grade 60 (the ultimate axial capacity for this grade is 26,500 lb and the ultimate shear capacity is 50% of the uniaxial capacity) was used for this study.

8.3.2.2 Geological Parameters

The behavior of fully grouted bolts is highly depended on the surrounding rock properties and the site geological conditions such as roof geology, and strata sequence, bedding plane locations, friction coefficient on bedding plane surface, overburden depth, and stress level. The effect of those geological parameters was discussed.

(1) Strata Strength Sequence

Different models with different roof geology have been conducted. The first roof layer was 3 ft thick overlain by the second layer of 2 ft thick. Table 8.1 shows the mechanical properties of the four possible geological combinations of the immediate roof strata that could be found in the field. The two layers are separated by a bedding plane. The uniaxial compressive strength and the ratio between the layers’ Young’s modulus and the coal’s Young’s modulus ($E_L/E_C$) were used to represent the weak and the strong geology as follows: For weak layer: $E_C < E_L < 4E_C$ and for strong layer: $4E_C < E_L < 8E_C$.

The Coulomb friction model was used to describe the relationship between the shear and normal stresses along the two opposite sides of the bedding plane. The following parameters were used for the models:

- Friction coefficient of the bedding plane = 0.25
- Friction coefficient at grout/rock interface = 0.5
- Overburden depth = 600 ft
- Stress level = 3
- Bolts = 4 ft long and 3/4-in in diameter

Figure 8.5 shows that there was a typical bolt axial load distribution along the bolt length for different roof geology. Along the bolt length, starting from the bolt head the axial load increased until it reached its maximum value then decreased. The extent of this uniformly loaded parts of bolt axial load distribution depended on the type of roof geology. The highest maximum axial load was observed for roof geology IV. This can be

<table>
<thead>
<tr>
<th>Roof Geology No.</th>
<th>First Layer</th>
<th></th>
<th>Second Layer</th>
<th></th>
<th>Interface shear strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UCS (psi)</td>
<td>$E_L/E_C$</td>
<td>UCS (psi)</td>
<td>$E_L/E_C$</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>700</td>
<td>4</td>
<td>1,200</td>
<td>8</td>
<td>200</td>
</tr>
<tr>
<td>II</td>
<td>1,200</td>
<td>8</td>
<td>700</td>
<td>4</td>
<td>200</td>
</tr>
<tr>
<td>III</td>
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<td>8</td>
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<td>520</td>
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<tr>
<td>IV</td>
<td>600</td>
<td>4</td>
<td>600</td>
<td>4</td>
<td>173</td>
</tr>
</tbody>
</table>
explained by the observed large vertical displacement occurred in that weak roof strata. Despite of this high axial load, it did not exceed the ultimate capacity of the bolt. On the other hand, the smallest maximum axial bolt load was observed for roof geology III. The immediate roof strata in that geology was strong, accordingly less vertical displacement was expected to take place in that roof. From the roof geology II, the bottom strong layer reduced the vertical displacement of the upper weak layer.

Since the fully grouted bolt is in full contact with the surrounding roof strata, it will respond to any rock deformation that occurs within the bolted area. Shear loads usually develop in the fully grouted bolts as a result of the horizontal movement for the immediate roof at the bedding plane location. At the bedding plane location, the shear load increases rapidly but decreased quickly away from it (Figure 8.6). The direction of the shear load varied according to the direction of the relative motion (sliding) on the surface of the bedding plane. The highest maximum shear load was observed for roof geology IV. This is because the weaker the roof strata are the larger horizontal relative movement along the bedding plane occurs. From the result of roof geology II, it can be seen that the bottom strong layer reduces the relative horizontal movement along the bedding plane.
The highest maximum shear stress was observed for roof geology IV. This is because of the large horizontal movement in the weak roof strata. Therefore, more shear analysis, it was concluded that roof geology surrounding the fully grouted bolt behavior of the bolts and the grout/rock interface could produce a large normal pressure on the interface surface. Therefore, more shear stress developed on that interface.

Figure 8.7 shows that the shear stresses distribution along the grout/rock interface. It is noticed that a sudden change in the shear stress at the grout/rock interface at the bedding plane location occurs. This behavior is due to the decoupling of the interface at the bedding plane location when any movement, however minor it is, occurs at the intersection of the bedding plane and the grout/rock interface. As a result the compatibility of deformation is lost across the grout/rock interface (Li and Stillborg, 1999)\(^{(29)}\). The highest maximum shear stress was observed for roof geology IV. This is because of the large horizontal movement in the weak roof strata. The large horizontal movement produces a large normal pressure on the interface surface. Therefore, more shear stress developed on that interface.

From the above analysis, it was concluded that roof geology surrounding the fully grouted bolt has a great effect on the loading behavior of the bolts and the grout/rock interface. A weaker rock will develop more deformation in the strata, which will transfer more loads to the bolt and more stresses on the grout/rock interface. In case of weak roof, the shear strength capacity could be exceeded and failure at grout/rock interface could occur causing bolts slippage.
Because coal deposits are usually associated with sedimentary rock that is made of a series of bedding planes or rock containing joints and fractures, it is very important to study the effect of bedding plane on the fully grouted bolt loads and also on the stresses distribution along the grout/rock interface. So, various models have been conducted to investigate the effect of bedding plane location on the bolt axial loads, shear loads and grout/rock interface behavior. The following parameters were used for the model:

- Friction coefficient of the bedding plane = 0.25
- Friction coefficient at grout/rock interface = 0.5
- Overburden depth = 1,200, 1,400, and 1,600 ft
- Stress level = 3
- Bolts = 8 ft long and 3/4-in in diameter
- Roof geology I (the first layer is weak (700 psi), overlain by a strong layer (1,200 psi))
- Locations of bedding plane = 1, 2, 3, 4, 6, and 8 ft from the roofline

Figure 8.7 Shear stress distributions along the grout/rock interface for different roof geology
Figure 8.8 shows that, for the first layer of less than 3 ft thick, the bolt axial load was increased by increasing the thickness of that layer. Then, for the first layer more than 3 ft thick, the bolt axial load was decreased by increasing the thickness of that layer. For the immediate roof layer up to 3 ft thick, the horizontal stress concentration was proportional to the thickness of the first layer. Therefore, the behavior of the bolt axial load was dominated by the effect of horizontal stress concentration. On the other hand, when the bedding plane located at a distance more than 3 ft from the roofline, it seems that the stress concentration was localized around the bedding plane. If the first layer was thicker beam deflection will determine the amount of vertical displacement imposed on the bolts. The thicker the first layer the less deflection occurred and thus less axial bolt load. Figure 8.8 also shows that maximum bolt axial load increases with increasing the overburden depth. However, the maximum axial load at any bedding plane location or overburden depth did not exceed the ultimate capacity.

Figure 8.9 shows that the location of maximum shear load along the fully grouted bolts varied according to the location of the bedding plane. The direction of shear load along the fully grouted bolt varied according to the direction of relative horizontal movement on the surface of bedding plane. The fully grouted bolts resist the lateral movement (sliding) that could occur at the bedding plane. The interaction between the grout and rock will allow within the grout annulus. Thus, the shear load will develop on the steel bolt at the bedding plane location. The bolt shear load mainly depends on the horizontal movement along the bedding plane surface. For the immediate roof layer up to 3 ft thick, the horizontal stress concentration is proportional to the thickness of the first layer and hence more sliding is expected along the bedding plane. As the stress concentration is localized around the bedding plane for the thicker immediate roofs, less sliding will occur along the bedding plane.

Figure 8.10 shows that, when the first layer is less than 3 ft thick, the maximum bolt shear load increases with increasing thickness of that layer. On the other hand, when the first layer is more than 3 ft the maximum bolt shear load decreases with increasing the thickness of the first layer. When there is no bedding plane crossing the fully grouted
Figure 8.9 Maximum shear load location varies with bedding plane location

Figure 8.10 Effect of bedding plane location on maximum bolt shear load
bole, the shear load developed into the bolt is very low. Figure 8.10 also shows that the maximum bolt shear load increases with increasing the overburden depth. Therefore, a combination of high overburden depth and bedding plane located between 2 ft and 3 ft from the roofline will cause the fully grouted bolts to fail in shear along the steel bar.

Figure 8.11 shows that the shear stress along the grout/rock interface increases within the first layer when it is less than 3 ft thick. Conversely when the first layer is thicker than 3 ft, the shear stress developed on the grout/rock interface decreases with increasing thickness. When the first layer is less than 3 ft thick, the horizontal stress concentration will cause the normal pressure on the grout/rock interface to increase, accordingly the shear stress along that interface increase too. For thicker layers, the horizontal stress will concentrate around the bedding plane. Therefore a zone of stress relief will be developed in the first layer and the normal pressure on the interface will decrease. Consequentially the shear stress at the grout/rock interface will decrease. Figure 8.11 also shows that the maximum interface shear stress increases with increasing the overburden depth.

(3) Stress Level

To investigate the effect of stress level on the behavior of the fully grouted bolts, different models have been run as described below:

- Friction coefficient of the bedding plane = 0.25
- Friction coefficient at grout/rock interface = 0.5
- Overburden depth = 600 ft
- Bolts = 4 ft long and 3/4-in in diameter
- Roof geology I (the first layer is weak (700 psi), overlain by a strong layer (1,200 psi))
- Stress level = 1.0, 2.0, 2.5, and 3.0

Figure 8.12 shows the increase of axial load of the fully grouted bolts with increasing stress level. Increasing stress level will lead to increase the induced horizontal
Figure 8.12  Axial load distributions along the fully grouted bolt with different stress levels

Figure 8.13  Shear load distributions along fully grouted bolt with different stress level

Figure 8.14  Shear stress distributions along grout/rock interface with different stress level
stress within the immediate roof. Hence more normal pressure will be applied at the grout/rock interface giving the chance for more stresses to be transferred from the surrounding rock to the fully grouted bolt causing increase in its axial loads. Moreover, increasing the horizontal stress can produce additional roof deflection by axially loaded the roof. The stress level which is the ratio of horizontal to vertical stress is a major factor affecting the stress distribution in the immediate roof and loads of the fully grouted bolts.

Figure 8.13 shows the increase of shear load of the fully grouted bolts with increasing stress level. By increasing the stress level within the immediate roof more horizontal stress will be apply to it, therefore, the sliding at the bedding plane surface will increase.

Figure 8.14 shows the increase of the shear stress along the grout/rock interface of the fully grouted bolt with increasing stress level. When the horizontal stress applied to the immediate roof increases, the grout/rock interface will subject to high confinement pressure which in turn will lead to increase in the shear stress developed at that interface.

8.3.2.3 Bolt Parameters

Bolt parameters, such as bolt length, bolt diameter, and coefficient of friction along the grout/rock interface, can affect the behavior of the fully grouted bolts. The effect of these parameters on the bolt loading will be investigated in the following section.

(1) Bolt Length

Bolt length is one of the critical parameters affect the fully grouted bolt system performance. The effect of varying bolt length on the bolt loads is investigated. The following parameters were used for the model:

- Friction coefficient of the bedding plane = 0.25
- Friction coefficient at grout/rock interface = 0.5
- Overburden depth = 600 ft
- Stress level = 3.0
- Bolts diameter = 6/8-in
- Roof geology I (the first layer is weak (700 psi), overlain by a strong layer (1,200 psi))
- Bolt length = 3, 4, 5, 6, and 8 ft

Figure 8.15 shows the decrease of bolt axial loads with increasing bolt length. With longer bolts, the beam thickness built by the fully grouted bolts increases. As a result, the beam deflection will decrease. This beam will provide the immediate roof with more rigidity and stiffness by decreasing the vertical displacement within the immediate roof. Less axial load will develop in the bolts providing the surfaces of the bedding plane with friction to reduce the vertical displacement.

Figure 8.16 shows the decrease of bolt shear loads with increasing bolt length. because of the full contact between the fully grouted bolts and the surrounding rock, increasing the bolt length builds a thicker beam which in turn can resist better the lateral
Figure 8.15  Maximum axial loads along fully grouted bolt with different bolt length

Figure 8.16  Maximum shear loads along fully grouted bolt with different bolt length

Figure 8.17  Maximum shear stress at grout/rock interface with different bolt length
movement along the bedding planes. Therefore less sliding will occur along the bedding plane surfaces and less shear load will transfer to the fully grouted bolts.

Figure 8.17 shows the decrease of shear stress at the grout/rock interface with increasing bolt length. As the bolt length increases, less loads will be transferred from the surrounding strata to the bolt. The shear stress along the interface only decreases by 38 psi from 3-ft to 8-ft long bolt. Accordingly, increasing the bolt length affects the shear stress along the interface. Increasing the bolt length has a great reinforcement effect for the rock strata as it helps build a thicker and stronger beam by reducing both vertical and horizontal displacements.

(2) Bolt Diameter

The bolt diameter is an important parameter because it can affect both the axial and lateral stiffness of the system. The effect of varying bolt diameter on the bolt loads is investigated. The following parameters were used for the model:

- Friction coefficient of the bedding plane = 0.25
- Friction coefficient at grout/rock interface = 0.5
- Overburden depth = 600 ft
- Stress level = 3.0
- Bolt length = 4 ft
- Roof geology I (the first layer is weak (700 psi), overlain by a strong layer (1,200 psi))
- Bolt diameter = 5/8, 6/8, and 7/8-in

Figure 8.18 shows that increasing the bolt diameter decreases the axial load of the fully grouted bolts. Increasing the bolt diameter will increase the bolt ultimate capacity and thus the stiffness of the fully grouted bolt system.

Figure 8.19 shows that the bolt shear load decreases very slightly with increase in the bolt diameter. As the bolt capacity increase, its ability to resist the relative movement occurs in the immediate roof and less shear load will develop along the bolt.

Figure 8.20 shows that the shear stress along the grout/rock interface decreases with bolt diameter. The shear stress along the interface decreases by about 7 psi when the bolt diameter changes from 5/8- to 6/8-in or from 6/8- to 7/8-in. That change in the shear stress along the interface is very small. Thus changing the bolt diameter affects slightly the stress distribution along the grout/rock interface.

(3) Friction Coefficient of the Grout/Rock Interface

Friction of the grout/rock interface plays an important role in controlling the reinforcement stiffness of the fully grouted bolt. The load transfer for the fully grout bolts is mostly dependent on the shear stress sustained on the grout/rock interface. The interface between the rock and grout annulus is much weaker than the rock or grout surrounding it. Shear stress will develop along that interface and hence sliding will occur. If the shear stress along the interface exceeds the interface strength, failure will occur.
Figure 8.18  Relationship between bolt diameter and maximum bolt load

Figure 8.19  Relationship between bolt diameter and maximum bolt shear load

Figure 8.20 Relationship between bolt diameter and maximum shear stress at grout/rock interface
Accordingly, the condition of that interface is very important for the loading transfer mechanism. However, there are no direct measurements on the coefficient of friction of the grout/rock interface. Therefore, it was assumed that the friction coefficients at the grout/rock interface ranges from 0.25, which represents the coefficient of friction at the bedding planes to 1.0 which represents a complete and perfect bonding at the grout/rock interface. The following parameters were used for the model:

- Friction coefficient of the bedding plane = 0.25
- Overburden depth = 600 ft
- Stress level = 3.0
- Bolts = 4 ft long and 6/8-in in diameter
- Roof geology 1 (the first layer is weak (700 psi), overlain by a strong layer (1,200 psi))
- Friction coefficient at grout/rock interface = 0.25, 0.4, 0.5, 0.6, 0.75, 0.8, and 1.0

Figure 8.21 shows the relation between the coefficient of friction along the grout/rock interface and the bolt axial load. Increasing the coefficient of friction at the grout/rock interface will increase very slightly the bolt axial load. The increase in the bolt axial load is significant when the coefficient of friction increases bolt axial load slightly. Increasing the coefficient of friction at the grout/rock interface will increase the shear stress on that interface. Consequently, more loads will be transferred from the surrounding rock to the fully grouted bolts.

Figure 8.22 shows the relation between the coefficient of friction along the grout/rock interface and the bolt shear load. Increasing the coefficient of friction at the grout/rock interface will decrease the bolt shear load. The decrease is more pronounced when the coefficient of friction is increased from 0.25 to 0.75. Further increase will only slightly affect the maximum shear load.

Figure 8.23 shows the relation between the coefficient of friction along the grout/rock interface and the shear stress on it. Increasing the coefficient of friction at the grout/rock interface will increase the shear stress. The increase in the shear stress is significant when the coefficient of friction increases from 0.25 to 0.5. But, further increase in the coefficient of friction increases the shear stress only slightly. Increasing the coefficient of friction at the grout/rock interface will increase the shear stress on that interface. Consequently, more shear stress will develop on that interface.

8.3.3 Summary

8.3.3.1 Fully Grouted Bolt Behavior

From the above study for the parameters that affect the loading behavior of the fully grouted bolt, the amount of internal bolt loads is governed by three sources:

(1) The Deformation Occurred in the Immediate Roof

The deformation occupied in the immediate roof is highly correlated with the roof geology, overburden depth and the location of bedding planes, and coefficient of
Figure 8.21  Relationship between coefficient of friction at grout/rock interface and maximum bolt axial load

Figure 8.22  Relationship between coefficient of friction at grout/rock interface and maximum bolt shear load

Figure 8.23  Relationship between coefficient of friction at grout/rock interface and maximum bolt shear stress distribution along it
friction along them. The internal loads of the fully grouted bolts are affected by both the vertical and horizontal deformations that occur in the immediate roof. The vertical deformation will generate axial bolt loads, while the bolt shear loads is mainly caused by the horizontal movement.

(2) The Stresses Induced in the Immediate Roof

As the in-situ horizontal stress increases, a higher induced horizontal stress is developed in the immediate roof and a greater normal pressure is applied to the grout/rock interface. Therefore, a higher axial bolt load will be developed due to the increase of the induced pressure at the grout/rock interface. It was observed that more shear loads were developed by increasing the stress level.

(3) Bolt Properties

The bolt parameters such as bolt diameter, bolt length and coefficient of friction along the grout/rock interface control the amount of bolt loading by different mechanisms. The induced normal pressure at the grout/rock interface depends on the coefficient of friction along the grout/rock interface. Increasing the values of this parameter will lead to an increase in the induced shear stress at the grout/rock interface. Therefore, more interaction between the bolt and the surrounding rock will be developed. After considering the influence of increasing the bolt diameter, the system strength will increase too. By increasing the system strength the deformation of the bolted area will decrease and as a result less internal loads will develop in the fully grouted bolt. Beam building concept was very obvious when different bolt lengths were studied. The longer the bolt, the thicker the beam that was created by the fully grouted bolt and as a result less deformation is expected in the immediate roof and more stability will occur.

8.3.3.2 Failure Modes

From the result of analysis, the following failure modes could occur for the fully grouted bolt.

(1) Bolt Axial Failure

Bolt axial failure was not observed for the studied parameters. This could be explained by the values of the studied ranges of these parameters. The high ultimate strength and high modulus of elasticity of the steel bolt make it able to sustain large loads. This conclusion matches with the field observation where actual cases are reported for bolt axial failure.

(2) Bolt Shear Failure

It was found that the bolt shear failure could happen in case of high overburden depth and the existence of bedding planes at 2 to 3 ft from the roofline. In this case, a high bolt shear load is developed and could exceed the ultimate shear capacity of the bolt. Because sliding along any bedding plane intersecting the bolt is the main reason for the induced shear bolt load, a low coefficient of friction along the
bedding plane will maximize the sliding that could occur and hence the possibility of bolt shear failure will increase.

(3) Grout/Rock Interface Failure

It was found that, the shear stress at the grout/rock interface is affected by the roof geology. In case of weak immediate roof, the shear stress at the grout/rock interface could exceed its shear capacity and shear failure occurs causing bolt slippage. As a result, in case of weak immediate roof the grout/rock interface controls the failure mechanism of the fully grouted bolt system.

8.3.4 Bolt Stability Measures

Bolt shear failure and grout/rock interface failure are the common two types of failures that could occur for the fully grouted bolt. The measures for evaluating bolt stability are: bolt shear load safety factor (BSHL_SF) and interface shear stress (ISHS_SF) and can be calculated as follows:

\[
BSHL \_ SF = \frac{\text{shear ultimate strength}}{\text{bolt shear load}} > 1.0 \quad (8.2)
\]

\[
ISHS \_ SF = \frac{\text{interface shear strength}}{\text{interface shear load}} > 1.0 \quad (8.3)
\]

8.4 Evaluation of Roof Stability

There are many factors that can affect roof stability, such as locations of bedding planes, roof strata sequence, in-situ horizontal stresses, overburden depth, and bolt parameters. So, in order to develop a successful design and roof stability control technique for underground coal mines, the effects of these factors on the stress distributions and roof stability are discussed by using the numerical modeling results.

8.4.1 Roof Stability Measures

In order to evaluate the roof stability, the following roof stability measures are used.

8.4.1.1 Roof Safety Factor

The roof safety factor is introduced to measure the ability of the roof to resist the shear failure. In addition the tension cut-off is considered in the calculation of the RSF to measure the ability of the roof to resist tensile failure. In this study, roof layers are assumed to follow the Mohr-Coulomb failure criterion. The stability of the immediate roof is estimated by calculating the safety factor for the bolted area of the immediate roof. The following formula will be used in calculating the RSF:

\[
\text{When } \sigma_3 > \sigma_i \quad RSF = \frac{\sigma_c + K \times \sigma_3}{\sigma_i} \quad (8.4)
\]

\[
\text{When } \sigma_3 < \sigma_i \quad RSF = \frac{\sigma_t}{\sigma_i} \quad (8.5)
\]
where, $\sigma_c$ = the uniaxial compressive strength of the rock, $\sigma_t$ = the tensile strength of the rock, $\sigma_3$ and $\sigma_1$ = the induced minimum and maximum principal stresses, and $K$ = the triaxial strength factor

8.4.1.2 Roof Deflection (RD)

Excessive roof deflection is considered one of the major causes of roof failure in underground coal mines. After excavation the immediate roof of the entry will sag in response to rock weight and separation along the bedding planes. In order to establish a deflection based stability criterion that assesses roof stability from the deformation point of view, a critical deflection value needs to be determined. Based on experience in the Pittsburg coal seam the maximum allowable deflection value, for the immediate roof, should be less than 1.1 in (Chen, et al., 2003)\(^{(30)}\).

As part of a parametric study, the roof deflection will be estimated for the study cases as one of the stability measures. The roof stability is determined by comparing the estimated roof deflection with the allowable roof deflection.

8.4.1.3 Roof Weighted Plastic Strain (RWPS)

The roof weighted plastic strain for the immediate roof is estimated as follows:

$$RWPS = \frac{\sum (P_E \times A_E)}{A_B}$$ \hspace{1cm} (8.6)

where, $P_E$ = the plastic strain at the center of each element, $A_E$ = the element area, and $A_B$ = the roof area with width = entry width and height = bolt length.

The above expression illustrates that the immediate roof stability depends on the extent of yielded roof area and the magnitude of plastic strain developed in every elements of the bolted area. RWPS is used as a relative measure for immediate roof stability. There is no available critical value for RWPS. But it can be used to differentiate the stability of different roof conditions/designs. Figure 8.24 shows two different immediate roofs with different plastic strain distribution. Despite of the small yielded area in Figure 8.24 (a) as compared to Figure 8.24 (b), case (a) could be less stable if it has higher magnitude of plastic strain.

![Figure 8.24 Different immediate roofs with different plastic strain distribution](image-url)
8.4.2 Numerical Model

Typical geological and mining conditions for the Pittsburg seam were used (Figure 8.25). A three entry system was used in which the entry width is 18 ft, the coal seam is 8 ft thick, and the mining height is 7 ft. The bolt spacing is 4 ft between bolts with side bolts 3 ft from the respective pillar rib. The pillar is $80 \times 160$ ft center to center. The bolt length and diameter are 8 ft long and 3/4 in, respectively. The overburden thickness is 600 ft. The immediate roof is layers roof with two prominent bedding planes. The mechanical properties for the rock, bolt and resin used in the model are listed in Table 8.2.

![Figure 8.25 Geological column and entry details](image)

Table 8.2 Mechanical properties for rock, bolt and resin grout

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Young’s Modulus ($\times 10^6$ psi)</th>
<th>Poisson’s Ratio</th>
<th>Uniaxial Compressive Strength (psi)</th>
<th>Strength Used in Models (psi)</th>
<th>Unit Weight (lb/ft$^3$)</th>
<th>Internal Friction Angle (°)</th>
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<tr>
<td>Sandstone</td>
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<td>13,000</td>
<td>3,250</td>
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<td>10,000</td>
<td>2,500</td>
<td>160.0</td>
<td>33</td>
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<tr>
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<td>0.27</td>
<td>2,400</td>
<td>600</td>
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<td>28</td>
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<tr>
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<td>0.27</td>
<td>3,000</td>
<td>750</td>
<td>150.0</td>
<td>29</td>
</tr>
<tr>
<td>Gray shale</td>
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<td>0.27</td>
<td>4,800</td>
<td>1,200</td>
<td>150.0</td>
<td>29</td>
</tr>
<tr>
<td>Silty shale</td>
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<td>4,800</td>
<td>1,200</td>
<td>150.0</td>
<td>29</td>
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<td>0.34</td>
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<td>900</td>
<td>87.6</td>
<td>28</td>
</tr>
<tr>
<td>Limestone</td>
<td>2.50</td>
<td>0.30</td>
<td>10,000</td>
<td>2,500</td>
<td>165.0</td>
<td>31</td>
</tr>
<tr>
<td>Resin Grout</td>
<td>0.435</td>
<td>0.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>29.0</td>
<td>0.30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The global model is used to analyze the overall response of the rock mass while the sub-model is to analyze the local response around the entry. The sub-model is linked with the global model by displacement. The total width of the global model is 1,214 ft and height of 191 ft as shown in Figure 8.26.

Using ABAQUS, four models have been conducted to simulate the effect of fully grouted bolts in the immediate roof during development as shown in Table 8.3. The horizontal-to-vertical stress ratio is 3 and the horizontal stress is applied as an initial stresses in the models. The models are conducted with and without bedding planes activated to study the effect of bedding planes. Two bedding planes are located in the models, at distance of 3 ft and 14 ft, respectively, from the roofline. The model of bedding planes is the same as Chapter 7. The Mohr-Coulomb failure criterion is also used as a yield criterion for rock strata.

![Diagram of models](image_url)

(a) Three-entry development  
(b) Cross-section 1-1 (2D finite element model)  
(c) Detail of the middle entry

Figure 8.26 Models for the fully grouted bolt system
8.4.3 Effect of Bedding Planes/Fully Grouted Bolt Installation on Roof Behavior

8.4.3.1 Stress Distribution around the Entry

(1) Vertical Stress Distribution in the Immediate Roof

Figure 8.27 shows the vertical stress distribution in the immediate roof for the four models. It appears that in case of model No.1 where no fully grouted bolts are installed and the bedding planes are not activated (Figure 8.27 (a)), there is vertical stress concentration at the pillar corners within one foot of the rib side. The vertical stress is relieved from the central area of the immediate roof. In Figure 8.27 (b) after the fully grouted bolts have been installed, the vertical stress increases considerably within the bolted area. The maximum stress concentration at the pillar corners is reduced to half of that without bolts (from 1,200 psi to 600 psi). In Figures 8.27 (c) and (d) the bedding planes are activated, it has the same trend as the case that bedding planes are not activated. The stress concentration at the pillar corners is reduced from 1,000 psi to 600 psi.

(2) Horizontal Stress Distribution in the Immediate Roof

Figure 8.28 shows the horizontal stress distribution in the immediate roof for the four models. It appears that in case of model No.1 where no fully grouted bolts are installed and the bedding planes are not activated (Figure 8.28 (a)), there is a horizontal stress concentration at the pillar corners within one foot of the rib side. The horizontal stress is distributed over the central area of the immediate roof. In Figure 8.28 (b), after the fully grouted bolts are installed, the horizontal stress decreases significantly within the bolting area and that above the pillar. The stress concentration near the rib reduces from 2,400 psi to 1,800 psi after the bolts are installed. In Figure 8.28 (c), where the two prominent bedding planes are activated, there is a high horizontal stress concentration at the first layer, especially near the first bedding plane. Figure 8.28 (d) shows the effect of fully grouted bolts on horizontal stress distribution in the immediate roof. The bolts reduce the stress concentration everywhere in the first layer and relieve the stress concentration around the first bedding plane.

(3) Shear Stress Distribution in the Immediate Roof

Figure 8.29 shows the shear stress distribution in the immediate roof for the four models. It appears that in case of model No.1 where no fully grouted bolts are installed and the bedding planes are not activated (Figure 8.29 (a)), there is a shear stress concentration at the pillar corners within one foot of the rib side, and the shear stress decrease to very low values over the central area of the immediate roof. Figure 8.29 (b)
Figure 8.27 Vertical stress distributions over the entry
Figure 8.28 Horizontal stress distributions over the entry
Figure 8.29  Shear stress distributions over the entry
shows the effect of the fully grouted bolts on shear stress distribution in the immediate roof. The shear stress is reduced everywhere especially at the pillar corner. It reduces from 600 psi to 300 psi. When the bedding planes are activated (Figure 8.29 (c)), there is a shear stress concentration at the pillar corners but its magnitude has been reduced comparing with Figure 8.29 (a). The sliding and separation along the bedding plane cause the shear stress to release from the first layer. Moreover, the shear stress reduces to low values near the center of the immediate roof. Figure 8.29 (d) shows the effect of fully grouted bolts on shear stress distribution in the immediate roof. The shear stress at the first layer reduces and it decreases from 500 psi to 200 psi at the pillar corner.

In summary, due to full contact between the fully grouted bolts and the surrounding strata and the rigidity of the bolts, the fully grouted bolts take loads from the surrounding strata and alter stress distribution in the roof and above the pillar significantly. The fully grouted bolts can strengthen the roof because their stiffness is much higher than those of the surrounding strata.

8.4.3.2 Plastic Strain Distribution around the Entry

The magnitude and distribution of the plastic strain in the immediate roof are important factors for determining the roof stability. The results can indicate which part of the roof is yielding and where the roof bolts are needed. Figure 8.30 shows the plastic strain distribution in the immediate roof for the models with the bedding planes activated before and after the fully grouted bolts are installed. Figure 8.30(a) shows the plastic strain is distributed over the first layer and concentrated around the first bedding plane. The magnitude of the plastic strain is higher near the pillar corners than that of the entry center. The magnitude of the plastic strain is much higher near the first bedding plane location. Above the four feet horizon in the immediate roof, there is no indication of yielding. Figure 8.30 (b) shows the effect of fully grouted bolts on both the magnitude and distribution of the plastic strain in the immediate roof. The plastic strain in the yielding zone has been decreased to much lower values when the fully grouted bolts are

![Figure 8.30](image-url)  
(a) Without fully grouted bolts  
(b) With fully grouted bolts

**Figure 8.30** Effect of fully grouted bolt on yield zone
installed. The plastic strain has been reduced by 80% near the first bedding plane and by 33% near the entry corner. The reduction on the plastic strain in the immediate roof, caused by the fully grouted bolt installation, will strengthen the roof.

8.4.3.3 Vertical Deformation in the Immediate Roof

Figure 8.31 shows the vertical displacement in the immediate roof for the models with the bedding planes before and after the fully grouted bolts are installed. The vertical displacement is larger toward the entry center and becomes smaller going farther into the roof. Figure 8.31 (a) shows the vertical displacement occurs in the immediate roof where no bolt is installed. When the fully grouted bolts are installed, as shown in Figure 8.31 (b), it can be seen a reduction in vertical displacement from 75% in the first foot of the immediate roof to approximately 60% above the first bedding plane. The reduction in vertical displacement will enhance the stability of the immediate roof.

8.4.3.4 Separation and Sliding along the Bedding Planes

Separation along the bedding planes will occur if the bedding plane is subjected to tensile stress perpendicular to its surface, and sliding will occur when the shear stress on the bedding plane exceeds its shear strength (Guo & Peng, 1991) (31). Figures 8.32 (a) and (b) show sliding occurs along the first and second prominent bedding planes. The sliding along the first bedding plane is greater than that along the second one, as the first bedding plane is the closest to the roofline. The maximum value of sliding occurs near the pillars and tends to be minimal at the entry center. After the fully grouted bolts are installed, the sliding along the first bedding plane decreases significantly (Figure 8.32 (a)). Figure 8.32 (c) shows the separation that occurs on the first bedding plane. Without the fully grouted bolts maximum separation occurs at the entry center and tends to be zero close to the pillar. It is obvious that the separation is, after the fully grouted bolts are installed, reduced to almost zero. The axial loads generated within the bolts increase the friction along the bedding plane and close the separation.
Figure 8.32 Effect of fully grouted bolts on sliding and separation along bedding planes

(a) Sliding along first bedding plane

(b) Sliding along second bedding plane

(c) Separation along first bedding plane
8.4.3.5 Roof Stability Measures

For models No.3 and No.4, the estimated roof safety factors are 2.15 and 3.85, respectively. It is obvious that the installation of the fully grouted bolt leads to the increase of RSF by 79%. The maximum deflection, at the center of the entry decreases from 4 in to 1.1 in. Figure 8.31 shows the hatched area which has a deflection values exceed the allowable roof deflection (= 1.1 in). In case of the unsupported roof, Figure 8.31 (a) shows a large unstable area whereas Figure 8.31 (b) shows a very small wedge area of the roof has a deflection value exceeding the allowable one. By installing the fully grouted bolts, the RWPS decreased from 1.06 to 0.715, which means that the installation of the fully grouted bolt increases the roof stability.

8.4.4 Effect of Bedding Planes on Roof Behavior

Rock masses in nature contain several types of discontinuities such as joints, faults and bedding planes. These discontinuities affect the roof stability strongly, especially after the excavation of entries in underground coal mines. In order to investigate the effect of bedding plane, several models have been conducted. In these models, overburden thickness was 400 ft and the stress ratio was 3. Grade 60 and 8 ft long bolts were used. The bedding plane location varied between 1, 2, 3, 4, 6, and 8 ft as measured from the roofline. Moreover, the coefficient of friction also varied between 0.2, 0.25, 0.3, 0.35, 0.4, and 0.5.

8.4.4.1 Effect of Bedding Plane Locations
(1) Effect on Stress Distribution

Figure 8.33 shows the effect of bedding plane location on the vertical stress distribution in the immediate roof. Due to the separation occurred along the bedding plane, the vertical stress releases at the center of the immediate roof and concentrates at the entry corner. Increasing the distance of bedding plane from the roof line will reduce the separation along the bedding plane. Consequently, more vertical stress will transfer to the first layer causing an increase in the vertical stresses at the entry center and corners.

Figure 8.34 shows the effect of bedding plane location on the horizontal stress distribution in the immediate roof. The horizontal stress increases by increasing the distance from 1 to 3 ft of the roof line. Whereas, the magnitude of the induced horizontal stress does not change when the bedding plane is located more than 3 ft from the roof line but distribute over a large area.

Figure 8.35 shows the effect of changing the bedding plane location on the shear stress distribution in the immediate roof. The shear stress zone increases with increasing distance of bedding plane. Increasing the distance of bedding plane from the roof line from 1 to 3 ft leads to increase in the shear stress at the entry corners. On the other hand, when bedding plane goes farther than 4 ft from the roofline, the shear stress area increases but its magnitude does not change much.
Figure 8.33 Effect of bedding plane location on vertical stress distribution
(a) Location of bedding plane = 1ft
(b) Location of bedding plane = 2ft
(c) Location of bedding plane = 3ft
(d) Location of bedding plane = 4ft
(e) Location of bedding plane = 6ft
(f) Location of bedding plane = 8ft

Figure 8.34 Effect of bedding plane location on horizontal stress distribution
Figure 8.35  Effect of bedding plane location on shear stress distribution
(2) Effect on Sliding along Bedding Plane

Figure 8.36 shows the sliding along the bedding plane at different locations measured from the roofline. Sliding has maximum values near the rib sides and reduces to zero at the entry center. Sliding along the bedding plane decreases as the bedding plane goes farther into the roof. Sliding along the bedding plane starts to decrease significantly when the bedding plane is located at more than 3 ft from the roofline. When the bedding plane is located at more than 6 ft from the roofline sliding becomes very small. Therefore, the closer the bedding plane to the roofline, the more it affects the stress distribution within the immediate roof and bolt load.

8.4.4.2 Effect of Coefficient of Friction

Figure 8.37 shows the effect of different coefficient of friction on the bedding plane sliding. It can be seen that sliding along the bedding plane decreases when the coefficient of friction increases. Besides, increasing the coefficient of friction will
increase the shear stress along it. Table 8.4 summaries the effect of different coefficient of friction on the stress distributions in the immediate roof. When sliding along the bedding planes decreases, the horizontal stress concentration around the bedding plane and its magnitude within the center of the entry decreases.

Table 8.5 shows that increase the coefficient of friction improves the roof stability. When the coefficient of friction along the bedding plane increases, separation will decrease, thereby the deflection of the immediate roof decreases. When the bedding plane is located at distance more than 3 ft from the roofline, the change in the coefficient of friction has little effect on the roof stress distribution.

Table 8.4 Effect of coefficient of friction at bedding plane on the stress distribution of the roof

<table>
<thead>
<tr>
<th>Coefficient of friction</th>
<th>Horizontal stress (psi)</th>
<th>Vertical stress (psi)</th>
<th>Shear stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>1,460</td>
<td>375</td>
<td>175</td>
</tr>
<tr>
<td>0.25</td>
<td>1,410</td>
<td>420</td>
<td>190</td>
</tr>
<tr>
<td>0.30</td>
<td>1,362</td>
<td>465</td>
<td>225</td>
</tr>
<tr>
<td>0.35</td>
<td>1,310</td>
<td>510</td>
<td>245</td>
</tr>
<tr>
<td>0.40</td>
<td>1,265</td>
<td>555</td>
<td>270</td>
</tr>
<tr>
<td>0.45</td>
<td>1,215</td>
<td>600</td>
<td>300</td>
</tr>
</tbody>
</table>

Table 8.5 Effect of coefficient of friction at roof stability measures

<table>
<thead>
<tr>
<th>Coefficient of friction</th>
<th>RSF</th>
<th>RD (in)</th>
<th>RWPS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>2.05</td>
<td>1.24</td>
<td>1.21</td>
</tr>
<tr>
<td>0.25</td>
<td>2.86</td>
<td>0.99</td>
<td>1.09</td>
</tr>
<tr>
<td>0.30</td>
<td>2.98</td>
<td>0.87</td>
<td>0.89</td>
</tr>
<tr>
<td>0.35</td>
<td>3.18</td>
<td>0.65</td>
<td>0.74</td>
</tr>
<tr>
<td>0.40</td>
<td>3.25</td>
<td>0.52</td>
<td>0.64</td>
</tr>
<tr>
<td>0.45</td>
<td>3.39</td>
<td>0.37</td>
<td>0.53</td>
</tr>
</tbody>
</table>

8.4.5 Effect of Strata Sequence on Roof Stresses

Because the fully grouted bolt is a passive support, its supporting mechanism is generated by roof strata movements. Therefore, the mechanism of a fully grouted bolt is strongly related to the geological conditions in which the bolt is installed. In order to investigate the stresses and yield distribution in the immediate roof, four models with different strata sequences and strengths have been conducted in the same way as in Section 8.3. A bedding plane is located at 3 ft from the roofline with a coefficient of friction 0.25. The overburden is 600 ft thick and the stress ratio is 3. The bolts are 4 ft long and 6/8-in in diameter. The rock mechanical properties of the four possible geological combinations of the immediate roof are shown in Table 8.1.
8.4.5.1 Effect on Stress Distributions

Figure 8.38 shows the vertical stress distribution in the immediate roof under different strata sequences. Comparing the vertical stress in the immediate roof of roof geology I and II, it is obvious that the magnitude of vertical stress in roof geology II is smaller than that in roof geology I. Moreover, the developed maximum axial load in the fully grouted bolts in roof geology II is much higher than that in roof geology I as mentioned in Section 8.3. When vertical deformation occurs in the first layer or separation occurs at the bedding plane, the fully grouted bolts will transfer from the passive to active bolts. As a result, vertical stress is generated in the first layer and the roof stability is increased. On the other hand, in the case of roof geology III, there is a released stress area extending to 6 ft from the roof line. Moreover, as mentioned in Section 8.3, it is obvious that the developed maximum axial load in the fully grouted bolts is small. These results indicate that the vertical displacement in the first layer and the separation along the bedding plane is very small. As the vertical stress distribution is very similar to the one without the fully grouted bolts, no vertical stress needs to be rebuilt in that layer. It is obvious that both magnitudes of vertical stress and developed axial load in fully grouted bolts in roof geology IV are the highest. This indicates that the deformation occurred in both roof layers in roof geology model IV is the largest among the four geologies types analyzed.

Figure 8.39 shows the horizontal stress distribution in the immediate roof under different strata sequences. Comparing the induced horizontal stresses in roof geology I and II, it is obvious that there are more deformations in roof geology I than that in roof geology II., and as a result more release in the horizontal stress occurs in the immediate roof of geology I. On the other hand, comparing roof geology III and IV, the induced horizontal stress in the immediate roof of geology III is much larger than that in geology IV. There is more deformation occurs in the roof geology IV, so larger release in the induced horizontal stress.

Figure 8.40 shows the shear stress distribution under different strata sequences. Comparing the magnitude of shear stress at the entry corner for roof geology I and II, it is obvious that the shear stress in roof geology I is smaller than that in roof geology II. The deformation occurred at the first layer of roof geology I, which is weaker than the first layer of roof geology II, cause the shear stress to release from the entry corner. On the other hand, comparing the magnitudes of shear stresses of roof geology III and IV, it is obvious that the highest shear stress occurs at roof geology III which has the lowest deformation.

8.4.5.2 Effect on Equivalent Plastic Strain Distribution

Figure 8.41 shows the plastic strain distribution under different strata sequence. The extent of the yielding area is directly related to the roof geology and strata sequence. Figure 8.41 (a) shows the yield zone in the immediate roof of geology I, in which the 1st layer is weak and overlain by stronger one. The plastic strain occurs only in the first layer. Figure 8.41 (b) shows the plastic strain distribution in roof geology II in which the first layer is strong. The plastic strain occurs only at the entry corners. There is a small amount of plastic strain between 3 and 6 ft from the roof line. In roof geology III, there is no
Figure 8.38  Effect of different strata sequence on vertical stress distribution

(a) Roof Geology I

(b) Roof Geology II

(c) Roof Geology III

(d) Roof Geology IV
Figure 8.39 Effect of different strata sequence on horizontal stress distribution
Figure 8.40 Effect of different strata sequence on shear stress distribution
Figure 8.41 Effect of different strata sequence on equivalent plastic strain distribution
plastic strain over the roof, except a very small value close to the entry corner. Conversely, the plastic strain is found over a large area in roof geology IV.

8.4.5.3 Roof Stability

Table 8.6 shows the effect of different strata sequence on the roof stability measures. The roof geology IV has the lowest roof safety factor and the highest roof deflection and weighted plastic strain. Whereas, the roof geology III has the highest safety factor and the lowest roof deflection and weighted plastic strain. The roof deflection in geology IV exceeds the allowable roof deflection which is 1.1 in.

8.4.6 Effect of Different Bolt Lengths on Roof Behavior

It is also important to choose a sufficient bolt length in order to achieve good and economic design. So, in order to investigate the effect of different bolt lengths on roof stability, different models with different bolt lengths, i.e. 3, 4, 5, and 6 ft, were conducted.

<table>
<thead>
<tr>
<th>Roof type</th>
<th>RSF</th>
<th>RD (in)</th>
<th>RWPS</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>3.02</td>
<td>0.95</td>
<td>0.90</td>
</tr>
<tr>
<td>II</td>
<td>4.42</td>
<td>0.46</td>
<td>0.45</td>
</tr>
<tr>
<td>III</td>
<td>4.89</td>
<td>0.34</td>
<td>0.24</td>
</tr>
<tr>
<td>IV</td>
<td>1.24</td>
<td>1.54</td>
<td>1.45</td>
</tr>
</tbody>
</table>

Figure 8.42 shows the effect of installing different bolt lengths on the roof vertical displacement. The roof deflection decreases from 1.4 in to 0.6 in when the installed fully grouted bolt length increases from 3 ft to 6 ft. It is obvious that increasing the bolt length is very effective in reducing the roof deflection. From the beam theory point of view, increasing bolt length increases beam thickness. Beam building concept is a major support mechanism in underground mines where the fully grouted bolts are installed. Due to its full contact between the fully grouted bolts and the surrounding rock, the roof will be much stronger and can resist both horizontal and vertical displacement.

8.4.6.2 Effect on Plastic Strain

Figure 8.43 shows the effect of installing different bolt lengths on the roof plastic strain. Both the size of yielded area and magnitude of plastic strain decrease significantly by installing longer fully grouted bolts. Increasing the bolt length will bind more roof layers together which will enhance the roof strength by increasing its flexural stiffness.

8.4.7 Roof Failure Modes

In the models that represent different geological conditions, a high stress level was applied to the roof and shear failure was expected. Based on the plastic strain distribution for different geological conditions, four shear failure zones with different sizes and shapes are estimated as shown in Figure 8.44.
Figure 8.42 Effect of different bolt lengths on roof vertical displacement

Figure 8.43 Effect of different bolt lengths on roof plastic strain distribution
Figure 8.44 Different modes of failure for different roof types

(a) Failure mode No.1

(b) Failure mode No.2

(c) Failure mode No.3
8.4.7.1 Shear Failure Mode No.1 (Roof Geology I)

For this type of roof condition, the entry corners are the weakest part because of the stress concentration and high magnitude of plastic strain as shown in Figures 8.38 (a) to 8.41 (a), i.e., roof failure could be initiated from there. A large yielded area extends through the first layer (the weakest layer). The second layer is in elastic stage as no yielding occurs in it as shown in Figure 8.41 (a). Failure could occur at the first layer and stop by the second one as shown in Figure 8.44 (a).

8.4.7.2 Shear Failure Mode No.2 (Roof Geology II)

This type of failure could occur in a roof where the first layer is stronger than the overlying one. Based on the plastic strain distribution shown in Figure 8.41 (b), there is plastic strain concentration at the entry corners and extend for about 0.75 ft from the roof line. There is no yield occur at the part of the roof near the roof line around the entry center and until 3 ft height from the roof line, which means the rock material is still in elastic state. Accordingly, a minor failure could occur close to the entry corner as shown in Figure 8.44 (b).

8.4.7.3 Skin Failure at Entry Corners (Roof Geology III)

Based on the plastic strain distribution of roof type III (Figure 8.41 (c)), there is a very small part of the roof close to the entry corners that has been yielded with a small magnitude of plastic strain, while the majority of the roof was in elastic conditions. At the entry corners only skin failure could occur.

8.4.7.4 Shear Failure No.3 (Roof Geology IV)

For this type of roof conditions, where the roof material is weak, plastic strain and a large yielded area extends throughout the immediate roof as shown in Figure 8.41 (d). As a result failure could initiated from the entry corners and extend through the rest of the roof causing arch shape failure as shown in Figure 8.44 (c).

8.5 Design Guidelines for Fully Grouted Roof Bolts

Design of the fully grouted bolts should consider the stability of both roof and bolts. Moreover, different modes of failure of the roof and fully grouted bolts should be considered in order to obtain a suitable design. Based on the analysis results, regression equations were established in order to estimate the roof and bolt stability. According to the geological type (I, II, III, and IV) discussed in this chapter, three types of regression equations are developed. Four regression equations were developed for each type of roof. These equations can estimate the roof and bolt stability measures. The stability measures for the roof are: bolt shear load safety factor, or grout/rock interface safety factor. The controlled variables are roof type, overburden depth, bolt length, bolt diameter, and entry width.

According to the analysis results, the fully grouted bolt is mainly subjected to axial loads, shear loads, and bending moments. In addition, there will be shear stress at
the grout/rock interface. The fully grouted bolt plays important roles in supporting the immediate roof of underground coal mines due to the following reasons:

1. Because of the full contact of the fully grouted bolt with the surrounding rock and its stiffness, the fully grouted bolt is able to increase the rock stiffness by joining the roof layers together – beam building effect. The deformations result from sliding and separation along the bedding planes and roof sagging, are reduced due to the increase of roof stiffness.

2. The fully grouted bolts restore the entry center to a triaxial state of stress by increasing its vertical stress at the relieved zone area at the entry center. Therefore, the rock strength can be increased by increasing the vertical stress in the stress relieved zone and providing the roof with vertical confinement.

3. The fully grouted bolt reduces the potential for shear failure because it reduces the shear stress concentration at the entry corners where the failure starts.

4. The fully grouted bolt fills up the hole such that the shear stiffness of the grout/rock interface resists the developed shear displacement along the bedding plane.

It is obvious that reinforcement of the fully grouted bolts is passively generated by rock movements. Therefore, the effect of fully grouted bolt is closely related to the geological conditions of the roof strata.

8.5.1 Design Procedures

Figure 8.45 shows the design procedures for the fully grouted bolts. The proposed design procedures for the fully grouted bolts consist of three main steps. In the first step the roof is classified based on the Young’s modulus of the first and the second roof layers. A set of regression equations, to estimate the roof and bolt stability measures, are assigned for each roof type.

Based on the defined roof type in Step 1, the roof and bolt stability measures are estimated for a given entry width, overburden depth, and an assumed values of bolt length and bolt diameter in Step 2.

In Step 3, the estimated roof and bolt stability factors are compared with the corresponding critical values. If the comparison shows a safe roof and bolt conditions, the assumed values for bolt length and bolt diameter will be accepted as appropriate bolt design. Otherwise the bolt length and/or the bolt diameter are increased. And Step 2 and Step 3 will be repeated with the new values of bolt length and diameter. These procedures will repeat until a safe roof and bolt design are obtained.

8.5.1.1 Roof Classification and Possible Type of Roof Failures

As demonstrated earlier the stability of roof or bolt is mainly affected by the roof geology. A successful design for the fully grouted bolt should consider different roof types. Hence, the design criteria for the fully grouted bolting should vary according to roof conditions.
Figure 8.45 Design procedures for fully grouted bolt
Roof type I represents a weak layer overlain by a relatively strong one. In this roof type, the Young’s modulus of the first layer is less than four times that of the coal. Finite element modeling was used to investigate the effect of bolt parameters, entry width and overburden depth on the stability measures of roof type I. From the possible failure modes, the bolt length for the design formulas can be estimated. For failure mode No.1, which is based on roof type I, the failure will start from the entry corners followed by the yielded area extending throughout the first layer (weakest layer) and then stopped at the second layer (the strongest one). For this kind of roof failure, the fully grouted bolt is designed to suspend the weak layer to the overlain strong layer. In this case, the fully grouted bolts will carry the dead weight of the first layer. Hence, the bolt length will be equal to the height of the first layer plus 1-ft.

Roof type II, which represents a strong layer overlain by a relatively weak one. In this kind of roof, the Young’s modulus of the first layer is more than four times and less than eight times that of the coal, whereas that of the second layer is less than four times that of the coal. From the failure mode No.2 discussed in the previous section, which is based on roof type II, a small failure occurs at the entry corners, the roof of which is still in elastic conditions. For this kind of roof, the bolt length should be equal or more than the thickness of the first layer. However, the bolt length needed by roof type II is much shorter than that needed by roof type I.

Roof type IV, which represents a very weak immediate roof for the first and the second roof layers, is very often encountered in the U.S. underground coal mines. In this kind of roof, the Young’s modulus of the first and second layers is less than four times that of the coal. From failure mode No.3 discussed in the previous section for roof type IV, a large area of the roof will yield and failure will extend through the weak immediate roof. In this kind of roof, the strong main roof is not located within the bolting horizon of the fully grouted roof bolts. Therefore, application of the suspension theory is not a practical solution. Hence the beam building concept will be the supporting mechanism for that type of roof. Increasing the bolt length will increase the roof stiffness by decreasing the later and axial displacement developed in this roof type. Nevertheless, the bolt lengths needed by roof type IV is much longer than those needed by roof type I or II.

For example, Figure 8.46 shows the roof safety factor and overburden depth under different roof types. It can be seen that the safety factor for roof type II is much higher than that for roof type I. Besides, comparing with the results of roof types I and II, the safety factor for roof type IV is low.

8.5.1.2 Regression Models
Finite element analysis was conducted under several combinations of parameters for each roof type to estimate the stability measures of roof and bolt. For each roof type, about 45 models were conducted with a total of 135 finite element models for three roof types. According to the stability analysis for roof and bolt, the stability measures have been determined for each model at different input parameters.
Linear regression analysis was performed by using JMP software (JMP4, 2000). Equations 8.7, 8.8, 8.11, 8.12, 8.15, and 8.16 describe the relationship between overburden depth, entry width, bolt length and bolt diameter and roof stability measures. Equations 8.9, 8.10, 8.13, 8.14, 8.17, and 8.18 relate the overburden depths, entry width, bolt length, and bolt diameter to the bolt stability measures. All the numerical constants in these equations are regression constants. Though the regression analysis, the following regression equations are obtained:

**Roof Type I:**

\[
RSF = 4.59 - 5.70 \times 10^{-4} \times H + 0.191 \times L + 2.36 \times D - 0.209 \times W
\]

\[
RD = -0.163 + 6.33 \times 10^{-4} \times H - 0.159 \times L - 1.01 \times D + 0.115 \times W
\]

\[
BSHL \_ SF = 2.74 - 2.19 \times 10^{-4} \times H + 7.45 \times 10^{-2} \times L + 0.427 \times D - 0.103 \times W
\]

\[
ISHS \_ SF = 1.52 - 3.40 \times 10^{-4} \times H + 0.106 \times L + 0.627 \times D - 5.33 \times 10^{-2} \times W
\]

Correlation coefficients for the above equations are 0.961, 0.944, 0.945 and 0.968, respectively.

**Roof Type II:**

\[
RSF = 7.24 - 8.73 \times 10^{-4} \times H + 0.282 \times L + 1.67 \times D - 0.251 \times W
\]

\[
RD = 0.264 + 2.96 \times 10^{-4} \times H - 6.75 \times 10^{-2} \times L - 0.467 \times D + 3.56 \times 10^{-2} \times W
\]

\[
BSHL \_ SF = 2.22 - 4.60 \times 10^{-4} \times H + 0.151 \times 10^{-2} \times L + 1.09 \times D - 7.80 \times 10^{-2} \times W
\]

\[
ISHS \_ SF = 2.25 - 3.30 \times 10^{-4} \times H + 0.129 \times L + 0.960 \times D - 6.58 \times 10^{-2} \times W
\]

Correlation coefficients for the above equations are 0.965, 0.937, 0.959 and 0.953, respectively.
Roof Type IV:

\[
RSF = 3.74 - 2.70 \times 10^{-4} \times H + 9.15 \times 10^{-2} \times L + 1.86 \times D - 0.229 \times W \tag{8.15}
\]
\[
RD = 1.14 + 5.62 \times 10^{-4} \times H - 0.305 \times L - 1.08 \times D + 0.106 \times W \tag{8.16}
\]
\[
BSHL\_SF = 1.02 - 5.00 \times 10^{-4} \times H + 7.91 \times 10^{-2} \times L + 1.39 \times D - 4.97 \times 10^{-2} \times W \tag{8.17}
\]
\[
ISHS\_SF = 0.341 - 5.60 \times 10^{-4} \times H + 0.167 \times L + 1.71 \times D - 5.04 \times 10^{-2} \times W \tag{8.18}
\]

Correlation coefficients for the above equations are 0.969, 0.954, 0.938 and 0.968, respectively.

where, \(RSF\) = roof safety factor, \(RD\) = roof deflection (in), \(BSHL\_SF\) = bolt shear load safety factor, \(ISHS\_SF\) = grout/rock interface safety factor, \(H\) = overburden depth (ft), \(W\) = entry width (ft), \(L\) = bolt length (ft), and \(D\) = bolt diameter (in)

Roof type III represents strong roof condition. The Young’s modules of two roof layers are more than four times and less than eight times that of the coal. The stress analysis and plastic strain distribution for that kind of roof is discussed in the previous section. Based on this analysis, that type of roof can support itself. According to the US regulations any coal mining activities should be done under a supported roof. Therefore, roof type III could have the minimum reinforcement.

8.5.2 Design Limits

The regression equations 8.7-8.18 are based on a limited number of finite element models. Because of this limitation, the studied parameters were valid within certain ranges. Therefore, these equations should be used within the range of the studied parameters.

8.5.2.1 Overburden Depth

During the analysis of the finite element models different overburden depths were used to simulate shallow and deep coal mines. The overburden depth that used in the finite element models ranges from 400 to 1,200 ft.

8.5.2.2 Entry Width

There are three entry widths currently adopted by the Eastern U.S. underground coal mines: 16, 18, and 20 ft. Due to the operational conditions, there is a specific roof bolt patterns used in these mines. A bolt pattern 4, 4, 4 (bolt spacing in a row), and 4 ft (row spacing) is used for entry width 16 ft. a bolt pattern 3, 4, 4, 4, and 3 ft is used for entry width 18 ft. a bolt pattern 4, 4, 4, 4, and 4 ft is used for entry width 20 ft. The two entry widths that were used to obtain the above-listed regression equations are 18 and 20 ft.

8.5.2.3 Bolt Length

There are different bolt lengths that are used to support the roof in underground coal mines. The smallest bolt length that could be used is 3 ft long. For the proposed design formula the following bolt lengths were used: 3, 4, 5, 6, and 8 ft long.
8.5.2.4 Bolt Diameter

Because the bolt stability is directly proportional to the bolt load capacity, under the same steel grade, bolt capacity increases with bolt diameter. The common bolt diameters were used in the numerical modeling are 5/8, 6/8, and 7/8-in.

8.5.2.5 Young’s Modulus of Roof Layer

The Young’s modulus of roof was expressed as a ratio of the Young’s modulus of the coal seam. The ratio ranged from 1 to 8.

8.5.2.6 Models Constants

During numerical analysis, some parameters were held constants. The mining height was 7 ft. The friction coefficient along the bedding planes was 0.25. The hole roughness represented by the coefficient of friction at the grout/rock interface was 0.5. The horizontal stress applied to the models was three times the vertical stress. The hold diameter was 1 in. The resin properties were constants during the finite element analysis and represented by the Young’s modulus of the resin.

8.5.3 Verification of Proposed Design Guideline

The development method for the fully grouted bolt design was verified by case studies. These case studies cover different types of roof including types I, II, and IV, and different roof failures.

8.5.3.1 Case Study Representing Roof Type I

A case study representing a mine roof type I was studied. The immediate roof is composed of two types of rock; the first layer is dark gray shale whereas the second layer is light gray sandstone. Field observations showed that, there was no roof falls noticed at that mine and only skin failure was noticed. The overburden at the test site is 400 ft and the entry width is 20 ft. The 4 ft-long of fully grouted bolt with spacing 4, 4, 4, 4 (bolt spacing in a row), and 4 ft (row spacing) and a diameter of 3/4-in were installed.

Table 8.7 shows the stability measures predicted by the regression equations 8.7 to 8.10 for that roof type. It is noticed that the roof has a high safety factor, both the bolt shear load and interface shear stress safety factor are greater than one.

8.5.3.2 Case Study Representing Roof Type IV

A case study for cutters and massive roof falls was investigated (Peng, 1999) (33). Cutters and massive roof falls started as soon as the development began. The cutter roof initiated and developed into roof falls. The entry width was 18 or 20 ft wide. The 4 ft long fully grouted bolts (3/4-in in 1-in hole) were installed in 4 × 4 ft pattern with side

<table>
<thead>
<tr>
<th>Entry width (ft)</th>
<th>RSF</th>
<th>RD (in)</th>
<th>BSHL SF</th>
<th>ISHS SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>2.63</td>
<td>1.00</td>
<td>1.20</td>
<td>1.21</td>
</tr>
</tbody>
</table>
bolts 3-4 ft from the entry corners. The overburden depth varied from 160 to 435 ft deep. The immediate roof was 9-11 ft thick and weak because it consisted of laminated black and gray shale, and sandstone. The immediate roof was highly subjected to weathering. In some entries roof falls were 9 ft high measured from the roof line. No bolt failure was observed in this case.

According to the description of roof failure and roof geology of that case study, the roof can be classified as roof type IV. Table 8.8 shows the stability measures predicted by the regression equations 8.15 to 8.18 for entry widths 18 and 20 ft. It is obvious that the predicted roof deflection of entry widths 18 and 20 ft were greater than the permissible roof deflection suggested by Chen, et al, 2003 (30). On the other hand the safety factor for the fully grouted bolts was greater than 1.0 which means no bolt failure was expected. This prediction results are in good agreement with the underground observations. The roof safety factor was less than 1.0 in case of the entry width 20 ft.

Table 8.8 Stability measures for case study of Roof Type IV

<table>
<thead>
<tr>
<th>Entry width (ft)</th>
<th>RSF</th>
<th>RD (in)</th>
<th>BSHL_SF</th>
<th>ISHS_SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>1.27</td>
<td>1.30</td>
<td>1.29</td>
<td>1.20</td>
</tr>
<tr>
<td>20</td>
<td>0.80</td>
<td>1.50</td>
<td>1.15</td>
<td>1.07</td>
</tr>
</tbody>
</table>
CHAPTER 9

SUMMARY

9.1 Prediction of Roof Geology

One of the main tasks of this research is to develop a method for predicting the roof geology based on the drilling parameters obtained during normal roof bolting operation. From the results of a series of laboratory and underground tests, a data interpretation methodology for identifying the geological features in the mine roof strata, for example the location and size of void/fracture and estimating rock strength, has been developed. In addition, a new software package, MRGIS, has been developed to allow mine engineers to make use of the large amount of roof drilling parameters for roof support design.

9.1.1 Data Interpretation Methodology

A system of quantitatively detecting voids/fractures and estimating roof rock strength in the entry roof using roof bolter drilling parameters has been developed. From the results of a series of underground and laboratory tests, the following conclusions can be made:

9.1.1.1 Effect of Drilling Setting and Machine Conditions on Drilling Parameters

As the data collection system is designed not for prediction of roof geology but for drilling control, drilling parameters which can be measured are the parameters in the hydraulic systems. This means that the drilling parameters contain not only for drilling rock but also for running the machine itself. First of all, one needs to know how much the drilling parameters consumed for running the machine itself and how much impact different drilling settings and machine conditions have on it.

The data for compensation runs make it clear that different drilling settings and machine conditions have no obvious impact on both the magnitude and trend of feed pressure consumed for running the machine. These results indicate that feed pressure curve is almost constant at any drilling settings and conditions. So, once feed pressure for compensation run is measured, it is easy to eliminate the effect of machine on feed pressure when drilling in rock. Moreover, it can be said that the magnitude of feed pressure reflects the strength of rock directly when drilling in rock. On the other hand, different rotation rates and machine conditions (ex. oil temperature) have obvious impact on the magnitude of rotation pressure. In addition, comparing with the data when drilling in rock, their impacts are too large to be ignored. Therefore, the effects of rotation rate and machine conditions have to be taken into account if the strength of roof rock is estimated based on the magnitude of rotation pressure.
9.1.1.2 Void/Fracture Detection

From the drilling data, it can be seen that the feed pressure changes dramatically to form a valley around the location of a void/fracture. It is also recognized that the rotation pressure changes around the location of a void slightly. But the magnitude of change of rotation pressure is much smaller than that of feed pressure. The other drilling parameters do not change at the presence of voids. Therefore, it can be concluded that feed pressure is the most relevant drilling parameter for the existence of voids/fractures.

When the penetration rate is controlled, the feed pressure tends to drop to the level of drilling in the air when a void is encountered. This is the major criterion for void prediction. But, it was also observed that sometime the bottom of feed pressure valleys do not reach the level when drilling in the air. Therefore, in order to enhance the prediction accuracy, a supplemental prediction criterion is developed considering not only the magnitude of feed pressure but also the shape of feed pressure valley. From the results of laboratory tests and underground tests, the prediction results show that a very high prediction percentage have been achieved for the 1/8-in or larger voids. But the 1/16-in void does not cause an obvious change in not only feed pressure but also all other drilling parameters. It seems that there is a limitation of the void size that can be detected by the current system. In other words, a void of 1/16-in or smaller cannot be detected by the system developed.

Mining engineers are interested in not only the void location but also its size in the roof strata when designing roof supports. From the result of laboratory tests, it can be seen that the width of the plateau at the bottom of feed pressure valley is much closer to the size of the void. It can be considered that the larger the size of the void/fracture is, the more possible it is to form a plateau at their bottom of feed pressure valley and the more precisely it can be predicted. On the other hand, it seems to be difficult to determine the void size when it is smaller than 1/2–in. because of the resolution of bit position sensors. The resolutions of current bit position sensors are 0.1485 in and 0.1593 in for the two stages of mast feed, respectively. So, the accuracy of predicting the void size using the drilling parameters measured by the current data collecting system is too low to be acceptable for the voids smaller than 1/2-in.

9.1.1.3 Estimation of Rock Strength

From the drilling data, it can be seen that different strengths of roof rocks have obvious impact on the magnitude of feed pressure. The harder the roof rock is the larger the magnitude of feed pressure is. On the other hand, the impact of rock strength on the magnitude of rotation pressure is not so clear cut. Besides, as mentioned above, the magnitude of rotation pressure consumed for running the machine itself changes dramatically with the change of rotation rate and machine conditions. Therefore, feed pressure is the most sensitive drilling parameters when the strength of roof rock changes under the current drilling and data collection system.

Both penetration rate and rotation rate have obvious impact on the magnitude of feed pressure. The higher the penetration rate and/or the lower the rotation rate are the larger the magnitude of feed pressure is. Moreover, the harder the roof rock is the larger
the impacts of different penetration and rotation rates on the magnitude of feed pressure are. Therefore it can be seen that not only the magnitude of feed pressure but also the impacts of different drilling settings are related to roof rock strength. There seems to be a good relationship between feed pressure and penetration rate or rotation rate for rock strength. In other words, the slope of feed pressure-penetration rate curve and/or feed pressure-rotation rate curve is a good indicator of roof rock strength.

Here, note that this roof bolter can drill a 52-54-in deep hole in one cycle using two feed stages, carriage and mast stages. There is about 7-10 output units difference between feed pressure in the carriage stage and that in the mast stage. This effect makes a wide variance in the distribution of feed pressure-penetration rate data points. In order to eliminate the machine effect, the net feed pressure which is the feed pressure for compensation run subtracting from the feed pressure when drilling in rock, is recommended for estimating rock strength instead of feed pressure.

The results obtained clearly show that the magnitude of net feed pressure correlates well with the rock strength and both penetration rate and rotation rate have obvious impact on the magnitude of net feed pressure. Therefore both parameters have to be considered when roof geology is predicted based on the magnitude of net feed pressure. From the relationship among net feed pressure, penetration rate and rotation rate, the boundary planes for classification of roof rock are proposed and verified based on the drilling data obtained. The strength of roof rock can be determined and/or classified based on the magnitude of net feed pressure because it takes both the effects of penetration rate and rotation rate into account when both penetration rate and rotation rate are controlled. In other words, the relationship among net feed pressure, penetration rate and rotation rate is a good indicator for estimating the strength of roof rock. The strength of roof rock can be estimated based on the location of data points in the net feed pressure-penetration rate and rotation rate system.

9.1.2 Data Visualization and Data Base Software

In a production environment, manually interpreting and managing the collected drilling parameters are not practical since hundreds of holes will be drilled during a production day. A new software package, MRGIS (Mine Roof Geology Information System), has been developed to allow mine engineers to make use of the large amount of roof drilling parameters for predicting roof geological properties automatically. This system provides data import, data management, data interpretation, and data display functions to meet the requirements of mine engineers.

A set of rules described above have been implemented in MRGIS for predicting voids/fractures and estimating rock strength in entry roof.

In MRGIS, three roof geology visualization methods are provided. The first is 2D+1D display and the second is 2D+2D display. It allows the user to select multiple holes (any cross section) at the same time and display the results of interpolation. The third visualization method is 2D+3D display. This feature can display a 2D mine map and
a 3D figure of roof drill holes at the same screen. It allows the users to do a “virtual walk” through the underground spaces to check out roof geological properties at different locations.

### 9.2 Roof Bolting Mechanism and Design Guidelines

In this research, tensioned and fully grouted roof bolting have been modeled using finite element modeling technique. Attempt has been made to model both roof bolting systems realistically by considering the physical dimension of bolts, bedding planes and in-situ stresses. According to the results of numerical analysis and field data, the following conclusions can be made about the modeling and the mechanisms of the tensioned and fully grouted roof bolting systems.

#### 9.2.1 Tensioned Roof Bolting

1. A finite element model using ABAQUS has been developed for the tensioned bolting design. In this mode, the tensioned bolt is modeled realistically by considering the physical dimension of bolts, excavation sequence, bolt installation procedure, bolt pre-tension, bedding planes and in-situ horizontal stress. Excavation sequence and bolt installation procedure are important to obtain the true bolt load increase after bolt installation to evaluate the effect of pre-tension on roof stability.

2. The tensioned bolts play four roles in supporting the immediate roof: (a) to reduce the deformation and increase the residual strength of yielding zone in the immediate roof so that the bolted strata will have the ability not only to support itself but also to receive the load from, and limit the deformation of, the upper strata; (b) to close the separations around the bearing plates within 3 ft of the immediate roof so as to increase both the strength of the affected roof layers and the stiffness of the whole bolted strata such that any subsequent vertical displacement is resisted by the high installation load in the bolt due to pre-tension; (c) to provide in the immediate roof within 3 ft of the roof line a compressive zone which can generate frictional forces along the fractures and bedding planes near the bearing plates.- beam building effect; (d) to suspend the gravity load and those vertical loads caused by rock expansion and bending of roof layers subjected to the horizontal stress. The study also showed that the tensioned bolts are subjected mainly to axial load, which is generated by the vertical displacement caused mainly by the horizontal stress. The compressive zone in the immediate roof caused by the bolt pre-tension are restricted to the area near the bearing plate except a triangle zone between two adjacent bolts, and between side bolts and roof corners.

3. The bedding planes greatly change the stress distribution in the roof above the entry if sliding and/or separation occurs: the vertical stress in an anchored zone
above the center of the entry reduce to very low values; the horizontal stress is concentrated more in the first layer of the roof; and the shear stress around the entry corner is also reduced. Moreover, the bedding plane close to the roof line has more effect on the stress distribution over the immediate roof than that high above the roofline. A bedding plane 3-4 ft within the roof line has the largest effect on the stress distribution over the entry and the load in the bolt.

9.2.1.2 Design Guidelines for Tensioned Roof Bolting

A new design approach is proposed for the tensioned bolting design using ABAQUS. The procedure has four steps to determine bolt length, bolt pre-tension, bolt spacing and bolt diameter. According to the bedding plane location, the immediate roof is classified into 4 types. This classification scheme will help reduce the number of models to be run in order to build a database for the tensioned roof bolting design using the new proposed approach. A computer program was developed for tensioned bolting design using the output data from numerical modeling. Recommendations for tensioned roof bolting design are as follows;

(1) The tensioned bolts are suitable for a weak roof under a low stress level. Since a tensioned bolt transfers its load to the anchors, it is important that the bolt is anchored in a stable roof layer. The tensioned bolts are not suitable for the roof with large deformation after bolt installation.

(2) If the roof yielding occurs only around the entry corners, possible failures would occur around the entry corners. In this case, the side bolts should be installed within 2-3 ft from the entry corners to prevent cutter failure.

(3) If the roof yielding develops over the entry, both the entry corners and center of the entry may fail. In this case, emphasis should be on the entry corners because both the horizontal and shear stresses are concentrated on these areas. The distance between the side bolt and the entry corner should be adjusted so that the bolts extend 2 ft beyond the potential failure plane both to anchor the bolt in a stable area and to limit the roof displacement in case the failure occurs.

(4) Basically, the bolt length should increase as the overburden depth and horizontal stress level increase, and the bolt length required by roof type I and III is shorter than that by roof type II and IV.

(5) If the roof is subjected to high horizontal stress and the immediate roof has multiple bedding planes. A large diameter bolt should be considered to prevent either bolt yielding or bolt breaking.

9.2.2 Fully Grouted Roof Bolting

Based on the strata sequence, roof geological conditions, and roof modes of failure, four roof types were proposed; roof type I, II, III, and IV. To assess the roof stability, the roof safety factor (RSF) and the roof deflection (RD) were used as roof stability measures. To assess the bolt stability, the bolt shear load safety factor (BSHL_SF) and the interface shear stress safety factor (ISHS_SF) were used as bolt stability measures. For each roof type, four regression equations were proposed in order to estimate the roof and bolt stability measures.
9.2.2.1 Mechanism and Functions of Fully Grouted Bolts in Supporting the Immediate Roof in Underground Coal Mines

(1) Because of the full contact of fully grouted bolt with the surrounding rock and its high stiffness, the fully grouted bolt is able to increase the rock stiffness by combining the roof layers together-beam building effect. The deformation results from sliding and separation along the bedding planes and roof sagging is reduced due to the increase of roof stiffness.

(2) The fully grouted bolts restore the entry center to a triaxial state of stress by increasing its vertical stress at the stress relieved zone area at the entry center. Therefore, the rock strength can be increased by increasing the vertical stress in the stress-relieved zone and providing the roof with vertical confinement.

(3) The fully grouted bolts reduce the potential for shear failure because it reduces the shear stress concentration at the entry corners where the failure starts.

(4) The fully grouted bolt fills up the hole such that the shear stiffness of the grout/rock interface resists the developed shear displacement along the bedding plane.

(5) The locations of the bedding planes have a great effect on the induced roof stresses. The farther the bedding plane from the roof line, the more vertical stress is transferred to the first layer. The horizontal stress concentration, at the first layer, increases with increasing bedding plane distance from the roofline until it reaches 3 ft. Thereafter the farther the bedding planes the less horizontal stress in the first layer but more concentration around the bedding plane. The effect of bedding plane location is significant until it is 3 ft from the roofline. Its effect vanishes when the bedding plane is located at a distance more than 8 ft from the roofline.

(6) Regarding the fully grouted bolt load and its modes of failure: the shear load of fully grouted bolts is mainly caused by the presence of bedding planes and affected by their location; the axial load of fully grouted bolts is mainly caused by the vertical deformation of roof strata; the shear stress at grout/rock interface is mainly affected by the surrounding rock properties; axial bolt failure rarely occurs; due to the high ultimate strength and high modulus of elasticity of the steel bolt, it can sustain large loads; low coefficient of friction along the bedding planes can maximize the sliding and as a result increase the possibility of bolt shear failure. In case of high overburden depth and the existence of bedding planes at 2 to 3 ft from the roof line there is a possibility for shear failure of bolt; and in case of weak immediate roof and short bolts, the shear stress at the grout/rock interface could exceed its shear capacity causing shear failure to occur and thus bolt slippage.

9.2.2.2 Design Guidelines for Fully Grouted Roof Bolting

The immediate roof has been classified into four roof types, I, II, III, and IV, based on different strata sequences and different modes of failure. Four regression equations were developed for each type of roof to estimate the roof and bolt stability. The
regression equations were developed for each type of roof to estimate the roof and bolt stability. The regression equations relate the roof stability measures to overburden depth and entry width, and assumed values of bolt length and bolt diameter. The estimated roof and bolt stability measures are compared with the corresponding critical values. If the comparison shows a safety roof and bolt conditions, the assumed values for bolt length and bolt diameter will be accepted as appropriate bolt design. Otherwise the bolt length and/or the bolt diameter are increased, and the roof and bolt stability measures are reevaluated with new values of bolt length and diameter. These procedures are repeated until a safe roof bolt design is obtained. Recommendations for the fully grouted bolting design are as follows:

(1) Longer bolts should be installed for weak immediate roof (roof type IV), so that less shear stress will develop along the grout/rock interface and slippage failure could be avoided.
(2) Longer bolts should be installed for high overburden depth and high in-situ horizontal stress, so that less bolt shear load and shear stress on the grout/rock interface would be developed.
(3) Shorter bolts are required for roof type I and II compared with those for roof type IV.
(4) Loading capacity is a controlling factor in successful rock reinforcement design. It can be controlled by the bolt diameter.
(5) For full utilization of the fully grouted bolts they are most suitable for supporting highly stressed roof strata with large deformation (weak roof).
CHAPTER 10

CONCLUSIONS

(1) Prediction of Roof Geology

A method for void detection / rock strength estimation has been developed using the HDDR dual roof bolter made by J.H. Fletcher & Co. As for void/fracture detection, the trend that feed pressure drops down to the level of drilling in the air when a void/fracture is encountered is used. On the other hand, the rock strength can be determined and/or classified by boundary planes defined in the net feed pressure-penetration rate-rotation rate system.

Since the method is sensitive to individual roof bolter design, it is necessary to calibrate the machine parameters when it is applied to a new roof bolter.

(2) Development of Roof Bolting Design Criteria

Numerical simulations were performed to investigate the mechanism of modern roof bolting systems: tension and fully grouted bolts. Based on the analysis of the mechanisms of both bolting systems and failure modes of the bolted strata, roof bolting design criteria and programs for modern roof bolting systems have been developed.
REFERENCES


APPENDIX A

ENERGY BENEFITS

<table>
<thead>
<tr>
<th>Energy Source</th>
<th>Current Technology (BTU/yr/unit)</th>
<th>Proposed Technology (BTU/yr/unit)</th>
<th>Energy Savings (BTU/yr/unit)</th>
<th># of units in 10 years</th>
<th>Cumulative Energy Savings (BTU/yr/unit)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oil/Gasoline/Diesel</td>
<td>(a)</td>
<td>(b)</td>
<td>(c = a - b)</td>
<td>(d)</td>
<td>(e = c × d)</td>
</tr>
<tr>
<td>Natural Gas</td>
<td></td>
<td></td>
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<tr>
<td>Coal</td>
<td>2,000</td>
<td>2.63 × 10^8*</td>
<td></td>
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<tr>
<td>Electricity*</td>
<td>2,000</td>
<td>6.50 × 10^10**</td>
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<tr>
<td>Other energy</td>
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<tr>
<td>Total per unit</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

* According to MSHA statistics, the annual clean-up cost of roof falls is in excess of $50 million. In view of the required equipment for roof fall clean-up operation, it is estimated that approximately 10% or $5 million of the total annual cost are attributable to electrical power cost. Assuming an average electricity cost of $0.10 per kilowatt hour (KWH), the annual electricity consumption for roof fall clean-up is 50,000,000 KWH or 525,000,000,000 = 5.25 × 10^11 BTU/year.

** Since the labor cost accounts for approximately 50% of the coal production cost, it is estimated that the annual labor cost for roof fall clean-up is approximately $25 million or 156,250 labor days lost (assuming an average wage of $160 per day). Since the average productivity of a US coal miner is 38 tons/day, the total coal production lost is

\[38 \times 156,250 = 5,937,500 \text{ tons/year}\]

Assuming the average heating value of underground coal is 11,000 BTU/lb, the total energy lost due to lost production is

\[11,000 \times 2,000 \times 5,937,500 = 130,625,000,000,000 \approx 1.3 \times 10^{14} \text{ BTU/year}\]