

**COMPREHENSIVE DESIGN METHODOLOGY FOR COAL MINING UNDER
COMPETENT SANDSTONE ROOF**

by

Cale DuBois

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ABSTRACT

This thesis presents a logical design methodology for coal mine extraction optimization under massive sandstone roof as developed through a case study analysis of the Quinsam Coal 4 South mine, a shallow underground room and pillar mine with a massive sandstone roof. This research is intended to guide Quinsam Coal and other coal mines globally in efforts to develop or optimize coal extraction and address the geomechanical challenges presented by massive sandstone roof. In this thesis, the tools required to facilitate effective site characterization, ground support design, excavation stability, pillar design, environmental risk management and mining method optimization are presented, as part of a comprehensive design methodology.

Guidelines for pillar design are presented based on software assisted gravity-wedge analysis, and review of empirical and analytical design methods. Tools for addressing temporal change in pillar size, shape and stress as well as pillar jointing effects are provided. Pillars are designed to accommodate stresses and strains arising from the known range of overburden depths.

An optimized non-caving checkerboard partial pillar extraction method is presented to mitigate environmental risk, address the poor and unpredictable caving mechanics of the massive sandstone roof and provide adequate coal extraction. Modeling of in-line pillar mining and checkerboard partial pillar mining methods was completed with Examine^{TAB}, a pseudo-3D displacement discontinuity program in support of checkerboard partial pillar mining. Instructional training is required with any modification in mining methods or conditions to apprise the underground workforce on the technical details of the mine design and the importance of adhering to the standards thereof.

Using this research work and analysis of the 4 South mine as a backdrop, the design of coal mines under massive sandstone roof is facilitated. The application of the design methodology to the 4 South mine illustrated serves as a 'terms of reference' document for other professionals addressing similar geotechnical and environmental challenges to design safe coal pillar extraction under a massive sandstone roof.

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CHAPTER 1 INTRODUCTION

At a coal mining operation, a mine should be respected for its unique qualities and characteristics which may vary substantially within a coal basin. A logical design methodology guides the engineer through the important design questions and acquisition of critical data to arrive at a comprehensive and safe plan for coal extraction. A design methodology gives respectful and thorough treatment of site characterization, ground support design, coal pillar design, excavation stability, environmental risk management and mining method selection. Design must be treated respectfully and thoroughly to reduce operational and environmental risk and improve project economics.

A comprehensive design methodology for coal extraction under massive sandstone roof is presented as applied to the optimization of the Quinsam Coal 4 South mine - a shallow depth coal mine with competent sandstone floor and roof, high-sulfur coal and environmental risk associated with water management and waste management from the coal washing process. The tools and reference information contained in this thesis intends to guide the mining professional through key aspects of the design process such that the complex and challenging geotechnical, geological and environmental issues are appropriately addressed and a work plan is developed to either increase the efficiency and safety of an existing coal mine or to gain a permit for a prospective coal mine.

This thesis intended to answer six study questions:

1. What is the safest, productive, and cost effective ground support strategy?
2. What are the key considerations for an effective site-specific pillar design?
3. What is the safest, productive and environmentally protective pillar extraction method?
4. What geotechnical instrumentation is best suited to guide mine planning and increase safety?
5. What is the critical roof span and cavability for massive sandstone roof?
6. What are the primary environmental risks associated with the 4 South mine?

Fieldwork for this study was completed between May 2002 and December 2005 and in July 2003 at the Quinsam Coal Mine on Vancouver Island. Rock mechanical testing and instrumentation fabrication was completed at the Geomechanics laboratory at the University of British Columbia during the same time period.

This thesis is divided into nine chapters. Chapter two presents a geological and operational overview of the Quinsam Coal Mine, and more specifically, the 4 South coal mine with a focus on coal quality and seam characteristics, sedimentation and structure.

Since 2005, Quinsam Coal Corporation has put the mine on indefinite stand-by status due to poor mine economics, waste management challenges and coal quality issues.

Specifically, blending at a ratio of 1:6 is required of the higher sulfur No.3 seam coal mined from the 4 South mine to the No.1 seam coal from the 2 North/3North mine. Also, coarse coal rejects produced from washing of the No.3 seam ROM coal through the plant are potentially acid generating and require sub-aqueous disposal in the 3 South pit – a storage site of limited capacity. Moreover, ROM coal stockpiled on surface is prone to spontaneous combustion in the warm summer months due to the high pyrite content of the coal.

The 4 South mine is strongly believed to be constrained to non-caving mining methods, due to the shallow overburden cover over the mine workings (100 meter average), the presence of large surface swamps above the workings and the unpredictable and dangerous caving response of its massive sandstone roof. Initial partial pillar recovery trials off No.1 Mains with recovery of pillars ‘in line’ resulted in unexpected localized roof falls. As a result, a partial pillar recovery mining method was successfully adopted to mitigate operational hazards. This thesis will investigate the cavability and critical span of the 4 South sandstone roof partly to evaluate the risks of full-extraction mining at the 4 South mine.

Chapter three presents site characterization work performed in support of an optimized ground support strategy and mining method for the 4 South mine. The rock mechanical

properties database for the roof and floor sandstones have been augmented with elastic property data and consideration for water saturation effects. Detailed joint and fault mapping, rock mass classification and in-situ stress state determination are also included. An extensive literature search is also presented in support of the following:

1. Developing design rock mechanical properties for the primary geologic units of the 4 South mine, and also to estimate subsidiary rock mechanical properties required for detailed numerical model studies,
2. Determining a design value for the in-situ strength of the No.3 seam coal, and
3. Evaluating the assumption of a hydrostatic in-situ stress state.

The site characterization data provides value to the evaluation and optimization of the 4 South mine and all prospective mines with massive sandstone roof conditions at the Quinsam mine site, and also to global coal mining projects with similar mining conditions. This data is fundamental to the numerical modeling, structural hazard assessment, and ground support optimization studies.

The results of a ground support optimization study for the 4 South mine are presented in Chapter 4. First, a review of the 4 South mine ground support strategies employed and the condition and effectiveness of the installed ground support is presented. Next, an extensive literature search on rock bolting support theories including suspension, beam building, beam and plate theory are presented which allow for estimate of maximum roof deflection and critical span and form the basis of ground support design tools used in practice. Section three provides a comprehensive review and application of dead-weight, rock load height and empirical/analytical roof support design, to evaluate the relationship between bolt length, bolt anchorage depth, bolt capacity and pattern spacing to the roof wedges defined by the joint sets in the massive sandstone roof. Next, a review of the mechanical bolt and forged-head and tensioned rebar bolt is presented, including a discussion of the advantages and disadvantages of each in application. This is followed

by a presentation of rock bolt pull test campaigns conducted in support of an optimized final ground support system for the 4 South mine.

Chapter five intends to present and apply the fundamental theories and techniques for diligent pillar design. Section one summarizes the theories of pillar loading including Tributary Area Theory (TAT) and abutment loading as applied in the NIOSH software Analysis of Retreat Mining Pillar Stability (ARMPS). Section two focuses on four important aspects of determining in-situ pillar strength, including effects arising from pillar size, shape and discontinuities and time. Section three presents and applies practical pillar design methods to the 4 South mine. Finally, pillar design recommendations are presented for the 4 South mine pillars.

In chapter six, the cavability and critical span of the 4 South sandstone roof is investigated to evaluate the risks associated with caving massive sandstone roof, which include windblast, and unpredictable, episodic caving. Considerable research is referred to. In particular, literature on cavability is drawn from the Indian coalfield experience where massive sandstone roof represents over 60% roof conditions in operating coal mines. Tools to evaluate critical span are presented as referred from published empirical research and field studies abroad, as well as beam and plate theory. The chapter is concluded with a review of critical convergence velocities and their application to the 4 South mine roof. Finally, a brief discussion of roof stability monitoring is presented and common technical instruments employed in this regard. Finally, a review of a non-electric convergence extensometer fabricated for this study is presented and the results of field tests in the 2 North/3 North mine.

Chapter seven illustrates the environmental risks associated with water management and coarse coal reject management stemming from mining and processing 4 South ROM coal. The spiking of sulphate concentrations to levels in excess of British Columbia water quality guidelines for the receiving environment in response to mining activity at 4 south is highlighted. The implications on mining extraction methods are reviewed against the environmental risks discussed. Section two presents the known chemical composition of the coarse coal rejects produced from washing of the No.3 coal seam. Heavy metal

concentrations in the effluent draining from test piles of coarse coal reject material are shown to present environmental risks which will be weighed against mining extraction methods.

Chapter eight presents two separate compliance-to-design and numerical modeling analyses for the in-line pillar and checkerboard partial pillar mining methods employed at the 4 South mine. A brief review of the mining history and evolution of pillar extraction techniques is provided. The final section presents an optimized pillar extraction method for the 4 South mine that accounts for all anticipated mining conditions detailed in this thesis.

Chapter nine outlines the primary recommendations and conclusions of this thesis, the contributions to the state-of-the-art stemming from this research, and recommendations for future work.

CHAPTER 2 THE 4 SOUTH MINE

2.0 INTRODUCTION

The Quinsam Coal Mine is located in the Quinsam Watershed, approximately twenty-eight kilometers southwest of the City of Campbell River. The land occupied by the mining operation consists of 143 hectares. Quinsam Coal is the only operating underground coal operation in the province of British Columbia. Figure 2.1 shows the location of the mine site.

The Quinsam mine is primarily an underground thermal coal mine – open pit operations were mined in the late 1980’s and early 1990’s -with an annual production rate of 520,000 clean tonnes. It is operated by Quinsam Coal Corporation and is wholly owned by Hillsborough Resources Limited. Hillsborough has been a publicly traded company since 1988 and shares of the Corporation are listed for trading on the Toronto Stock Exchange under the symbol “HLB”.

Quinsam produces high volatile bituminous coal with a calorific value of 6,600 Kcal/kg (11,900 Btu/lb on an air dried basis) and 6,200 Kcal/kg (11,200 Btu/lb on an as received basis). The coal is processed at an onsite Preparation Plant and hauled 32 kilometers by B-train trucks to the Middle Point Barge Terminal, located north of Campbell River. Local coal orders - up to 30,000 tonnes - are shipped via barge. International shipments – up to 75,000 tonnes - are shipped from the Texada Island Loading Facility.

The mine serves the local and west-coast cement industry as well as local niche markets such as pulp and paper plants. Approximately one-third of annual production is available for the spot international market

The Quinsam Coal mine currently comprises a north and south mining area. The north mining area was initially developed in 1986 as an open pit operation (2 North Pit) and continued as an open pit operation until mid-1991. In late 1989, an underground test mine was developed at the south end of the North Pit. Production of coal from the

Figure 2.1: General Location Map for the Quinsam Coal Mine.



underground test mine began in early 1990. The North Pit operation went entirely underground in mid-1991. In June 1992, these entries were reclaimed and new underground entries (No.1 Mains) were developed in the centre of the east pit wall. The

No.1 Mains entries are the primary access to the 2 North and 3 North coal reserves. The 3 North coal reserves have been mined since September 1997.

The south mining area was initially developed in mid-1991 via the 1, 2 & 3 South Pits, located approximately 5 km south of the north pit area and southwest of Long Lake. A change from open pit to underground mining in late 1993 resulted in reclamation of the 1 South Pit and the partial backfilling of Pits 2 & 3 in preparation for development of the underground portals. Three parallel underground portals were developed in the 2 South Pit in late 1993 and the 3 South Pit was used as short-term overflow storage for the south mine area water management system. Production from the south pits ended in May 1994. Production from the 2 South underground occurred between February 1994 and October 1996.

The 4 South and 242 underground mine portals were developed in 1996. Coal production from the 242 underground mine began in September 1996 and ended in November of 1996. Continual coal production from the 4 South underground mine began in February 1996 and ended on April 30, 1999. Small amounts of coal were mined from the 4 South mine in 2003, 2004 and 2005. As of 2004, total in-situ tonnes for the 4 South mine were 8.7 million with 7.3 proven reserves.

2.1 SAFETY

Safety is job number one at Quinsam Coal; however it has not always been that way. From 1996 through 1998 there were 40 to 60 lost time accidents per year. Productivity was poor - measured in terms of tonnes per employee hour it was 2.11 tonnes per hour, far below any industry standards. Labour relations were also poor. During this time, there were as many as 2 dozen grievances per year compared to 1 or 2 per year now. On January 6, 1998, there was a terrible accident in the mine that took the lives of two miners.

In midst of these times, corporate and operational management took stock of the situation and resolved to recommence operations with many fundamental changes that included:

- Management style
- Focus on safety
- Mining methods
- Employee involvement
- Appropriate Capital Investment

Between 1999 and 2007, the mine has experienced only 7 lost time accidents. For the past 7 years in a row, Quinsam coal has won British Columbia's Small Underground Mining Award. In 2001 and 2003 the mine won the prestigious CIM sponsored John T. Ryan Safety Award presented to the safest coal mine in Canada.

Currently, targeted underground miner training programs are an important part of Quinsam Coal Corporations excellent safety record. Quinsam Coal currently employs approximately 100 hourly workers with an average age of 37 years and five years of underground work experience. Comparatively, in 2004, the average age of underground miners was 51 years with 26 years of underground work experience.

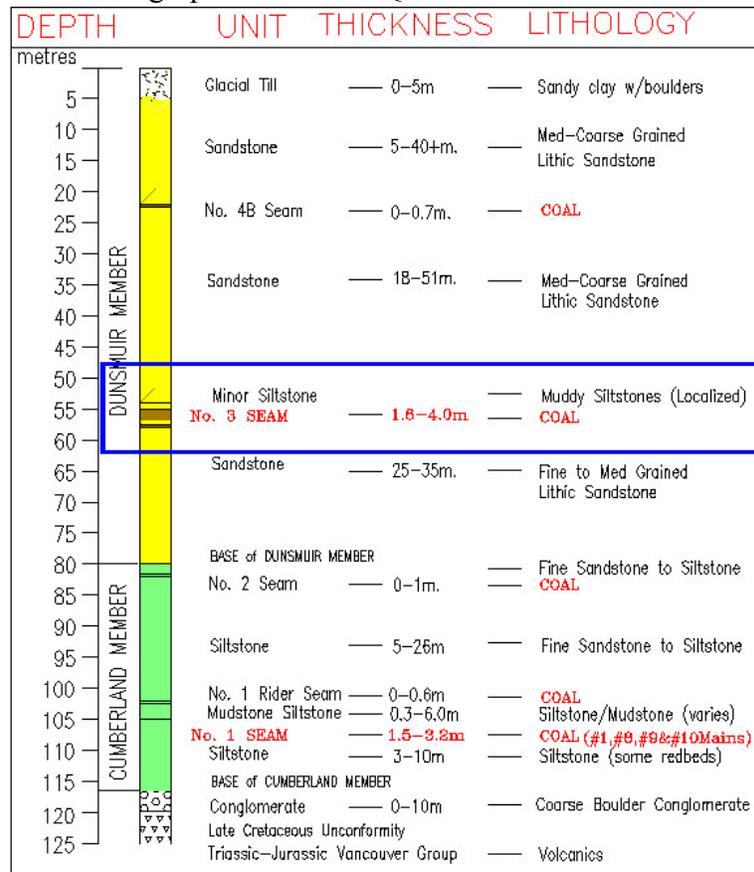
2.2 SITE GEOLOGY

The Quinsam Coal Mine is located in the Nanaimo Lowlands physiographic region on the east coast of Vancouver Island. The entire area was glaciated during the Pleistocene epoch. The coal measures of economic importance occur along the east coast of the Island in an area extending from Fanny Bay to Campbell River, a distance of 75 km. The area is underlain by the Late Cretaceous Comox Formation, consisting of sandstone, siltstone and coal units. The Comox Formation coal-measures dip uniformly at angles of 6 to 10 degrees, generally to the east or northeast.

Three major coal seams were deposited in the Quinsam area. Figure 2 shows a typical stratigraphic section for the Quinsam Coal Mine. The No.1 and No.2 Seam are hosted in the Cumberland Member. The No.1 seam, found throughout the property is the source of most of the coal extraction at the Quinsam Coal Mine. The No.3 Seam is hosted in the overlying Dunsmuir Member, which consists of thick-bedded sandstones containing

localized silty interbeds. The No.3 seam is mineable only southeast of the Quinsam River. Figure 2.2 shows a typical stratigraphic section for the Quinsam Coal Mine.

Figure 2.2: General Stratigraphic Section – Quinsam Mine.



The sediments were laid down in two distinct depositional cycles:

1. The lower cycle consists of medium greenish-gray to brown Cumberland siltstone, mudstone and the No.1 and No.2 coal seams. This lower cycle was formed in a quiescent coastal swamps and lagoons. Coal Seam No.1 is 1.5 to 4 meters thick. This coal seam has very low sulphur content and constitutes super compliant coal for power generation.
2. The upper cycle consists of white to gray, medium to coarse grained arkosic Dunsmuir sandstone, with minor siltstone, mudstone and No.3 and No.4, and No.5 coal seams. The upper cycle was formed in a higher energy delta front environment. Only the No.3 and No.4 coal seams are economically mineable. They are up to 4.5 m thick. The average separation between No.1 and No.3 coal seam is approximately 60 m.

2.2.1 Sedimentation and Structure

In the 4 South area, the Dunsmuir exhibits cross-bedding, channel splays and partial washouts, indicating a fluvial, deltaic depositional environment. The No.3 seam, where it approaches the old pre-Cretaceous basement hill separating the 4 South area from the 2 South area to the southwest is split by a number of thick sandstone and siltstone partings. As the distance from this feature increases to the north and east, the No.3 seam exhibits a marked change in character. The numerous rock partings thin out and disappear, leaving a relatively clean coal section 3 to 5 meters in thickness.

The structure of the 4 South mining area is complex. The area is bounded by faulting, with the occurrence of pinching out or thinning of the seam or outcropping along basement highs. To the northwest, the coal is up thrust on the Long Lake Fault Zone. The coal is faulted or pinches out to the southeast. To the southwest, the coal thins and is bounded by a faulted anticline.

Within the developed north side of the 4 South mine, the seam dips at an average of 7° to the east. In the undeveloped south side of the 4 South mine, the seam flattens and dips north. The mine is cut by several north/south trending faults with displacements up to 10 meters. Between these major faults, gentle folding may be present.

2.2.2 Coal Quality and Seam Characteristics

The boundaries and character of the No.3 seam are well defined by drilling. In the direction of the thinning seam, a coal thickness of at least 1.5 meters and a 30 meter minimum overburden cover is used to establish the 4-South coal boundary. This thickness must be characterized with a maximum average raw coal ash content of 29%.

The No.3 seam is within the higher sulfur reserve with an average of 4.0% from drilling and 3.8% from raw coal sampling. Drill holes indicate the propensity for sulfur to concentrate in both the base and top of the coal seam. The average ash of the coal seam is 27%.

2.3 CLIMATE

In general terms, the island is characterized by a humid, marine-type climate typical of the Pacific North-West Coast. There are very definite seasonal fluctuations in precipitation. Seasonal precipitation in the form of rain and snow on the higher elevations occurs mainly between the months of November and April. The dry season spans the balance of the year (May to October).

Temperatures on Vancouver Island range from -20°C to +32°C. Mean temperatures in summer are in the low to mid 20's and in winter are in the 5 to 10°C range in the lower elevations.

The climate of the East Coast of Vancouver Island is classified as temperate. Winters are mild and rainy; summers are dry. In each month between October and March, the weather station at the Campbell River airport records between 100 and 275 mm total precipitation. Between April and September average monthly precipitation totals are less than 80 mm. Precipitation can occur as snow anytime between November and April. The regional water equivalent of annual snowfall ranges between 100 and 150 mm, or 7% to 10% of the total annual precipitation.

The micro-climate of the Middle Quinsam Lake area (area around the immediate minesite) differs from the regional climate in that the area is in a localized depression with high ridges to the south and east and high mountains in the west. The fog and low cloud during the winter months is trapped within the depression of the Middle Quinsam valley (Parkes, 1995). In comparison to the Campbell River Airport precipitation records, average monthly precipitation is 50% less at the Quinsam Mine site. Table 2.1 summarizes average monthly rainfall recorded at the Campbell River Airport and at the Quinsam Coal mine.

An analysis conducted of the available climate data from the Environment Canada station at the Campbell River Airport and mine site indicated a 3% increase in average annual

precipitation¹ (rainfall and snowfall) and a 5% increase in average annual rainfall over the past 10 years (Goeller et al. 2007). Figure 2.3 below shows the 1 in 200 year precipitation event for the Quinsam mine area is 118mm based on historical precipitation records from 1993-2007.

Table 2.1: Average monthly precipitation (mm) recorded at the Campbell River Airport and at the Quinsam Coal Mine; after Goeller et al. 2007.

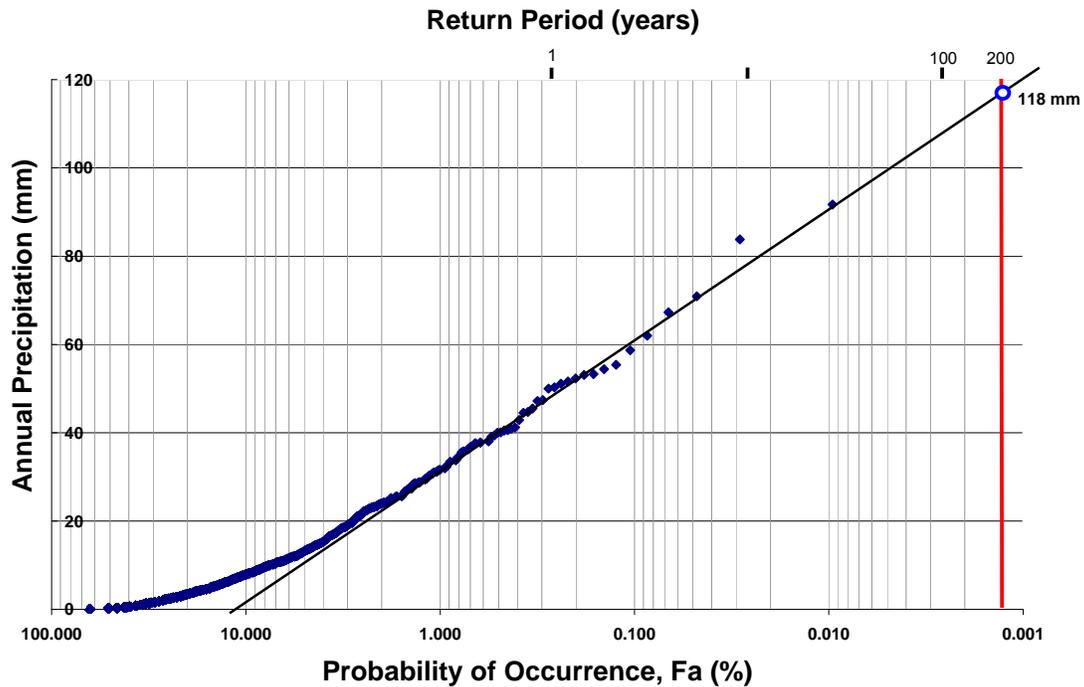
Month	Campbell River Airport	% of Annual Average	Quinsam Mine Site	% of Annual Average
January	198.5	14%	134.12	14%
February	158.7	11%	71.34	7%
March	136	9%	89.36	9%
April	84.2	6%	53.34	6%
May	67.1	5%	53.69	6%
June	61.2	4%	41.69	4%
July	40.4	3%	27.83	3%
August	48.6	3%	28.93	3%
September	58.9	4%	32.77	3%
October	152.9	11%	120.23	13%
November	230.7	16%	174.17	18%
December	214.5	15%	125.86	13%
Average	120.9	-	79.4	-

2.4 MINE OPERATIONAL OVERVIEW

At the Quinsam Coal Mine, the room and pillar mining method is employed. Room and pillar mining is very common in underground mines, by nature of its flexibility and minimum requirement for large capital expenditure. Room and pillar mining involves developing interconnected tunnels termed roadways or entries (on strike) and cross cuts (perpendicular) within the coal seam. The excavated tunnels form pillars of coal, sized to provide support for the overlying strata.

¹ Precipitation includes rainfall and snowfall

Figure 2.3: Daily Frequency-magnitude precipitation for Quinsam Coal Mine Site, 1993-2007.



Tunnels are developed 6.0 meters wide by 2.5 meters in height on average. Where 2.4 m rock bolts are used for ground support, the minimum required mining height is 2.4 meters. Adverse rib and roof conditions are often associated with increases in the mining height and roadway width due to sloughing and rock failures.

Underground mining development at the Quinsam Coal Mine can be classified into two categories: Mains and Panel. Quinsam Coal uses a 3-entry Mains layout with crosscuts and entries spaced 30 to 38 meters apart. All main water pipelines, communication lines, and high-voltage electrical feeder lines and primary ventilation flow are directed to production areas via the Mains. The Belt-road – or “B” road – is the central entry and flanked by the fresh air entry (“A” road) and exhaust air entry (“C” road).

Panels are typically driven perpendicular to the Mains in a five-entry layout. Supersections, defined as panels with greater than five developed roadways, are developed occasionally. Panel widths are generally 86 to 84 meters wide with considerable

variability on their length - 250 to 550 meters. Slab cuts and/or fan-outs of 6m depth are typically taken during pillar extraction where ground conditions permit.

Barrier pillars of 30 meter width separate room and pillar mining panels. Their primary function are to prevent migration of water and explosive gases to the adjacent panels and shield developing panels load transfer from caved panels nearby. Barrier pillars are essential in room and pillar mines in the prevention of chain pillar collapses. In some instances, extensive room-and-pillar workings can collapse with little warning and pose a serious risk to underground miners.

Tunnels are typically advanced in 6 meter wide and deep cuts by Joy 12CM12, and 12CM11 Continuous Mining machines. Extended cuts of 9-meter length are made where safe conditions permit. Joy 10SC32 Shuttle cars (electric drive) and Jeffery 4100 Ramcars (diesel) are employed to move cut coal to the Stamler feeder at the head of the conveyor. The Joy 10SC32 Shuttle car is an 18-tonne track driven electric drive machine with a 13.5 tonnes payload. The design of the Joy Shuttle car lends itself to high productivity in coal movement application. High-voltage trailing cable on this mining machine requires diligent and constant surveillance to avoid damage during operations. In contrast, the Jeffery ram car is a diesel driven, rubber-tired haulage vehicle with a payload of 8-10 tonnes. Coal is unloaded via a retractable steel plate located at the back of the haulage bed. The primary advantage of the ram car is the ability to cycle multiple cars to the Continuous mining machine – 3 to 4 ideally. The Isuzu diesel engine powering the Jeffrey ram cars at Quinsam Coal are equipped with a catalytic exhaust treatment to remove carbon monoxide and diesel particulate from the exhaust. Ground support installation is performed with dual-boom Fletcher CHDDR roof bolters. Figure 2.4 shows the typical mining district layout for the 4 South mine.

At the 4 South mine, two pillar extraction methods have been employed: in-line pillar recovery and partial checkerboard pillar recovery. Coal extraction involves pillar robbing and sweeping of up to a meter of coal from the floor. Final mining height ranges between three to four meters. Efforts are made to mine the 2.1 to 2.7 meter section of clean coal

located above the lower sandstone parting to avoid the soft, wet, and inferior quality coal lying below the parting. Details of these mining methods can be found in Chapter 8. Table 2.2 shows a summary of total extractable mining height by planned panel. Panel locations can be reference in Figure 2.5 which shows an updated mine plan for the 4 South mine.

Table 2.2: Total Extractable Mining Height by Mining Panel – 4 South

PANEL ID	Avg. Seam Thickness (incl. rock partings) (m)
101	3.70
102	3.65
103	3.80
104	4.00
105	3.70
106	4.00
107	4.00
108	3.70
109	3.70
110	3.00
111	3.00
112	3.00
113	4.00
114	4.00
115	3.50

2.4.1 4 South Mine Ventilation

Fresh air enters the mine on the “A” road entry and to a lesser extent on the “B” road conveyor entry. Exhaust air exits the mine via an upcast ventilation raise located on C-Rd, #7 x-cut of No.1 Mains.

Two Engart fans in parallel provide up to 100,000 cfm of fresh air to the mine. Normally, one fan runs at a pressure of 1 inch W.G., circulating approximately 60,000 cfm of air, while the other is on stand-by against breakdowns.

Concrete block and Kennedy stopping are used to separate the fresh intake air from the contaminated exhaust return air. The conveyor entry (B-rd) is isolated and regulated to limit the air flow to a maximum of 10,000 cfm. Minimal leakage has been detected in

historical ventilation surveys across these barriers. Figure 2.6 shows a plan of the panel district mining ventilation.

Figure 2.4: Typical Mining District Layout – Quinsam Coal 4 South Mine.

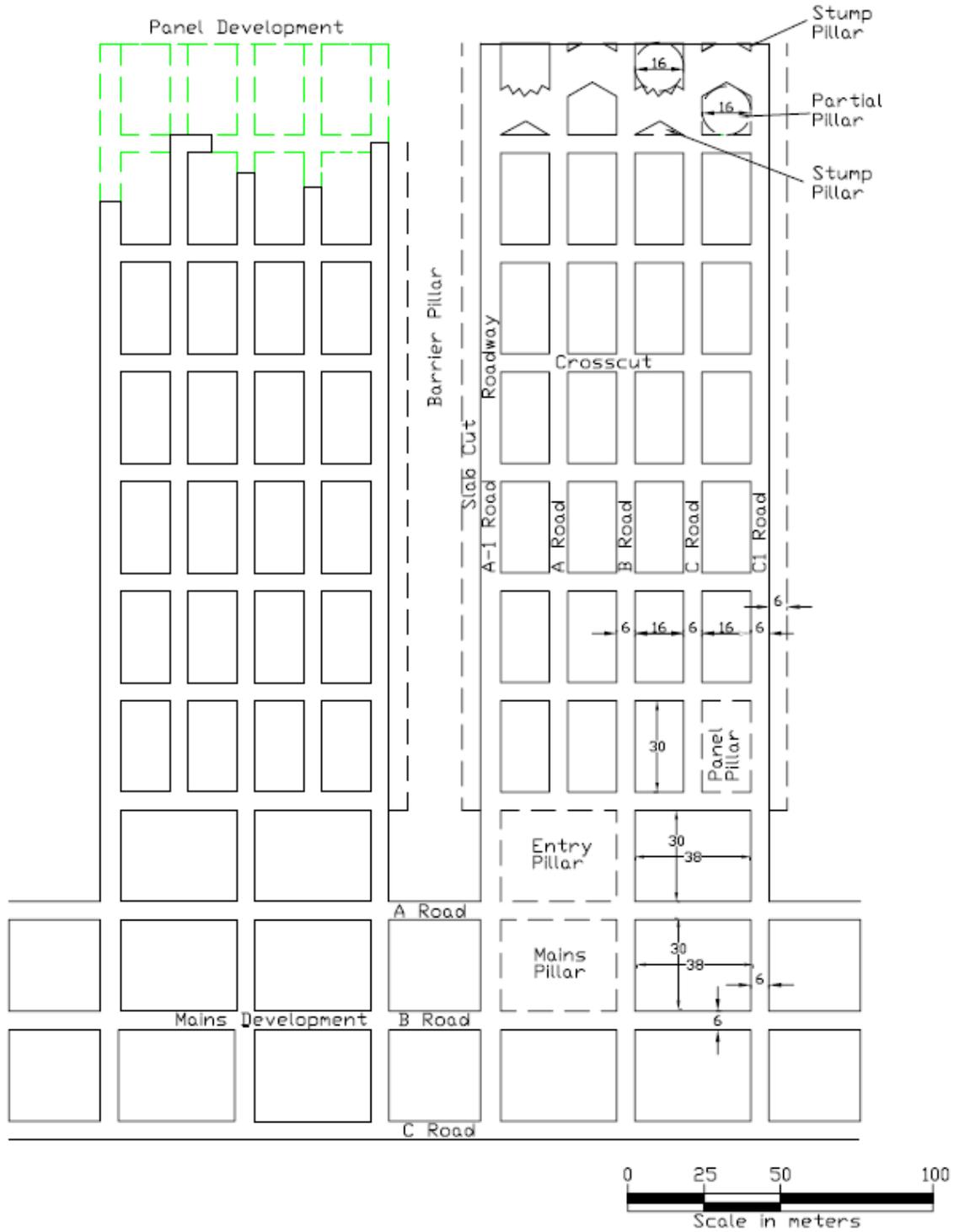


Figure 2.5: Plan View Map of the 4-South Mine at Quinsam Coal – March 2008.

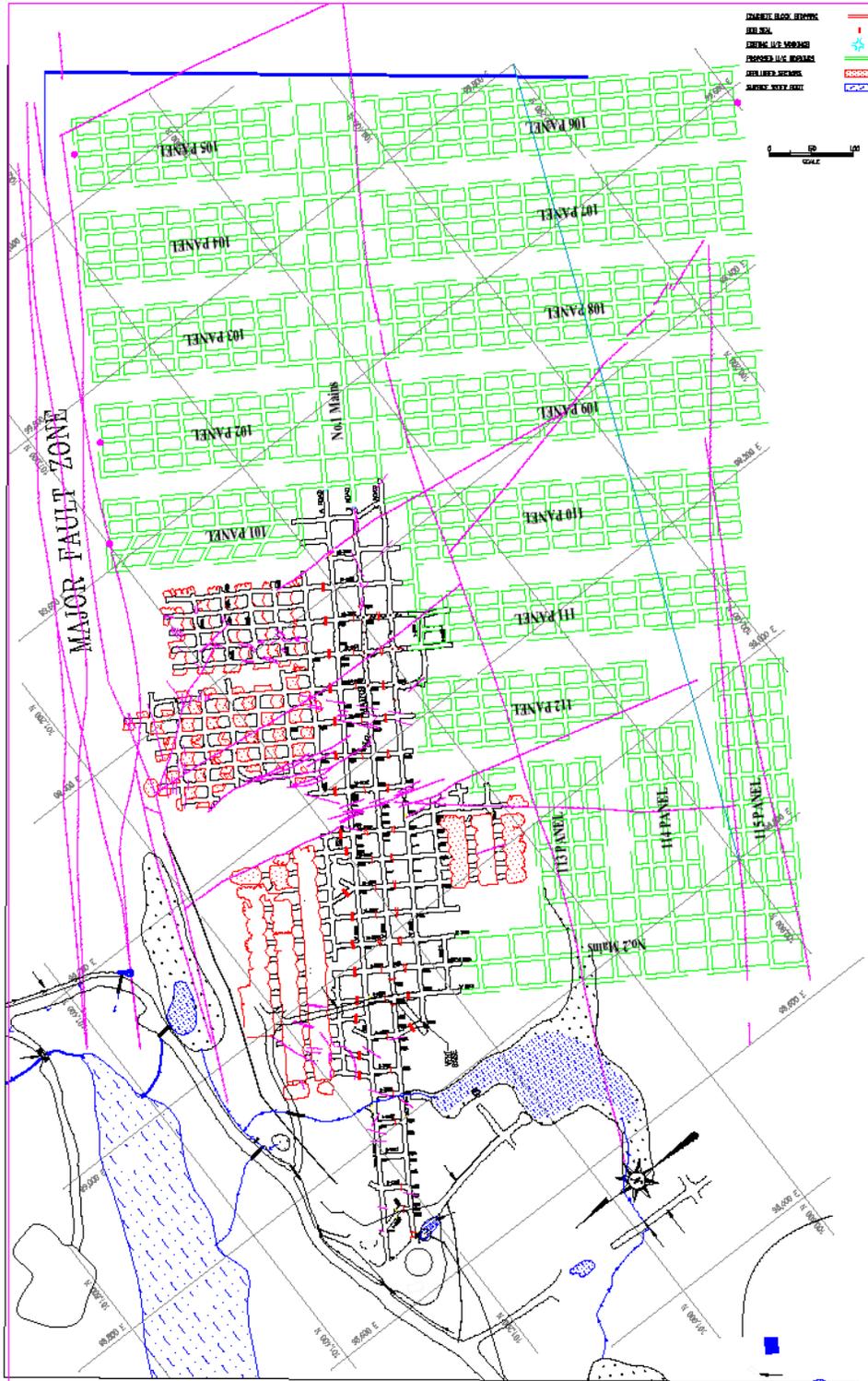
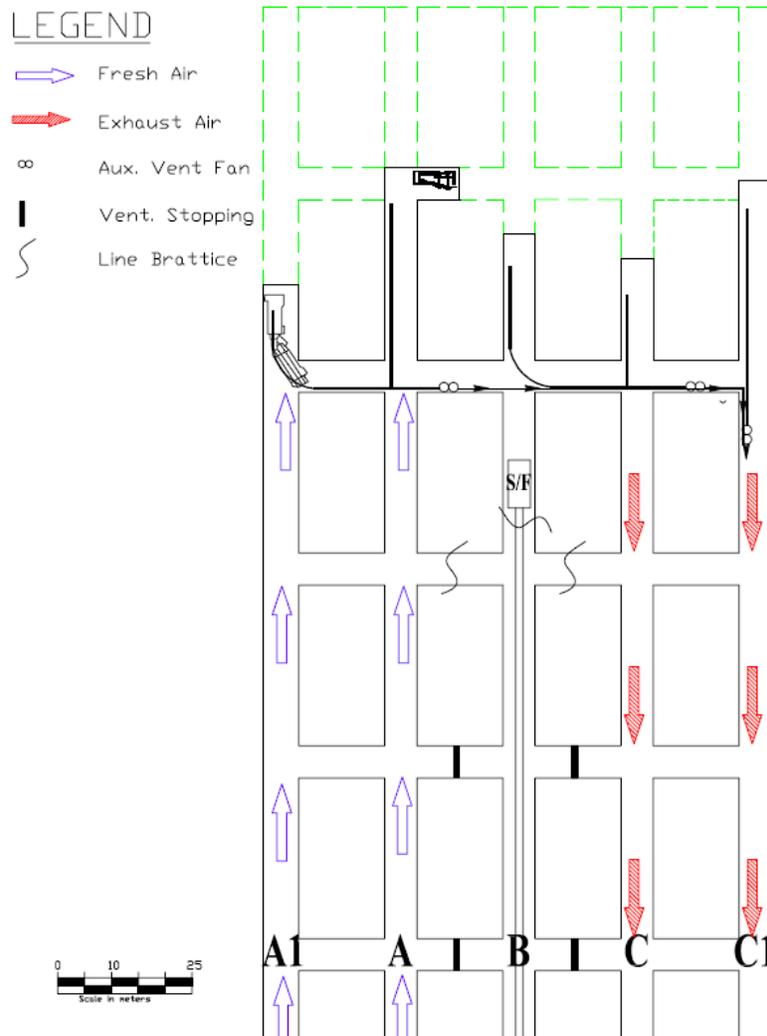


Figure 2.6: Mine District Ventilation and Face Setup.



As shown, the “A” and “A-1” entries supply fresh air for production activities, while the “C” and “C-1” entries route exhaust air out of the panel to the exhaust entry on the Mains. “B” road typically transmits a relatively small amount of fresh air.

Concrete stoppings and brattice constructed at half-distance in the crosscuts adjoining “B” road to “A” and “C” roads separate fresh and exhaust air flow. Auxiliary fans placed in the last open crosscut pull diesel particulates, equipment exhaust and release coal gases from the mining faces and direct it to the exhaust entry roads. The auxiliary fans are sized to prevent re-circulation of face ventilation.

CHAPTER 3 SITE CHARACTERIZATION

3.0 INTRODUCTION

Site characterization is fundamental in the application of empirical, analytical and numerical methods of analysis to design safe ground support systems, complete mining layouts and sequence extraction of coal. Forgeron (1999) notes the role of geological assessments are part of an on-going process, that constitutes a continuous cycle of input and feedback where deficiencies are recognized and adjustments are made to improve the process.

Site characterization work for the 4 South mine was conducted during the summer of 2003 and 2004 at the Quinsam Coal mine. The purpose of this work was to:

1. Develop a rock mass classification map for the current mine workings, using the Geomechanics Rock Mass Classification system (RMR),
2. Perform rock mechanical tests on 4 South roof sandstone core to for strength and elastic properties to augment the historic database and facilitate the determination of design values for analysis,
3. Perform a detailed structural mapping exercise and review mine mylars for mapped structure to determine major fault and joint sets to facilitate structural analysis,
4. Investigate the mine workings for signs of horizontal stress to determine if the assumption of hydrostatic stress conditions ($k=1$) is reasonable.

Section 1 reviews the Geomechanics Rock Mass Classification system (RMR) and the Coal Mine Roof Rating (CMRR). The CMRR is discussed as it is being implemented more commonly at Quinsam Coal mine in the 2N/3N mine; however, this study uses the RMR solely for design and assessment input. Design values for 4S sandstone roof and No.3 seam coal are provided and integrated into a rockmass classification map.

Section 2 will review the historical record of rock mechanical property testing data and laboratory test data compiled for this study. Design values for 4 South sandstone, coal

and mudstone/siltstone strength and elastic properties are presented. Indirect methods for determining the rock mass modulus and in-situ coal strength are also summarized.

In section 3, a graphical and statistical summary of structural mapping data is provided. Finally, results of fiberscope stratascope borehole tests are presented in support of an evaluation of the structural and geological composition of the immediate roof.

Section 4 presents a statistical summary of pre-mining vertical stress by panel for the 4 South mine and a comprehensive list of tools for evaluating the horizontal to vertical stress ratio, k , in the absence of in-situ measurement. These tools are shown to support the assumption of hydrostatic stress conditions for the 4 South mine.

3.1 ROCK MASS CLASSIFICATION

The rock mass is a complex composition of rock and discontinuities that requires specialized classification methods to evaluate the usefulness of the rock mass for different constructions. Rock mass classification schemes have been developing for over 100 years with most of the multi-parameter classification schemes being developed from civil engineering case histories. These systems combine the defined properties of the rock mass with practical experience from different kinds of constructions in rock. Parameters are often related to the discontinuities like number of joint sets, joint distance, roughness, alteration and filling of joints, are graded and compared with a graded scale that is calibrated for different kinds of rock construction.

RMR (Bieniawski 1974) and the *Q*-system (Barton et. al., 1974) are the most commonly used rock mass classification systems. Both were developed for designing tunnels and rock caverns, but their area of application has expanded to both mining and slopes. The Coal Mine Roof Rating (CMRR) was developed 14 years ago to fill the gap between geologic characterization and engineering design and address the limitations of the civil engineering classification systems in their application to coal mining. It combines many years of geologic studies in underground coal mines with worldwide experience with rock mass classification systems. Specifically, it addresses the layered geology and

geologic structures typical of coal mine roof. The coal mine roof rating system is increasingly used for many aspects of coal mine design (Molinda and Mark 1994).

The two classification systems employed to evaluate the rock mass of the 4 South mine roof are the Geomechanics Rock Mass Rating (RMR) and the Coal Mine Roof Rating (CMRR).

3.1.1 Geomechanics Rock Mass Rating (RMR)

Bieniawski (1974) introduced the Geomechanical Classification System that provides a general rock mass rating (*RMR*) increasing with rock quality from 0 to 100. The rating system is based on experience from shallow tunnels in sedimentary rock. In 1976 a modification of class boundaries took place and in 1979 there was an adoption of the ISRM rock mass description. The 1976 version is employed for rock mass classification at the 4 South mine.

According to Bieniawski (1989) the *RMR*-system has been applied in more than 260 cases, such as tunnels, chambers, mines, slopes, foundations and rock caverns.

Bieniawski (1989) also states that the reason for using this classification system is the ease of use and the versatility in engineering practice. The *RMR*-system has been calibrated using experience from coalmines, civil engineering excavations and tunnels at shallow depth.

RMR is based upon five basic parameters:

1. Uniaxial compressive strength of intact rock material
2. Rock quality designation (*RQD*)
3. Ground water conditions
4. Joint or discontinuity spacing
5. Joint characteristics

A sixth parameter, orientation of joints, can be used for specific applications in tunneling, mining and for foundations. The RMR classification comprises five parameters to be collected from inspection of either drill cores or of pillars.

A data entry sheet was developed for the Quinsam Coal Mine to use the Geomechanics RMR (1976) classification system. Graphs showing the developed relationships between a) the rock quality designation (RQD) and mean joint spacing and b) rock hammer index tests for UCS estimation are included on the sheet for easy reference.

3.1.2 Coal Mine Roof Rating (CMRR)

The Coal Mine Roof Rating (CMRR) is a rockmass classification system tailored to coal measure rock. It weighs the following individual geotechnical factors:

- Uniaxial compressive strength of the intact rock;
- Spacing and persistence of discontinuities like bedding planes and slickensides;
- Cohesion and roughness of the discontinuities; and
- Presence of ground water and the moisture sensitivity of the rock.

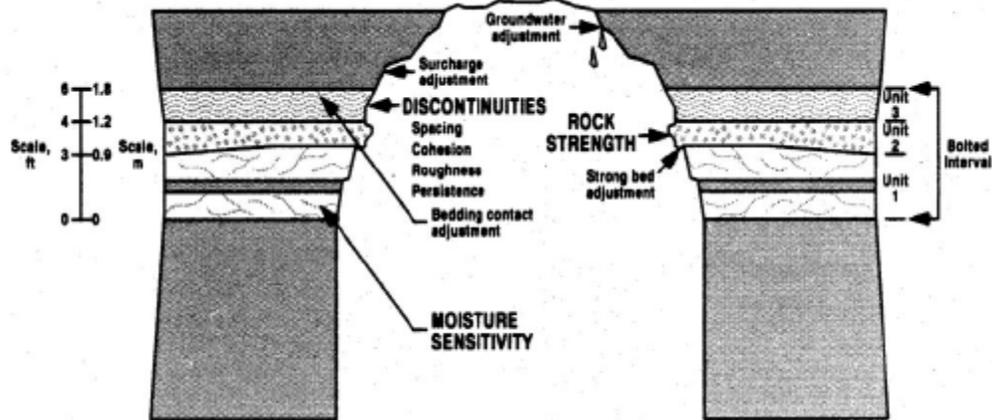
Simple index tests and observations are used to rate each of these parameters, which are combined in a single rating on a scale of 0 to 100. Figure 3.1 shows a graphical summary of the components of the CMRR system.

The CMRR can be calculated from underground exposures like roof falls and overcasts or from exploratory core. In the case of drill core, point load tests are used to estimate the compressive strength and the cohesion. A computer program is available to assist in the determination of the CMRR.

Mark (2000) notes that although the CMRR incorporates most of the geologic factors that affect the mine roof, it does not address large-scale features, like faults, sandstones channel margins, or igneous dikes.

The CMRR has had limited use at Quinsam Coal mine, due in part to lack of instructional training on its use. Any significant application of the CMRR at Quinsam Coal will require training and field-testing for consistency in evaluations across are users, the culmination of which should form a standard assessment manual for the mine.

Figure 3.1: Components of the CMRR System; after Mark et al. (2002).



3.1.3 Rockmass Classification Map – 4 South Sandstone Roof

A rock mass classification map is generated by contouring spatially distributed rock mass classification values over the mining plan. A database of nine RMR (1976) and one CMRR (1994) records were used to develop a rock mass classification record map for the 4 South mine sandstone roof. The author augmented the historical record with five assessments. It is assumed that all records are representative and accurate of the conditions described using the rock mass classifications systems employed. Figure 3.2 shows the classification record developed for the 4 South mine.

The distribution of RMR ratings in the database is shown below in Figure 3.3. Seventy percent of all records were between 65 and 75%. The assessed strength of the massive sandstone roof ranged between 40 to 100 MPa. Cullen (1996) reports a value of 79 MPa as a typical value for 4 South sandstone. The significant range in assess strength values indicates the need for an assessment standard in this area. A RMR design value of 70% is used for this study. Figure 3.4 illustrates condition and nature of the 4 South mine roof.

3.1.4 Rockmass Classification - No.3 Coal Seam

Five underground RMR records were recorded during July 2004 for No.3 seam coal. Records were taken where clean pillar exposures were found and accurate assessments could be made.

Figure 3.2: Rock Mass Classification Record Map for 4 South mine Sandstone Roof.

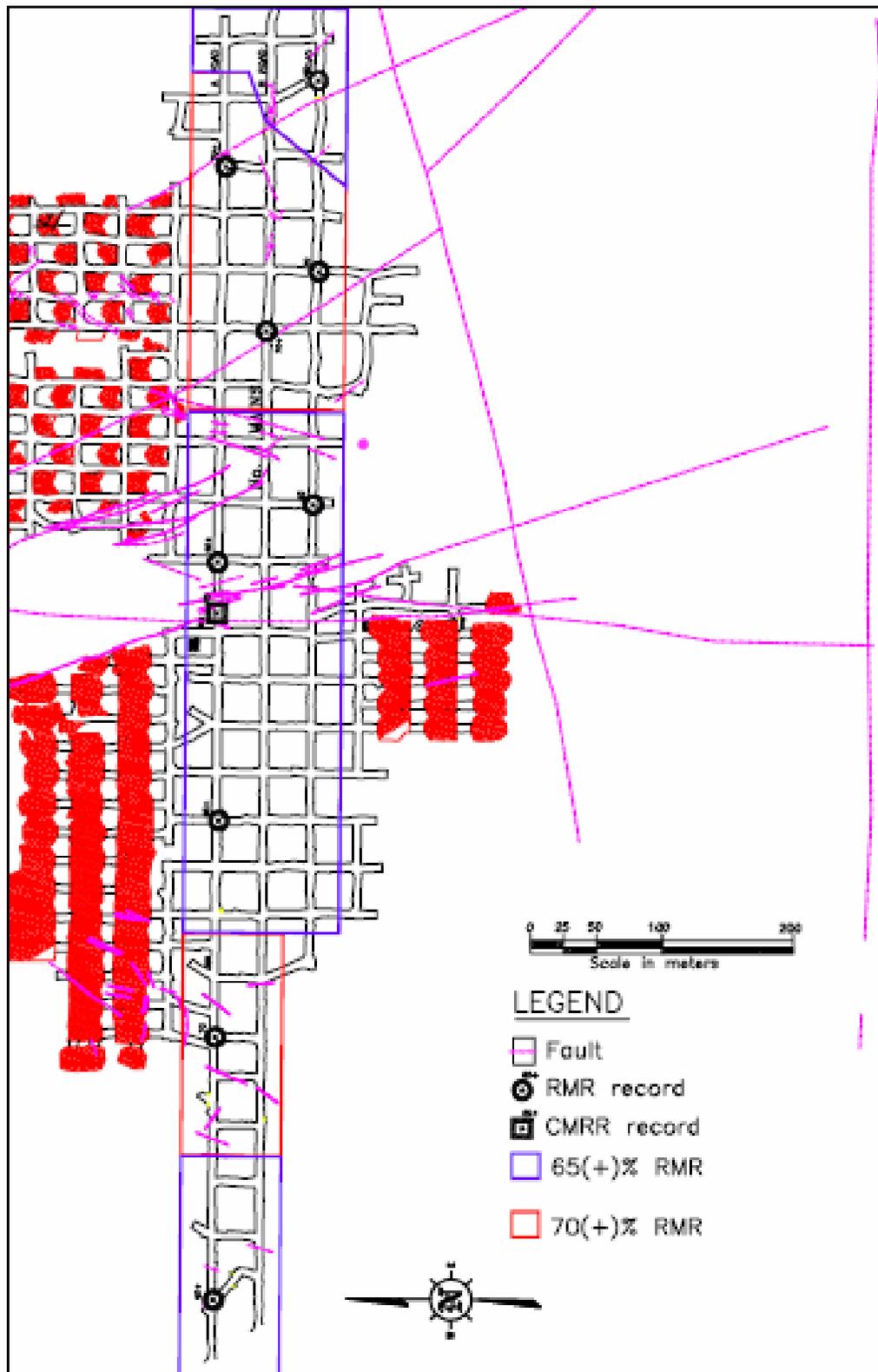


Figure 3.3: Distribution of RMR Records – 4 South Mine.

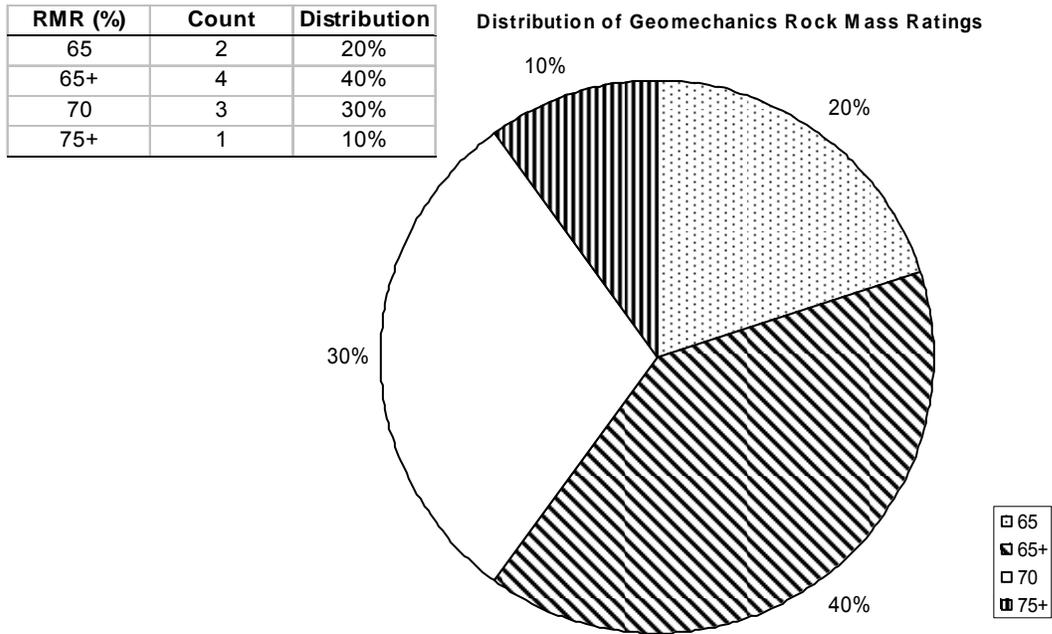


Figure 3.4: Massive Sandstone Roof at No.1 Mains, #1 Cross Cut, 4 South Mine.



The average RMR for No.3 seam coal was 36%. Cullen (1996) recorded a typical RMR for the No.1 coal seam of 37%. A design value of 35% was selected.

3.2 ROCK MECHANICAL PROPERTIES

The determination of rock mechanical properties is generally achieved using test methods and procedures advanced by the American Society of Testing and Materials (ASTM) and the International Society of Rock Mechanics (ISRM). The Physical and mechanical properties of rocks include the primary set of rock mechanical properties. Mechanical properties can be subdivided into elastic and strength properties and further differentiated between static and dynamic properties.

Rock mechanical properties serve as generic input for both preliminary, non-specific design analysis and more rigorous numerical analysis. The estimated stress-strain response of a rockmass is dependent on rock mechanical properties as input. Careful consideration of the application of rock mechanical properties in design is necessary as well as an understanding of the effects of geologic discontinuities, water and stress on in-situ performance of rock pillars, floor and roof.

Methods and correlations exist to estimate rock mechanical properties where none are available or difficult to obtain. The relative importance of a rock mechanical property in its use for design provides an engineer with guidance on when estimation will not overly impair the results of an analysis.

The 4 South mine is developed within No.3 seam coal, overlain by massive sandstone and underlain by a 0.5-meter thick coaly mudstone/siltstone base which is further underlain by massive sandstone. These rock units were considered relevant to mine design and assigned strength and elastic rock mechanical properties, either through direct testing or relevant referencing as detailed in Table 3.1. ASTM and/or ISRM test procedures and methods were used for all testing. Design values were selected based on the weighted average of the database.

Hasenfus et al. (1992) provide the following list of strength and elastic property values for sandstone and coal from their paper on longwall development design, as per Table 3.2 below.

Table 3.1: Rock Mechanical Properties for 4 South Mine, Standard Deviation in Square Brackets, Number of Tests in Parentheses.

Material	Source	Elastic properties		Strength properties		Physical properties	
		Young's Modulus E (GPa)	Poisson's ratio ν	Uniaxial Compressive Strength σ_c (MPa)	Tensile strength σ_T (MPa)	Density (g/cc)	Slake Durability Index
4S Sandstone	Cullen (1996)	20.7 [9.8] (2)		47.6 [3.5] (4)			
	Cullen (1999)	13.1 [6.5] (5)	0.16 [0.02] (5)	68.6 [24.5] (5)		2.6 [nd] (2)	97 (1)
	Forgeron (1999)	13.1 [nd](9)	0.16 [nd] (5)	54.9 [nd] (16)			
	DuBois (2003)	8.8 [2.4] (15)	0.16 [0.10] (15)	40.5 [15.3] (15)	5.44 [2.89] (24)		
	DESIGN	11.2	0.16	50.5	5.44	2.6	97
No.1 coal seam	Cullen (1996)	2.8	-	20.1	1.0	-	97
	Forgeron (1999)	-	-	11.0 [nd] (13)	0.9 [nd] (18)	1.3[nd](4)	
	DESIGN	2.8		15.0	1.0	1.3	97
Siltstone Mudstone	Cullen (1996)	8.9		35.9		-	
	Cullen (1999)	10.66 [3.2] (4)	0.20 [0.09] (4)	57.13 [16.0] (4)	-	-	
	DESIGN	10	0.2	50.0	5.0	2.6	

[#] = Standard Deviation (#) = Number of Tests nd = no data

Table 3.2: Typical Rock Mechanical Properties for Sandstone and Coal; after Hasenfus et al (1992).

Rock Type	Young's Modulus (MPa)	Poisson's Ratio	UCS (MPa)	Cohesion (MPa)	Internal Angle of Friction	Density (t/m ³)
Sandstone	14479	0.22	60.9	17.9	33	2.6
Coal	2897	0.35	23.8	6.2	35	1.3

Good agreement is found between known and referenced strength and elastic properties in the Hasenfus (1992) dataset. Design values for i) internal angle of friction, ii) cohesion for both sandstone and coal and iii) Poisson's ratio were referenced from this dataset.

Mark (1999) determined the anisotropic ratio (axial to diametral point-load strength) to be 1.9 from point-load tests on 4S sandstone roof samples from drill core. The anisotropy of roof units can indicate the shape of a roof fall. Whereas strongly bedded and highly anisotropic rocks tend to "stair step" and create flat-topped falls, massive isotropic rocks like sandstone tend to form blocky, inverted V-shaped falls.

No rock mechanical testing has been done on the No.3 seam coal. Cullen (1996) presented strength and elastic properties for the No.1 seam coal based on his own testing program and point-load test data from Piteau Associates (1992) and CANMET (1995). Forgeron (1999) also tested No.1 seam coal, performing both UCS and tensile strength tests. Weighted averages of No.1 seam coal elastic and strength properties were selected

for No.3 seam coal. Rock mechanical property testing of No.3 seam coal is required and recommended for future work.

There are no direct tests of basal mudstone/ siltstone comprising the immediate 4 South mine floor. Cullen (1996, 1999) perform laboratory tests on the mudstone and siltstone comprising the 2 North / 3 North. It has been assumed that the character of the mudstone / siltstones described are similar. Design values for the basal 4 South mudstone / siltstone are referenced from the 2 North / 3 North roof mudstone / siltstone. The tensile strength is assumed to be one-tenth of the UCS value. The mudstone/siltstone density is assumed to be 2.6 t/m³ for this study.

3.2.1 Effect of Water on UCS and Elastic Response of 4S Sandstone

Wet conditions for the 4 South mine were evident during the 1995, 1997 and 2003 drilling programs at the mine site, with weeping groundwater indicated on many of the exploration drill holes, up to 10 USGPM (US gallons per minute). Mining operators and supervisors working in the 4 South mine recalled wet conditions during mining and roof bolting on a regular basis (Morely et al., 2008). The majority of faults and well-developed joints in the sandstone roof of the existing mine workings area were observed to be water bearing.

A roof joint-water flow survey was completed on October 2005 in the 4 South mine to determine the quantity of water flowing into the mine from roof joints and faults. Water flow was measured by recording the time to fill a bucket of known volume.

Approximately 8.2 USGPM of water was measured during the survey from six joints and two faults in the roof sandstone. The average flow for faults and joints was 0.6 and 2.3 USGPM respectively with an overall average of 1.0 USGPM per roof discontinuity feature. The sandstone roof was noticeably deteriorated in the vicinity of water bearing faults, and unaffected for joints. In consideration of the demonstrated ability of joints and faults in the sandstone roof to transmit water, it is reasonable to assume that roof drilling will continue to be burdened with wet conditions.

The effect of water on the strength and elastic properties of sandstone was investigated. It is understood that water can significantly impair the strength and stiffness of rock units. It has been reported that UCS and UTS are influenced by water content (Burshtein, 1969; Mimuro et al, 1991; Dube and Singh, 1972; Price, 1960). Price (1960) reported that the UCS of sandstone decreases as water content increases (saturated, natural, dry). Using five types of sandstone, Dube and Singh (1972) found a non-linear correlation between UCS and water content.

Kramadibrata et al. (2000) concluded that, with respect to sandstone, the influence of water content on the strength properties of sandstone is significant on the UCS and ITS, but less influence occurs on the Young's Modulus. He also found that the UTS and ITS test results revealed that as the water content increased from dried to saturated condition, the values of UCS, E, and ITS decreased as much as, 45%, 26% and 43% respectively.

The author investigated the effect of water on the UCS, Young's Modulus and Poisson's ratio on three large sandstone core samples from the 2003 laboratory test program. Core samples were divided to create two samples of approximately equal length. One of the samples was oven dried and then saturated in water for 24 hours, while the other was unaltered and unprocessed. Unconfined compressive strength tests complemented with axial and circumferential strain gauges were conducted on both samples using similar pre-load and loading rate conditions. Table 3.3 shows the results of the testing.

Table 3.3: Effect of Water Saturation on the Strength and Elastic Properties of 4S Sandstone. Percent Reduction Indicated by Positive Values.

Sample	UCS (MPa)	Young's modulus (GPa)	Poisson's Ratio
D	50%	16%	54%
I	88%	95%	-20%
J	74%	113%	-
Average	71%	75%	17%

The results indicate considerably higher reductions of the UCS and Young's Modulus are possible due to water saturation than reported by Kramadibrata (2000). These results corroborate concerns expressed in the presence of wet roof conditions in coal mines,

particularly where conditions occur at intersections. Water promotes pore pressures on discontinuities and diminishes the frictional properties of joint surfaces, both which are detrimental to roof stability. These results should not be confused with the weatherability of sandstones which are known to be very resistant to the weathering effects of moisture due mainly to their inert mineralogy.

In light of these investigations, wet roof conditions, particularly at intersections, should be routinely assessed for additional surface support and/or upgraded pattern bolting.

3.2.2 Indirect Estimation of the Rock Mass Deformation Modulus

The rock mass deformation modulus is an important parameter in the representation of the mechanical behavior of a rock and of a rock mass.. This parameter is required for most numerical finite element and boundary element analyses for studies of the stress and displacement distribution around underground excavations.

The rock mass deformation modulus is often estimated indirectly from observations of relevant rock mass parameters that can be acquired easily and at low cost. In-situ tests like Plate jacking tests (PJT), Plate loading tests and Radial jacking tests are effective, but are operationally difficult and expensive to perform. Plate jacking tests were shown to provide the most accurate in-situ measurement results (Palmstrom, 2001).

Empirical relationships have been developed for estimating the in-situ deformation modulus on the basis of classification schemes such as the Geomechanics Rock Mass Rating (RMR), The Tunneling Quality Index (Q), and the Geological Strength Index (GSI). Figure 3.5 shows a summary of empirical equations developed for predicting the rock mass deformation modulus. Note Table 3.4 lists the equations referenced in Figure 3.5.

Figure 3.5 reinforces the importance of performing in-situ tests when determining the rock mass deformation modulus for a given material. Most empirical methods relating

Figure 3.5: Empirical Equations for Predicting Rock Mass Deformation Modulus Compared with Data from In Situ Measurements; after Hoek et al. (2006).

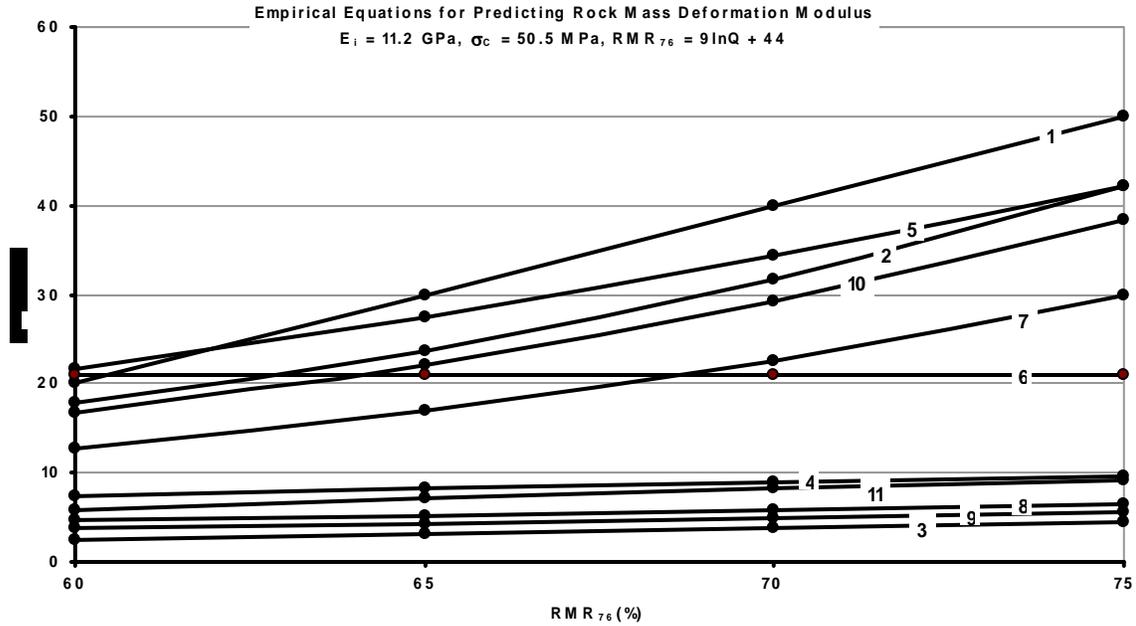


Table 3.4: Data and Fitted Equations for Estimation of Rock Mass Deformation Modulus Plotted in Figure 3.5; after Hoek et al. (2006).

1. $E_{rm} = 2 \times RMR - 100$	Bieniawski (1978)
2. $E_{rm} = 10^{((RMR-10)/40)}$	Serafim and Pereira (1983)
3. $E_{rm} = E_i / 100(0.0028 \times RMR^2 + 0.9 \exp(RMR / 22.82))$	Nicholson et al. (1990)
4. $E_{rm} = E_i \left(0.5 \left(1 - \cos \left(\frac{180 \times RMR}{100} \right) \right) \right)$	Mitri et al. (1994)
5. $E_{rm} = 0.1(RMR/10)^3$	Read et al. (1999)
6. $E_{rm} = 10Q_C^{1/3}$ where $Q_C = Q\sigma_{ci}/100$	Barton (2002)
7. $E_{rm} = (1 - D/2)\sqrt{\sigma_{ci}/100} \times 10^{((RMR-10)/40)}$, $D = 0$	Hoek et al. (2002)
8. $E_{rm} = E_i (s^a)^{0.4}$, $s = \exp((GSI - 100)/9)$, $GSI = RMR$ $a = 1/2 + 1/6(\exp(-GSI/15) - \exp(-20/3))$	Sonmez et al. (2004)
9. $E_{rm} = E_i s^{1/4}$, $s = \exp((GSI - 100)/9)$	Carvalho (2004)
10. $E_{rm} = 7((\pm 3)\sqrt{Q'})$, $Q' = 10((RMR - 44)/21)$	Diederichs and Kaiser (1999)
11. $E_{rm} = E_i \left(0.02 + \frac{1 - D/2}{1 + e^{((60 + 15D - GSI)/11)}} \right)$, $D = 0$	Diederichs and Kaiser (2006)

rock mass classification systems to the intact rock modulus, E_i overestimate the static rock modulus. Only five of the eleven equations predict values of the rock mass deformation modulus less than the laboratory static Young's modulus value for 4 South sandstone with values ranging from 3.7 to 8.9 GPa. The average value of the five equations is 6.2 GPa.

Palmstrom (2001) reviewed the indirect estimates based on the RMR, Q (equations 1,2 and 6 from Table 3.2.2.1) and the Central soil and rock mass index (RMI) and estimated from laboratory tests adjusted for scale effects and compared the results of several in-situ deformation tests correlated with the rock conditions at each site to the estimates provided with these methods. Palmstrom advised against the use of the RMR based estimate methods for slightly jointed to massive rock masses and recommended the following equations based on laboratory testing adjusted for scale effects:

$$E_m = 0.2\sigma_c \quad (3.1)$$

and where rock type is known,

$$E_m = \frac{0.5\sigma_c \times MR}{1000} \quad (3.2)$$

where $MR = \frac{E}{\sigma_c}$, MR = modulus ratio

Equations 3.1 and 3.2 predict a rock mass modulus of 10.1 and 5.5 GPa respectively for the 4 South sandstone. A design value of 6 GPa was selected for the rock mass deformation modulus of the 4 South mine roof sandstone.

3.2.3 Indirect Estimate of In-situ Coal Strength

Bieniawski et al. (1994) stated that the RMR had never been used to estimate the rock mass deformation modulus of coal. The ratio of rock mass modulus to strength ratio is typically 500:1 . Bieniawski et al. determined that for coal strata in-situ, the uniaxial compressive strength of a coal seam may be estimated as:

$$\sigma_M = 20 \times RMR \quad (3.3)$$

Bieniawski found an 18.2% prediction error - defined as the difference between the observed value and the predicted value expressed as a percentage of the predicted value - with the above equation.

Other research at Penn State has shown (Bieniawski, 1994) that if the strength of coal material σ_c is known with confidence, another expression for the in-situ coal strata strength may be used:

$$\sigma_M = 0.5 \left[\frac{RMR - 15}{85} \right] \sigma_c \quad (3.4)$$

Applying equations 3.3 and 3.4, the predicted in-situ strength of No.3 seam coal is 4.82 and 2.36 MPa respectively.

The classic approach to estimating in-situ coal strength involves applying a “size effect” to the UCS of small coal specimens tested in the laboratory. This approach was thoroughly discredited by a comprehensive study which found that there was no correlation between the UCS and actual pillar strength (Mark and Barton, 1996). This study also found that design formulas were far more successful in predicting the performance of coal pillars when a uniform strength of 900 psi (6.2 MPa) was employed. Studies conducted in South Africa and Australia came to a similar conclusion (Galvin et al., 1999). This approach is endorsed for this study.

3.3 GEOLOGIC DISCONTINUITY ASSESSMENT

A detailed structural mapping program was performed in July 2004 to determine the primary joint and fault orientations for the 4 South mine. Structural data was also referenced from mine mylars of the underground workings. All orientations referred to in this study use the Quinsam Coal Mine co-ordinate system which is rotated 52° counter-clockwise from the true north co-ordinates.

A geotechnical mapping study minimizes geotechnical risk prior to a pillar extraction operation by identifying potential areas of weakness hidden or disregarded during development and supporting the evaluation of installing additional roof support. Su et al. (1999) note that the potential for rapid variations in geology makes assessing site specific support needs difficult because such changes are not always detectable or obvious from surface drill data, mine opening observations, and standard roof and rib bolting operations.

Exploration drill hole data adds value to the structural mapping assessments. Commonly, drill hole data is integrated with mine plans to verify or predict major geologic structure promoting dislocation and displacement of the coal seam. Ledvina (1996) notes that when the range of lithologies and structures within a mine area are known, drill hole data can be interpreted or re-interpreted to predict or explain problem areas. Drill hole data can also be used to confirm the projection of structures and lithologies based on mapping, often in advance of mining. Also, roof falls give excellent views of vertical roof sequences and contribute greatly when mapping for the diagnosis of roof control problems.

3.3.1 Structural Mapping at 4 South

The geologic structural characteristics of the 4 South mine are relatively complex as compared with mid-continent coal mines. The area is mostly bounded by faulting, with the coal locally pinching out or thinning along underlying basement highs. To the northwest, the coal is dislocated on the Long Lake Fault Zone. The coal is faulted or pinches out to the southeast. To the southwest, the coal thins and is bounded by a faulted anticline.

Within the developed north side of the 4 South mine, the seam dips at an average of 7° to the east. In the undeveloped south side of the 4 South mine, the seam flattens and dips north. The mine is cut by several north/south trending faults with displacements up to 10 meters. Between these major faults, gentle folding may be present.

The face cleat of the No.3 seam strikes east/west and is oriented vertically. The butt cleat is sub-horizontal, parallel to the bedding planes. The coal is very competent and visually exhibits no weak orientation with respect to the roadways and crosscuts of the underground development. Morely et al. (2007) noted that coal pillars at 4 South are competent and do not slab or slough excessively.

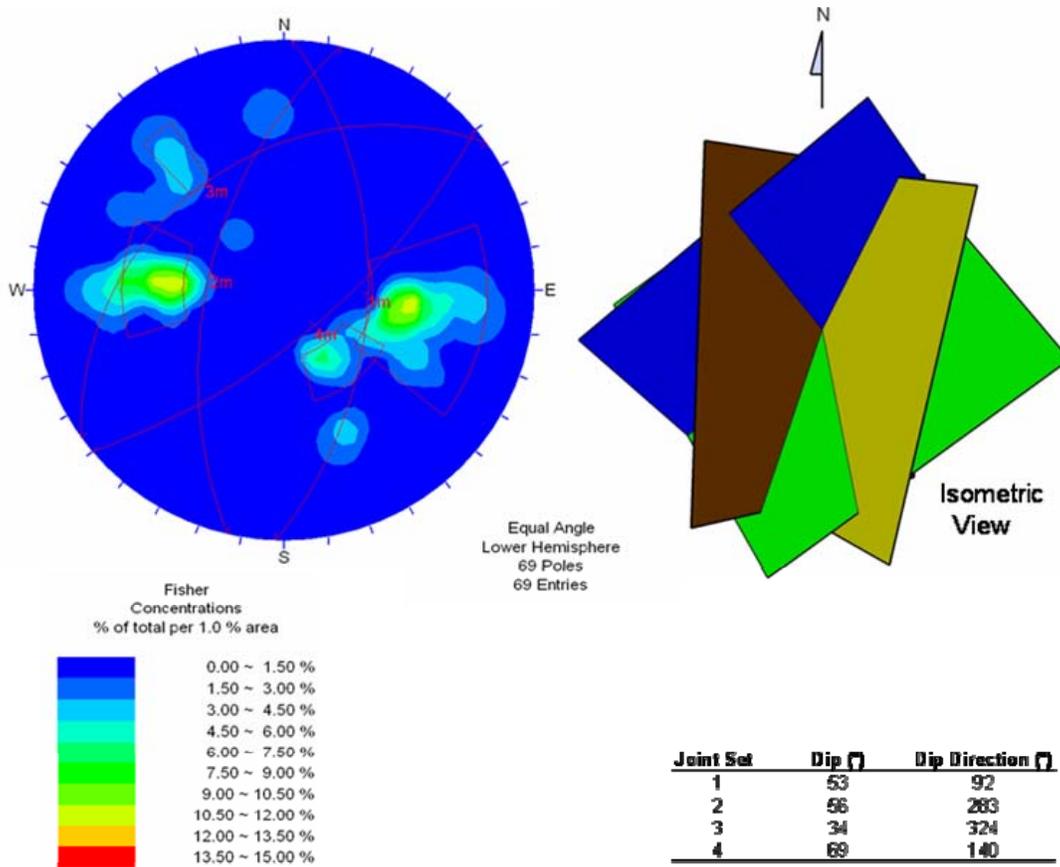
The sandstone roof is very competent and holds development spans effortlessly. It is very abrasive and lends to difficult to cutting, sparking and significant wearing of the carbide picks of the Continuous mining machine. Similarly, the abrasiveness of the sandstone results in reduced life of spade drill bits employed to drill roof bolt holes for ground support installation. Morely et al. (2007) report typically installing only two bolts per spade bit.

Joints in the sandstone roof are very difficult to discern as they are infrequent and tight. Water inflow from joints is commonly used as an identification precursor. Joint presence of density and number required to define major roof wedges is uncommon. One wedge identified underground recorded on A-rd, 22 cross-cut, was estimated to be 1-meter long by 2-meters wide by 0.5 meters, weighing approximately 1 tonne.

At Quinsam Coal Corporation, structural mapping is integral to risk management of adverse mining conditions including structural wedges, joint dominated water inflow, and gas accumulation. Experience-based projection of dominant structural features enables technical staff to evaluate the alternate placement of roadways and cross-cuts - where such action does not unduly disrupt operational ease - and modification to pillar extraction such to increase safety. Structural mapping also provides key input to two-dimensional and three-dimensional stability analyses of structural wedges. Rocscience provides a commonly employed software package Unwedge (2005) to assist in the evaluation of support systems for three-dimensional wedges in underground excavations.

Figure 3.6 shows an equal angle pole/plane plot for the 4 South mine with the dominant joint/ fault sets indicated graphically and numerically.

Figure 3.6: Pole/plane Plot from Joint Mapping – 4 South Mine.



Statistically, 44% off all fault offsets fall within the range of 0 to 0.99 meters. The next significant range is from 1.0 to 1.9 meters with 38% off fault offsets falling within this range. Fault offsets range from 0.1 to 5.0 meters and average 1.2 meters. Dominant joints and faults strike North/South and North-east/ South-west with dips ranging from 21 to 54°. Table 3.5 summarizes the fault offset data.

Roof conditions appear unaffected by seam dip in the existing working, but do appear to be controlled by shear/faults. Roof conditions typically deteriorate in the proximity of major faults and shear zones. Figure 3.7 shows a section through a normal fault located on A-Rd, between 14 and 15 cross-cut, the extent of which was overestimated by means of seismic survey. A five-meter fault offset is indicated.

Table 3.5: Statistical Summary of Fault Offset Data – 4 South Mine.

Offset Range (m)	Distribution (%)	Count
0 to 0.99	44%	20
1 to 1.99	38%	17
2 to 2.99	7%	3
3 to 3.99	7%	3
4 to 5	4%	2
<hr/>		
Average	1.2	
Stdev.	1.1	
Maximum	5.0	
Minimum	0.1	

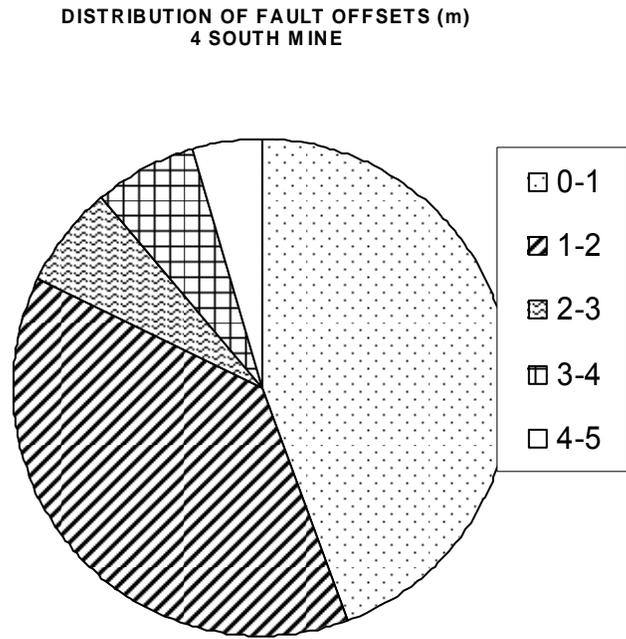
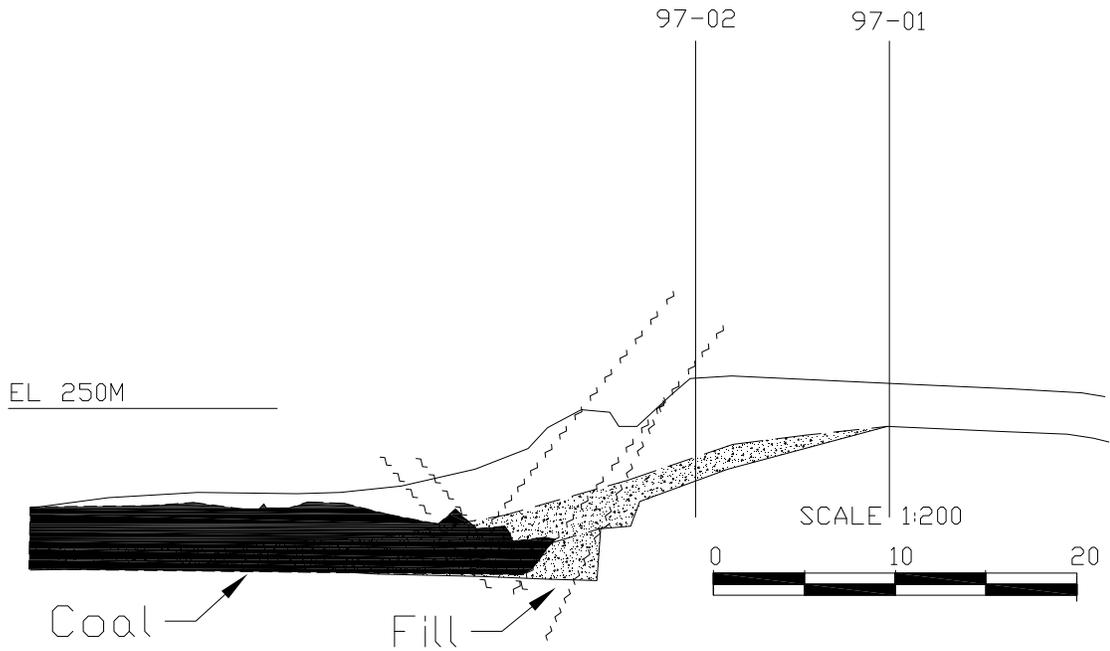


Figure 3.7: Fault Section View North at A-Rd between #14 and 15 Cross Cut, 4 South mine.



3.3.2 Stratascope Fiberscope Borehole Investigations

Two stratascope fiberscope borehole investigations were performed to assess in-situ conditions of the immediate roof. Both tests were performed at the northern start of No.2 Mains development. Forgeron (1999) also conducted a borehole stratascope investigation in the 4 South mine.

A Reichert flexible stratascope fiberscope was inserted into holes drilled into the immediate roof of 25mm diameter and 3.7 meter depth. The stratascope fiberscope is 19mm diameter and can be used in either the forward viewing or right angle mode. Illumination of the borehole is provided by coupling the fiber-optic scope to a standard miners' lamp. A picture of this instrument and its specifications are shown below in Table 3.6.

Table 3.6: Reichert Flexible Stratascope Fiberscope Specifications.

Fiberscope Specifications:	
Outside Diameter	0.75 inches
Flexible length	130 inches
Focus Eyepiece	Adjustable Eyepiece
Focus Objective	Fixed Focus
Depth of Field	0.6 to 1.375 inches
Viewing Direction	Forward Right Angle (with adapter)
Illumination	Internal Lightguide Coupled to Standard Miners Lamp
Protective sheathing	Brass/Bronze Braid Covered with Plastic
Watertight	yes
Minimum Bend Radius	5 inches



Figure 3.8 shows the location of the stratascope fiberscope investigations. Table 3.7 provides a summary of the results.

Forgeron (1999) found the immediate roof to be massive with no apparent joints and fractures filled with carbonaceous cement. The author identified one joint during the investigation at approximately 1.5 meters height into the immediate roof. Bedding planes were identified with an average vertical spacing of 1 to 2 meters. Dry conditions persisted

in all investigations. These investigations indicate the presence of massive, relatively sandstone roof for the developed area of the 4 South mine.

Figure 3.8: Location of Stratascope Fiberscope Borehole Investigations.

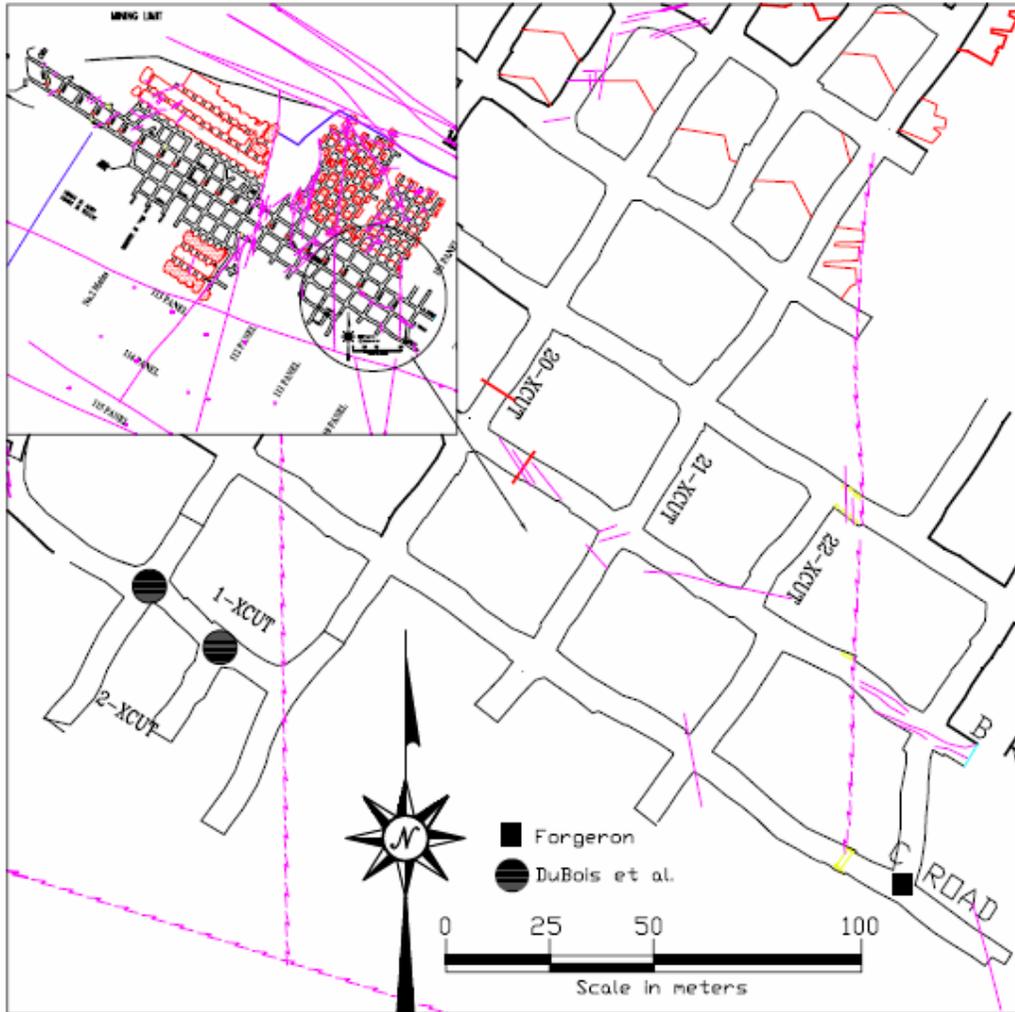


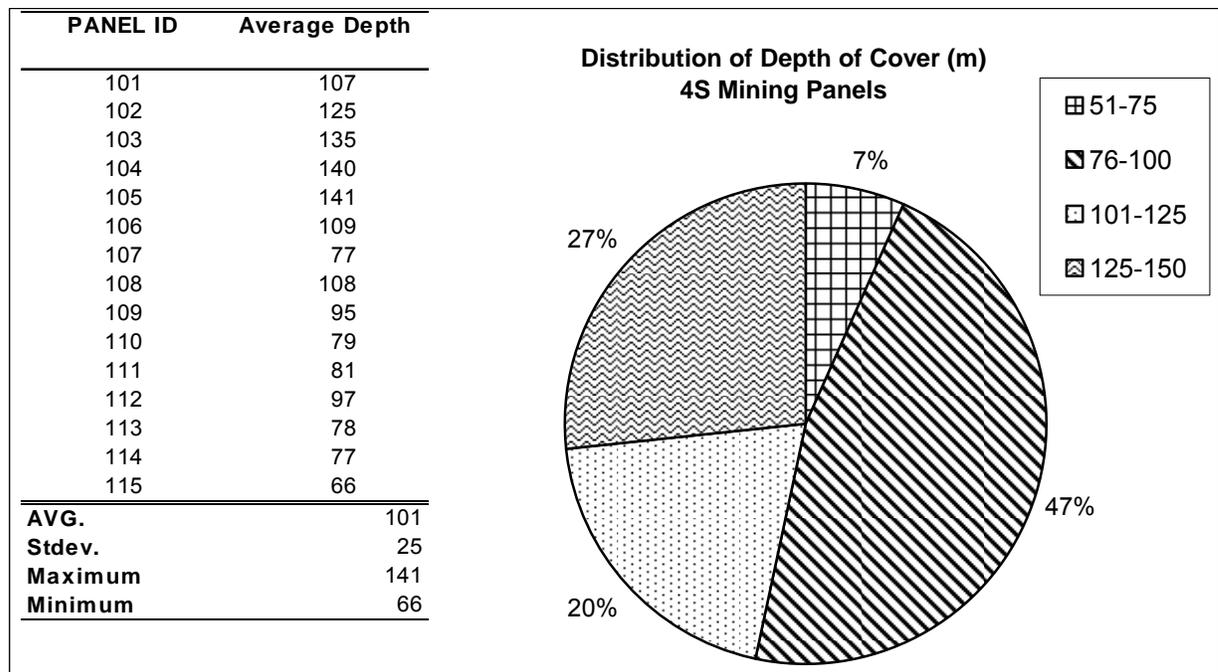
Table 3.7: Summary of Stratascope Fiberscope Investigations.

Hole:	04-01
Location:	No.2 Mains A-rd #1 cxut
Date:	Jul-04
Depth into roof (m)	Feature
	3.56 Bedding plane horizontal
	0.3-0.51 Thin bedding planes
	1.45 sub-horizontal joint
Hole:	04-02
Location:	No.2 Mains B-rd #1 cxut
Date:	Jul-04
Depth into roof (m)	Feature
	2.59-2.64 Bedding plane 40° to CA
	1.65 Bedding plane 45° to CA
	0.91-1.32 Weak Bedding Planes 10-20° to CA

3.4 IN-SITU STRESS STATE

The vertical normal stress underground is normally proportional to the depth of overburden and the density of the overlying strata.. The 4 South mine is a shallow depth underground mine with overburden cover ranging from 66 to 141 meters and averaging 101 meters. Approximately 67% of all current and planned extraction panels occur within an overburden depth of cover range of 76 to 125 meters. Figure 3.9 summarizes the overburden depth by extraction panel and range.

Figure 3.9: Depth of Cover Distribution by Panel – 4 South Mine.



The initial state of stress originates from overburden load and tectonic history of the rock mass. The gravitational component of the horizontal normal stress is a function of Poisson's ratio of the material and the vertical stress. If the material can be considered linear-elastic and isotropic and a one-dimensional state of strain is assumed, these relationships can be theoretically expressed by the following formulae:

The vertical stress, σ_v :

$$\sigma_v = \rho g z \tag{3.5}$$

where ρ = density of the rock mass (kg/m^3)
 g = gravity acceleration (m/s^2)
 z = the depth below ground surface (m)

The horizontal stress, σ_h :

$$\sigma_h = \sigma_v \left(\frac{\nu}{1-\nu} \right) \quad (3.6)$$

where:

σ_v = vertical stress (MPa)
 ν = Poisson's ratio of the rock.

Poisson's ratio ranges from 0.15 to 0.35 for most rock types.

It is understood that in-situ horizontal stresses are often higher than those predicted by the above equation because they are modified by topography, tectonic stresses, folding uplift, faulting, stiffness variations between strata, etc.

If the rock cannot be considered elastic, the magnitude of the horizontal stress component will after some time be equal to the vertical component. This is called a hydrostatic or lithostatic stress field. This means that the horizontal component due to gravitational factors can be expressed as

$$\sigma_h = k\sigma_v \quad (3.7)$$

where k is a factor that varies from 0 to 1 depending on restraints on displacement.

There have been no identifiable signs of horizontal stress at the 4 South mine. Both Forgeron (1999) and Cullen (2002) found no evidence of elevated horizontal stress during their site visits to the Quinsam Coal 4 South mine. No in-situ measurements of stress have ever been conducted at the Quinsam Coal mine. Future work should investigate the cost and benefit of doing in-situ stress measurements at Quinsam Coal mine.

In coal mining the use of stress mapping for the determination of horizontal stress directions has been described by Mucho and Mark (1994). Table 3.8 summarizes the features of horizontal stress manifestation underground.

Table 3.8: Summary of Underground Stress Mapping Techniques; after Mark and Mucho, 1994).

Feature	Observation Noted	Relationship to G_H
1 CUTTER - GUTTERING OR KINK ROOF	Location in entry, especially tendency passing through intersections	Entry location gives indication of angle of mining to stress field. Through intersection tries to align with G_H
2 TENSILE FRACTURES	Direction	Gives direction of G_H
3 ROOF POTTING	Direction of major and minor axes	Major axis gives direction of G_H
4 ROOFBOLT HOLE OFFSETS	Direction of roof movement	Roof layers move in direction of G_H
5 SHEAR PLANES AND ROCK FLOUR	Direction	Planes and rock flour lines are in the direction of G_H
6 STRIATIONS ON ROOF ROCK	Direction	Striations are parallel to G_H
7 ROOF FALLS	Location, shape and appearance	Location gives clues as to the general directionality of the stress field. High angular shape usually indicates high horizontal stress with stepped shear failures usually predominating on one side.

Bush et al. (1988) provided the following equation for the stress gradient of sandstones as a polynomial equation with x representing the overburden depth in feet:

$$242 + 0.88x - 4.265x^2 10^{-5} - 3.3x^3 10^{-9} \quad (3.8)$$

Sheorey (1994) developed an elasto-static thermal stress model of the earth. The model considers curvature of the crust and variation of elastic constants, density and thermal expansion coefficients through the crust and mantle. A detailed discussion on Sheorey's model is beyond the scope of this chapter, but he did provide a simplified equation which can be used for estimating the horizontal to vertical stress ratio k . This equation is:

$$k = 0.25 + 7E_h (0.001 + 1/z) \quad (3.9)$$

where, z = depth below surface (m)

E_h = horizontal modulus of deformation

Figure 3.10 shows a results summary for predicted in-situ horizontal stress for the 4 South mine with depth. The hydrostatic stress condition ($k=1$) is approximately an average of the Bush et al. (1988) and Sheorey et al. (1994) equations over the range of overburden depths encountered at the 4 South mine.

It appears reasonable to assume hydrostatic stress conditions for the 4 South Mine. Figure 3.11 shows the expected in-situ stress state for all planned mining panels for the 4 South mine.

Figure 3.10: Prediction of In-situ Horizontal Stress by Depth of Overburden - 4 South Mine.

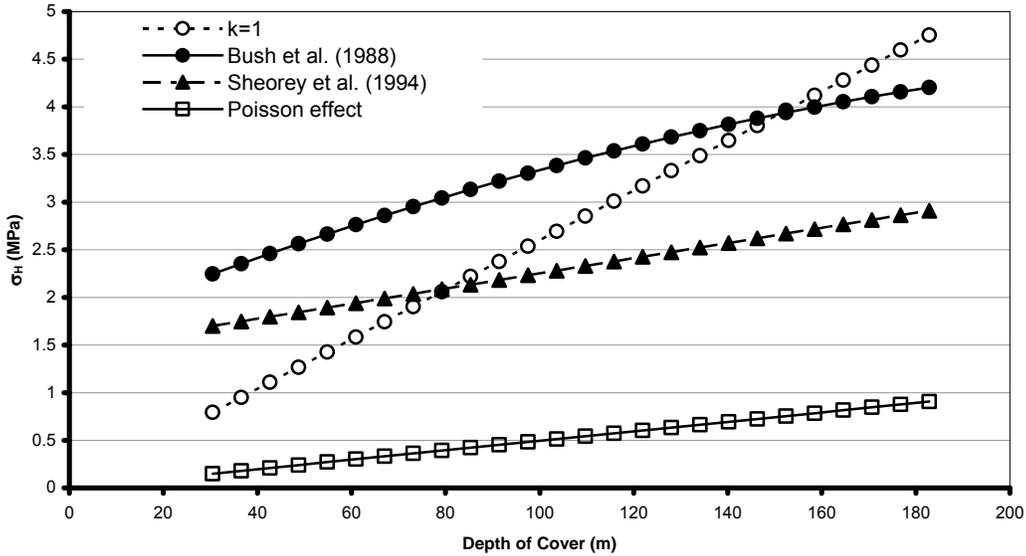
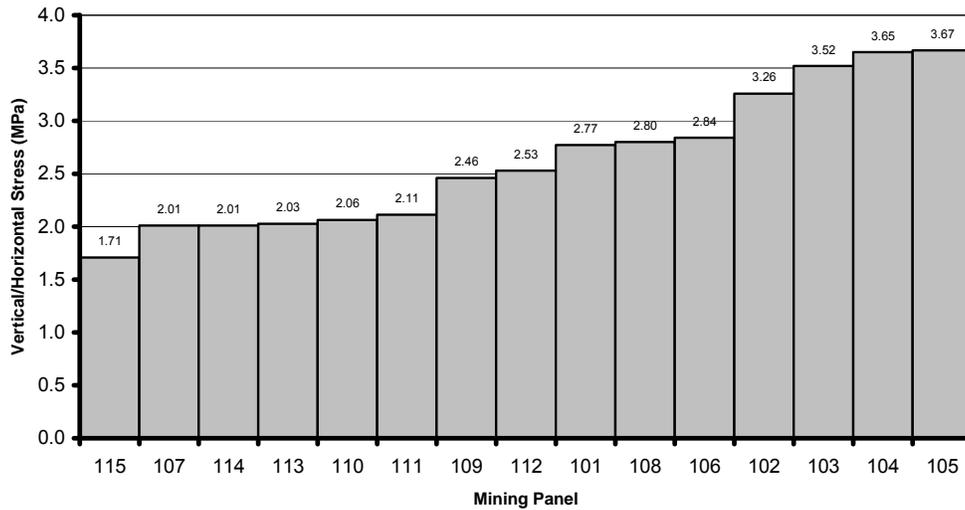


Figure 3.11: Initial Stress State by Panel – 4 South Mine.



3.5 CONCLUSIONS AND RECOMMENDATIONS

Site characterization for the 4 South mine involved reviewing historical records of information, performing an extensive literature search and augmenting data where

necessary and practical. The objective of this work was to provide design values for mine design work conducted for this and future studies as well as complement the global data set of site characterizations with massive sandstone roof.

A rock mass classification map was developed for the 4 South mine based on underground observations. The average RMR for the 4 South sandstone roof and No.3 seam coal was 70 and 35% respectively.

Design values for primary elastic, strength and physical properties of the 4 South mine geologic units were determined and are summarized below in Table 3.9.

Table 3.9: Summary of Design Rock Mechanical Properties – 4 South Mine Geologic Units

Material	Elastic properties		Strength properties		Physical properties	
	Young's Modulus E (GPa)	Poisson's ratio ν	Uniaxial Compressive Strength σ_c (MPa)	Tensile strength σ_T (MPa)	Density (g/cc)	Slake Durability Index
Sandstone	11.2	0.16	50.5	5.44	2.6	97
No.3 coal seam	2.8	0.35	15.0	1.0	1.3	97
Siltstone/Mudstone	10	0.2	50.0	5.0	2.6	-

Wet conditions, originating from the sandstone roof, can be expected for new development in the 4 South mine. The majority of mapped faults and major joints in the sandstone were water bearing. Additionally, during exploration, groundwater was intersected at rates averaging 10 USGPM. Underground water inflow mapping from joints in the sandstone roof indicated an average flow of 1.0 USGPM per water bearing roof joint feature.

During rock mechanical testing, water was found to reduce the average unconfined compressive strength, Young's modulus, and Poisson's ratio of saturated specimens relative to dry specimens by 71, 75 and 17% respectively. These values are considerably higher than values reported in the literature and indicate the need for roof surveillance of wet conditions to mitigate roof instability and encourage the installation of secondary ground support to ensure mine safety.

Using indirect evaluation methods, a design value for the rock mass modulus of the 4 South sandstone of 6 GPa was selected. A value of 6.2 MPa for the in-situ strength of No.3 seam coal was also chosen.

The 4 South mine is intersected by many major faults with displacements of up to 10 meters. The sandstone comprising the immediate roof and floor is very competent. Stratoscope fiberscope investigations conducted for this study indicate massive unjointed sandstone roof up to 3.7 meters depth.

The dominant joint and fault sets strike North/South and North-east/ South-west, dipping between 21 and 54°. Fault offsets averaged 1.2 meters. These structural sets are assumed to be ubiquitous to the mineable No.3 seam coal reserve.

Overburden depths over future mining panels in the 4 South mine ranged from 66 to 141 meters and average 101 meters. No in-situ stress measurements have been conducted at the 4 South mine. There are neither classic indications of nor reports of horizontal stress in the existing workings. Hydrostatic stress conditions are assumed.

CHAPTER 4 GROUND SUPPORT OPTIMIZATION

4.0 INTRODUCTION

The unique mining environment of the 4 South mine has been quantified and qualified in the previous chapters. The sandstone roof is massive and relatively unjointed, characterized by good strength and elastic properties and equating to a good quality rockmass with strong resistance to weathering, but susceptibility to strength and stiffness reductions in the presence of water. Roof stability is strongly governed by pore water pressures and discontinuity attitude, density and persistence. The shallow overburden cover for the 4 South mine provides low horizontal stresses and correspondingly low clamping forces on gravity wedges and slabs in the roof. To properly maintain roof stability in massive sandstone roof, strong regard for roof structure and interaction of bolt systems with structural wedge scenarios is required. Furthermore, a good understanding of the limitations and advantages provided by different bolting systems is essential, ideally supported with in-field testing of installation effectiveness and suitability to the mining environment.

In this chapter, we will begin by reviewing the current ground support standard at the Quinsam coal 4 South mine. This will be followed by a discussion of ground support theories and how they relate to the support of coal mine roof. The theories discussed include suspension, beam building, beam theory, plate theory and voussoir beam theory.

In section three, discussion of common rock support design methods will be presented including dead-weight design, rock load height design, and empirical and analytical roof support design. Within this section, considerable discussion and analysis is provided regarding CAD assisted evaluation of intersection wedge stability relative to roof support designs. Simple CAD based analysis is effective and practical to coal mines with limited access to expensive analysis tools.

Empirical ground support design guidelines will be presented and shown to have practical application in the assessment of primary ground support for coal mine roof.

Section four provides a review of three suitable rock bolts for massive sandstone roof: mechanical bolts, tensioned rebar, and forged-head rebar. This is complemented with a summary of results from pull tests conducted on full-column forged head rebar, point-anchor resin rebar and mechanical bolts to establish design values for bolt yield load, strain at yield and bond strength.

4.1 4 SOUTH GROUND SUPPORT

Primary roof support consists of 20mm diameter, 1.8-meter long, tensioned resin point anchored bolts spaced on 1.2-meter centers with 6.3mm, 15x15 cm square domed plates. No wire mesh is prescribed. One 1.2-meter LIF37 Fosroc resin tube is used per hole. Theoretical resin encapsulation is 1.05m (61% of available bolt length). Ground support is installed with dual-boom Fletcher CHDDR roof bolters. 1-1/16" spade bits are used for drilling. All drilling is dry. Figure 4.1 illustrates both the competency and roof bolt pattern installed in the 4 South mine roof.

Figure 4.1: 4 South Sandstone Roof Support with Primary Ground Support Pattern, #15 Crosscut, A-Road, No.1 Mains.



Secondary ground support consists of timbers, 6-inch wide thin metal straps and No.9 gauge weldmesh screen. Timbering is employed in a passive roof support and rib confinement application in fault zones. Metal straps in conjunction with roof bolts to

cross support major structure, secure potential wedges and add shear resistance across breaks in the roof. Weldmesh screen is employed in faulted/sheared roof zones.

The installed ground support at 4 South mine is in good condition. All bolts are tight to the roof and show no signs of yielding. A thin film of rust coats most of the exposed plates and rebar ends in the mine, installed beyond approximately one year.

The corrosion of a rock bolt is very complex and its mechanisms and rate are affected by many factors, which also change during the corrosion process. Sandolm et al. (1993) list the following contributing factors:

- Oxygen
- Other gases (Carbon dioxide, sulphur dioxide, and sulphur trioxide)
- Salts in the dissolved water, pH value, organic compounds, rate of water inflow, and velocity of the flow
- Humidity conditions
- Temperature
- Pressure
- Joints in the rock mass
- Consistency of the water
- Conductivity of the rock type

The longevity of a rock bolt is enhanced with resin encapsulation. Kendorski (2000) reported on the the U.S. Bureau of Standards empirical relationship for calculating the pitting corrosion ratio for steel in soil of various types. The governing equation is:

$$P = kT^n \quad (4.1)$$

where: P = pit depth in 0.001-inch units

k = 28.8 for open-hearth low-alloy steels

n = 0.58 for open-hearth low-alloy steels

T = time in years

Rearranging to solve for T :

$$T = (P/k)^{1/n} \quad (4.2)$$

It is understood that bolt failure resulting from corrosion would occur prior with some percentage of the bolt diameter still intact. A 19-mm diameter rebar (375 0.001-inches half-diameter) used in the 4 South mine is predicted to corrode in 83.5 years.

The corrosion resistance of rock bolts was reported by Kendorski (2000) who referenced a study done in Yxhult Mineral AB's Centralgruvan mine in Sweden by Helfich (1990). Helfich overcored different types of rock reinforcement installations in a corrosive underground environment and found very minimal corrosion in 16-year old ungrouted mechanical and resin-grouted rock bolts.

4.2 ROCK BOLTING SUPPORT THEORIES

The ultimate purpose of roof bolting is to assist the rockmass in supporting itself by increasing frictional and shear resistance across bedding planes, and other discontinuities. Dolinar et al. (2001) suggests that the reinforcement mode is dictated to the bolts by the ground. Because roof bolts interact with the rock mass, they have many advantages when compared with earlier standing support systems.

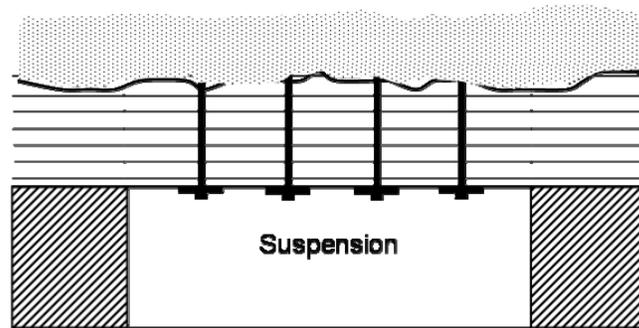
Rock bolting theories are the foundation of ground support design. Rock bolting support theories idealize the interaction of bolting systems with the rockmass. In this chapter five rock bolting support theories are discussed: suspension, beam building, beam theory, plate theory, and Voussoir beam building. Supporting massive sandstone roof with rock bolts is governed strongly by beam building, the effect of which can be estimated using beam and plate theory within the limits of the analyses. Suspension theory relates primary to ground support applied to stabilizing roof wedges and increasing the stand-up time of large intersection spans.

4.2.1 Suspension

Suspension involves pinning, via rockbolts, the immediate to an overlying, more competent bed that spans the opening. The strength of the roof bolt must exceed the weight of the rock it suspends by the ratio of the imposed safety factor. The overlying

unit is typically self-supporting. Figure 4.2 illustrates the roof suspension with rock bolts.

Figure 4.2: Roof Suspension with Rockbolts



The bolts have to carry the dead weight of the strata between bolt heads and anchors. Theoretical equations (Peng, 1984) assume that immediate roof would completely separate from the main roof such that it is suspended entirely by the bolts. The portion of weight of the immediate roof supported by the abutments on both sides of the opening is ignored. Therefore, such equations estimate the upper limit of load a bolt could bear while achieving the suspension effect.

Suspension governs the support of gravity wedges in the absence of surface support with high shear strength. In particular, the use of deep anchored cablebolts in conjunction with shallower penetrating rock bolts aims to secure the immediate roof with rockbolts and suspend the entire system with cablebolts.

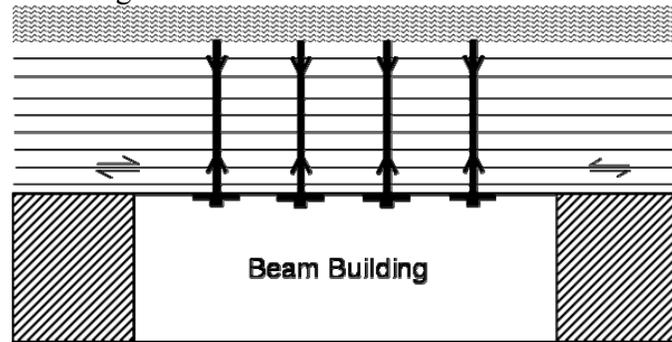
In cases where the strong and self-supporting main roof is beyond the anchoring horizon of the roof bolts to achieve adequate suspension, the mechanism of beam building can be employed to reduce sagging and separation of roof laminae cause manifested as vertical movement and horizontal movement along the bedding interfaces.

4.2.2 Beam Building

Beam building relies on tying the roof together with rock bolts to create a beam where an overlying, competent and self-supporting anchor horizon is out of reach. In coal measures, the bolts reinforce the strata by maintaining friction on the bedding planes, and

controlling dilation of failed roof layers. It is general, the practice of beam building is much more complex than that of suspension, particularly in design. Figure 4.3 illustrates the practice of beam building with rock bolts.

Figure 4.3: Beam Building with Rock Bolts.



Beam building design can be subdivided into beam theory and plate theory. Beam theory is an approximation of the plate theory and is generally used to analyze roof condition in entries, where the ratio of entry length to room width is greater than two. The theory is also limited to cases where the ratio of room width to layer thickness is greater than eight. Each roof layer is idealized as a transversely loaded beam spanning the entry. The ends of the beam, over the pillar ribs, are commonly considered fixed, although simply supported end conditions are sometimes applied.

Plate theory is used when the ratio of the longer lateral dimension (length) of the plate to the shorter dimension (width) is less than two. This situation occurs at intersections where the length and width are usually equal. The plate theory is limited to cases where the width to plate thickness is greater than 4. The edges of the plate are commonly assumed to be fixed, and the maximum stresses and deflection occur at the center of the plate.

Application of these theories is limited to relatively thin roof layers having properties that can be approximated by a homogeneous, linearly elastic material. Both beam and plate theory are elaborated on in subsequent sections.

4.2.3 Beam Theory

When the distance that ordinary roof bolts can reach to anchor for suspension is

insufficient, rock bolts can be used to tie bedding planes to together such that a rock beam is created. Without rock bolts, sagging and separation of roof laminae induces both vertical movement and horizontal movement along the bedding interfaces. Bolts through these layers can prevent or greatly reduce horizontal movement, and the tension applied to the bolts manually on installation or induced by rock vertical displacement clamps the layers together, making all the layers have to move with the same magnitude of vertical displacement. Frictional forces, corresponding to the magnitude of bolt tension, are induced along the bedding interface, also making horizontal movement difficult.

Peng (1984) provided the following theoretical equation to determine the maximum bending strain at the clamped ends of an elastic and homogeneous composite beam:

$$\varepsilon = \frac{wL^2}{2Et} \quad (4.3)$$

where, ε = maximum bending strain,

w = Unit weight of the immediate roof,

E = Modulus of Elasticity,

t = Thickness of the composite beam;

L = Length of immediate roof,

Equation 4.3 shows that the thicker the beam, the smaller the maximum strain induced at the clamped ends. In other words, the clamping action produces a beam building effect.

Panek (1956) indicated that beam building effects increase with decreasing bolt spacing, increasing bolt tension, increasing number of bolted laminae, and decreasing roof span.

Xiu (1990), who studied the mechanisms of rock bolting in gateroads of retreating longwalls in China found that beam building increases both the bending strength and stiffness of the composite rock beam created.

For a uniformly loaded, fixed-end beam, the maximum shear and tensile stresses occur at the ends and the maximum stresses and deflection are calculated as follows (Hanna et al. 1991):

$$\tau_{\max} = \frac{3qW}{4t}, \sigma_{\max} = \frac{qW^2}{2t^2}, D_{\max} = \frac{qW^4}{32Et^3} \quad (4.4)$$

where, τ_{\max} = maximum shear stress

σ_{\max} = maximum tensile stress

D_{\max} = maximum deflection

q = uniform load per unit length

W = room width

t = roof layer thickness

E = Young's modulus

Equation 4.4 shows that a 50% increase in entry width corresponds to a factor of 2.25 increase in the maximum tensile stress, and a five-fold increase in the maximum deflection. Furthermore, a 50% increase in the beam thickness corresponds to a factor of 2.25 reduction in the maximum tensile stress and a factor of 3.4 reduction in the maximum deflection.

Singh et al. (1994) describe the failure mechanism in laminated sequences with thick beams overlain by thinner beams with smaller flexural rigidity:

“Failure initiates first in the thick beams. With sufficient deflection of the thick beams, load is transferred to the thin beams which may also fail. Because ultimate failure occurs at smaller deflection for thin than for thick beams, the total weight of the thin beams may eventually be transferred to the thick beams, causing their failure.”

Stankus and Peng [1996] proposed the Optimum Beaming Effect (OBE), which is defined as the roof beam that has no separation within or above the bolted range and used the shortest possible bolt. It purports that high installed tensions can be substituted for bolt length. They argue that longer bolts elongate more in response to load, therefore

allowing more roof deformation. This method has been implemented in a finite-element model. There does not appear to be sufficient justification for this theory. Molinda et al. [2000] found that shorter, tensioned bolts had higher roof fall rates than longer, untensioned ones in three or four cases where both bolts were used in the same mine.

Van der Merwe (1998) lists the following three primary modes of failure for sandstone beams:

1. Flexure under the influence of its own weight plus the weight of material underneath which is suspended from it by bolting and the weight of softer material overlying it (flexure mode), or
2. A combination of the above mode and the effect of horizontal stress, or
3. Pure buckling caused by excessive horizontal stress, as determined with the Euler equation.

4.2.4 Plate Theory

Plate theory is governed by the following equations which determine the maximum tensile and deflection of a uniformly loaded, clamped edge plate (Hanna et al. 1991):

$$\sigma_{\max} = \frac{6BqW^2}{t}, D_{\max} = \frac{AqW^4}{Et^2} \quad (4.5)$$

where,

σ_{\max} = maximum tensile stress

D_{\max} = maximum deflection

q = uniform load per unit area

W = shorter lateral dimension (width)

t = thickness

E = Young's modulus

Values for constants A and B are given in Table 4.1.

4.2.5 Voussoir Beam Building

Voussoir beam theory conceptualizes the roof as a series of laminae interrupted by vertical joints, the support of which involves developing a compression arch. Evans

(1941) introduced the Voussoir arch concept into rock engineering to explain the stability of a jointed rock beam. Diederichs et al. (1999) updated the model which considers deflection due to self weight, external loads such as load from the rock above, water pressure and support, and the deformability of the beam.

Table 4.1: Plate Theory Values for Constants A and B.

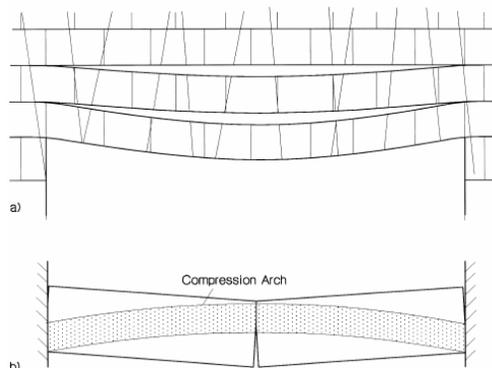
L/W	A	B
1	0.0138	0.0513
1.1	0.0164	0.0581
1.2	0.0188	0.0639
1.3	0.0209	0.0687
1.4	0.0226	0.0726
1.5	0.024	0.0757
1.6	0.0251	0.078
1.7	0.026	0.0799
1.8	0.0267	0.0812
1.9	0.0272	0.0822
2	0.0277	0.0829

L/W Length-width ratio of plate

The primary modes of failure assumed for a Voussoir beam are buckling or snap-through failure, lateral compressive failure (crushing) at the midspan and abutments, abutment slip and diagonal fracturing. Shear failure is observed at low span-to-thickness ratios (thick beams), while crushing and snap-through failure is observed at higher span-to-thickness ratios (thin beams).

Voussoir beam theory differs from elastic beam theory primarily in the fact that the Voussoir beam material has no tensile strength in the horizontal direction. Figure 4.4 illustrates the concept of a Voussoir arch. Stability guidelines for Voussoir beams will be presented in the discussion of critical span.

Figure 4.4: Idealized Jointed Beam (a) and Voussoir Analogue (b); after Diederichs et al. (1999)



4.3 ROOF SUPPORT DESIGN METHODS

Various methods for the design of roof support systems have been proposed through the years with the ultimate objective of maintaining safe conditions in underground mining environments. The effectiveness of roof support systems is intimately linked to the geotechnical behavior and geologic makeup of the rockmass comprising the roof. Detailed geotechnical investigations are therefore, important to design effective roof support systems.

Two- and three-dimensional dead weight wedge analysis, historically conducted with a stereonet and mathematical formulas is now facilitated with powerful CAD software like Autodesk AutoCAD and Google Sketchup, and the more tailored software package from Rocscience, Unwedge. Empirical and analytical tools round out the primary design references which were investigated to determine the most effective roof support system for the 4 South mine.

In this section, three categories of roof support design will be detailed: limit equilibrium, empirical and analytical.

4.3.1 Dead-Weight Design

The concept of dead-weight design was proposed by Obert and Devall (1967). It is a limit equilibrium method wherein dividing the sum of the forces promoting the event by the sum of the forces opposing the event produces a safety factor. The following equation, developed by Obert and Duvall, can be used to determine the required bolt capacity to support a dead weight slab of roof rock:

$$P = \left[\frac{U \times t \times W_e \times R}{n + l} \right] SF \quad (4.6)$$

where: P = required bolt capacity

U = unit weight of the rock;

t = thickness of suspended rock;

n = number of bolts per row;
 W_e = entry width;
 R = row spacing; and
 SF = safety factor.

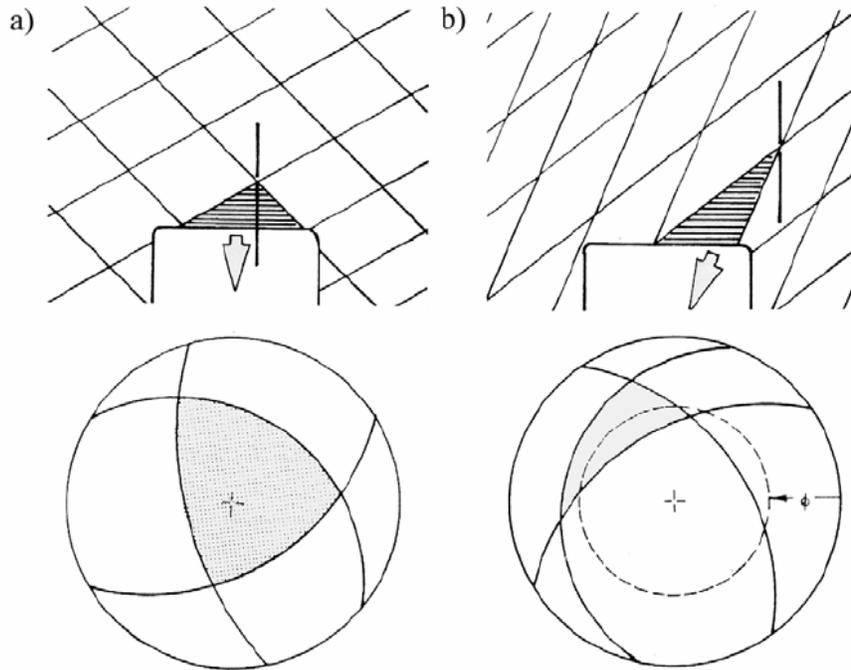
Equation 4.6 predicts bolt capacities of 7.5 and 11.2 tonnes for support patterns of five bolts per row on 1.2-meter centers and four bolts per row on 1.5-meter centers in sandstone ($U = 2.6 \text{ t/m}^3$, $t = 1.6$ meters) for a roadway of 6-meter width respectively. Both standard rebar and mechanical bolts are relevant to this analysis.

A more commonly assessed application of dead-weight design is stereonet wedge support analysis. Modes of structurally controlled failure can be analyzed by the means of the stereographic projections technique, described in detail by Goodman (1989).

For a wedge to form in the roof of an excavation, at least three joint planes must exist that separates the wedge from the rock mass. This will be visible on the stereonet by the great circles of three joint planes intersecting each other and form a closed figure. If this figure surrounds the centre point of the stereonet it will be a gravity driven fallout. (Hoek and Brown, 1980)

If the great circles of the joint planes form a closed figure but do not surround the centre point of the stereonet, failure can only occur by sliding on one of the joint surfaces or along one of the lines of intersection (Hoek and Brown, 1980). This condition is represented stereographically by the three great circles falling to one side of the centre of the net. An additional condition which must be satisfied for sliding of the wedge to occur is that the plane or the line of intersecting along which sliding is to occur should be steeper than the angle of friction ϕ . This condition is satisfied if at least a part of the intersection figure falls within a circle that is represented by the angle of friction in the stereonet. Figure 4.5 illustrates the conditions of gravity fall roof wedges and sliding failure of roof wedges.

Figure 4.5: Conditions for a) Gravity Fall or Roof Wedges and b) Sliding Failure of Roof Wedges; after Hoek and Brown (1980).



A wedge analysis using hemispherical projection assumes that the joint strength is purely frictional and disregards the contribution from the stresses in the vicinity of the opening to the normal stresses acting on a joint, may increase the stability considerably.

The Rocscience software, Unwedge (2005), is an excellent tool for analyzing wedges defined by structurally mapped joint and fault sets in the presence of standard rock bolt support systems. Discontinuities are assumed to be ubiquitous; however, the wedge geometries defined by discontinuities can be adjusted to reflect in-situ location and dimensions where appropriate. Joints, bedding planes and other structural features included in the analysis are assumed to be planar and continuous. Software input consists of site-specific parameters of the selected shear strength criterion (Mohr-Coulomb, Barton-Brandis, or Power Curve) water pressure and joint waviness to determine the safety factor. Ground support can be added to the analysis to assess the capacity of a ground support system to support select wedges defined in the analysis.

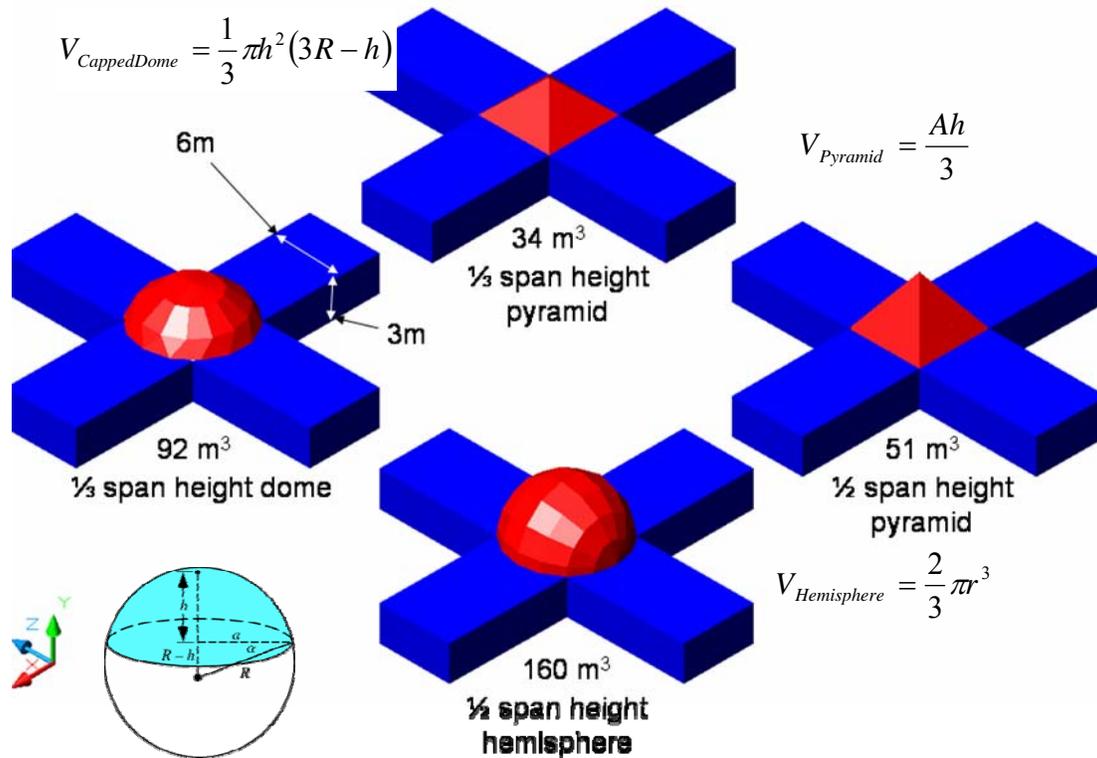
In the absence of structural data, geometric idealizations of critical roof wedge dimensions can be reviewed to determine the capacity of the ground support systems in a

roof wedge support application. Half-span and third-span symmetrical 2-D wedges and 3-D pyramids are typical examples of idealized wedges. Uncommon to this analysis are hemisphere and capped-dome geometries for wedges. Three-dimensional software packages like Autodesk AutoCAD and Google Sketchup are very useful in this application, particularly in the assessment of effective anchorage depths of roof bolts. In the case where anchorage is achieved, but insufficient to provide the full capacity of the bolt, the “working” capacity of each rock bolt can be determined from the anchorage depth achieved and the bond strength (generally expressed in units of force per unit length) of the rock bolt. Figure 4.6 shows three-dimensional graphical representations of four intersection wedge geometries: third-span height dome, third-span height pyramid, half-span height hemisphere, and half-span height pyramid. The geometries were restricted to half-span and third-span height which is regarded as a practical limit for wedge development.

The percent difference in volume between the half-span and third-span height pyramids and the half-span height hemisphere and third-span height dome is approximately 40%. Pyramid or cone wedge geometries for roof intersections in massive sandstone are realistic assumptions, while dome and hemispherical geometries are more applicable to intersection roof wedge geometries in weaker mudstone / siltstone geology.

Figure 4.7 shows three-dimensional graphical representations of the four intersection wedge geometries supported with #6 1.8-meter long point-anchor rebar on 1.2-meter centers. Resin encapsulation is represented in green. A half-meter of resin anchorage (30% of installed bolt length) and a 15-tonne bolt capacity is assumed. The illustrations were produced using Autodesk AutoCAD. Figure 4.8 illustrates the same model setup and analysis, but with the pattern bolting spacing increased to 1.5 meters center to center. Comparison of the two figures clearly illustrates the importance of bolting density, position and anchorage depth in the support of intersection wedged. Table 4.2 summarizes results of the both analyses with regards to effective support load provided and the resulting factor of safety. Stress-confinement joint-friction effects are not considered in the analysis.

Figure 4.6: Three-Dimensional Symmetrical Intersection Wedge Geometries with Calculated Volumes and Equations for 4-way Intersections Formed by Cross-cuts and Roadways of 6-meter Width.



The results indicate 1.8-meter point-anchored #6 resin rebar provide adequate support capacity for third- and half-span pyramid geometries in 4-way intersections of 6-meter square dimensions. Anchorage depth and bond strength provided by pattern bolting was completely ineffective for the half-span height hemisphere geometry and partially effective for the third-span height dome geometry.

Two-dimensional analysis of wedges and angled rock slabs is another useful CAD assisted analysis. The objective of the analysis is to assess the capacity of a pattern bolt system to support wedges and angled slabs. The inclination of discontinuities in the model is varied and the support capacity of the pattern bolt system is assessed to arrive at a critical discontinuity dip for a given support pattern. Figure 4.9 illustrates this analysis applied to #6 1.8-meter long point-anchor rebar pattern bolted on 1.2-meter and 1.5-meter centers respectively. The assumed resin-anchorage and support capacity for each bolt is assumed to be 0.5 meters and 15 tonnes respectively.

Figure 4.7: Three-dimensional Representation of Idealized 4-way Intersection Wedge Geometries with Installed Pattern Bolting of #6-1.8 meter Long Point-anchor Rebar on 1.2-meter Centers.

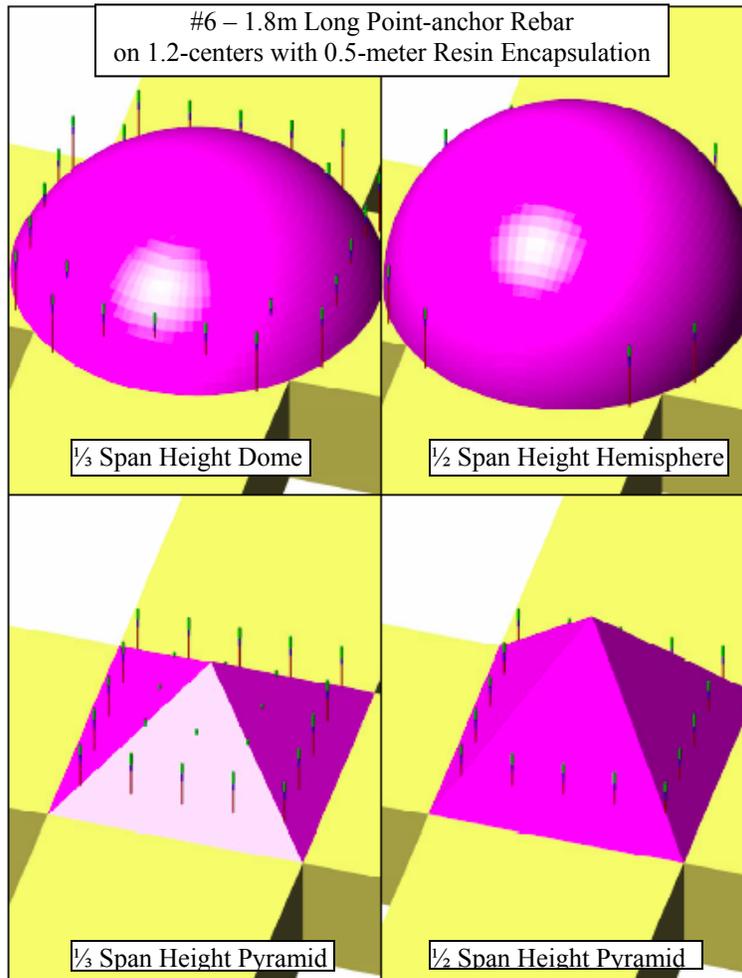
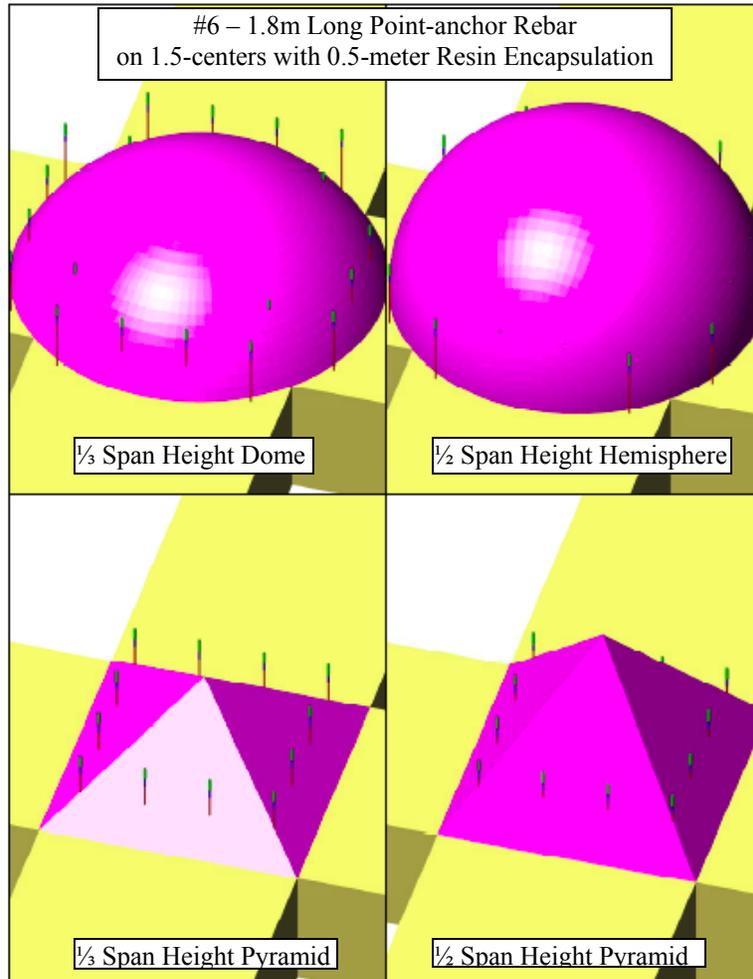


Table 4.2: Safety Factors Against Failure for Idealized 4-way Intersection Wedges with Patterned Point-anchor Rebar Support Installed.

	1/3 Span Height Dome	1/2 Span Height Hemisphere	1/3 Span Height Pyramid	1/2 Span Height Pyramid
Weight (t) [2.5t/m³]	230	400	85	127.5
1.2 x 1.2 m Pattern				
[# Good Bolts/Support Provided (t)]	24/360	0/0	20/300	16/240
1.5 x 1.5 m Pattern				
[# Good Bolts/Support Provided (t)]	12/180	0/0	12/180	12/180
1.2 x 1.2 m Pattern				
Factor of Safety	1.57	0	3.53	1.88
1.5 x 1.5 m Pattern				
Factor of Safety	0.78	0	2.12	1.41

Figure 4.8: Three-dimensional Representation of Idealized 4-way Intersection Wedge Geometries with Installed Pattern Bolting of #6-1.8 meter Long Point-anchor Rebar on 1.5-meter Centers.



Based on this analysis, the critical inclination angle for symmetrical wedges and inclined slabs supported with #6 1.8-meter point-anchor resin rebar on a 1.2-meter center support pattern is approximately 40 and 25° respectively. In comparison, the 1.5-meter center pattern realized critical inclination angles of 35 and 20° respectively for symmetrical wedges and inclined slabs. The same exercise applied to #6 1.5-meter length point-anchor resin rebar on 1.5-meter centers results in critical angles of 30° and 18° respectively.

Figure 4.10 below illustrates the use of the Rocscience software, Unwedge for the 4 South mine to analyze the capacity rock bolting patterns to support intersection roof wedges defined by the primary joint / fault sets for the mine. Figure 4.10 shows an

evaluation of 1.8-meter length mechanical and resin-anchored bolts on 1.2 and 1.5-meter centers to stabilize the largest possible intersection roof wedge. Unwedge is a powerful and simple tool in this application.

Figure 4.9: Two-dimensional Symmetrical Gravitational Wedge Support Analysis for Wedge and Inclined Slabs at Varying Angles of Inclination for a 6-meter Wide Roadway Supported with 1.8 meter Long #6 Point Anchor Rebar on 1.2-meter and 1.5-meter Center Patterns.

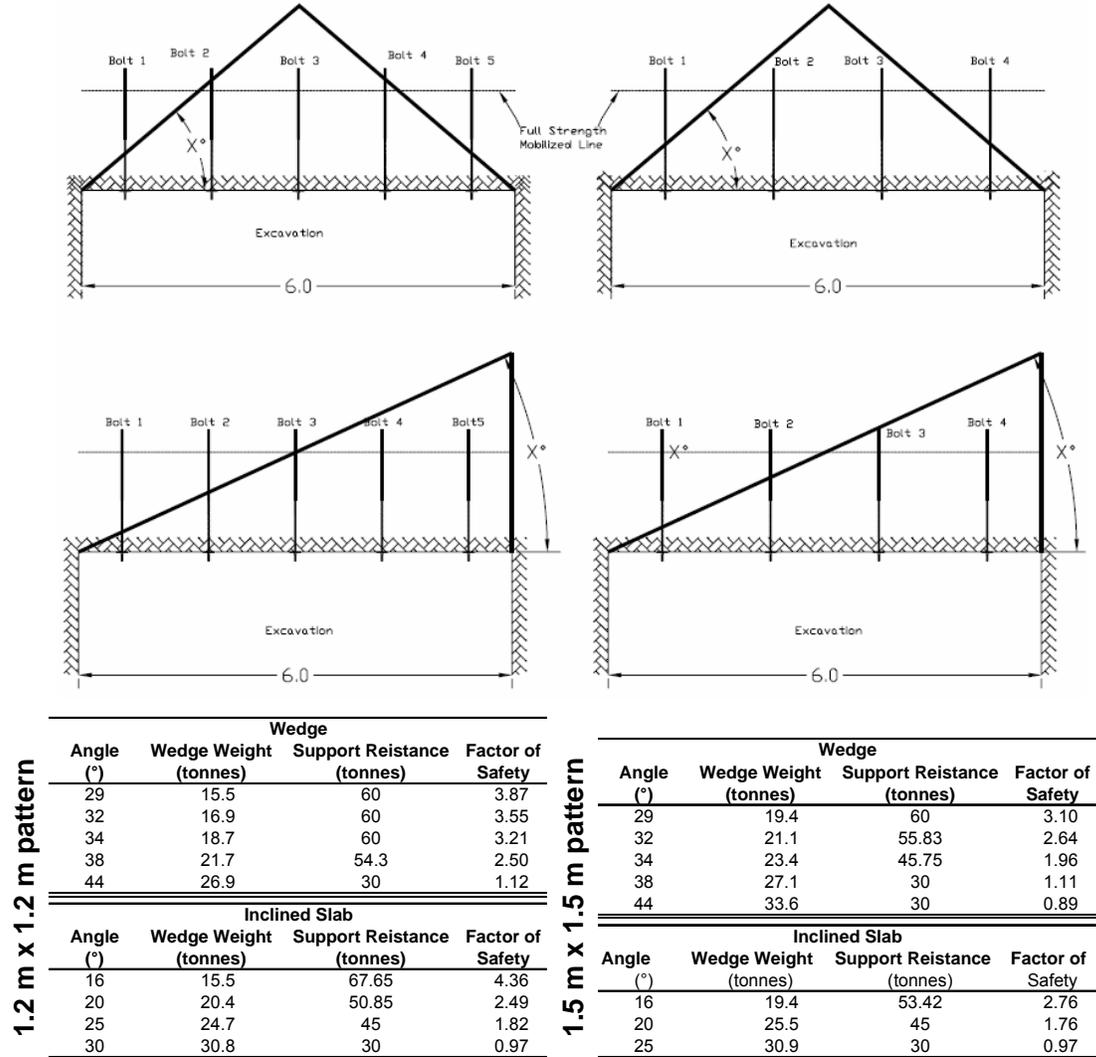
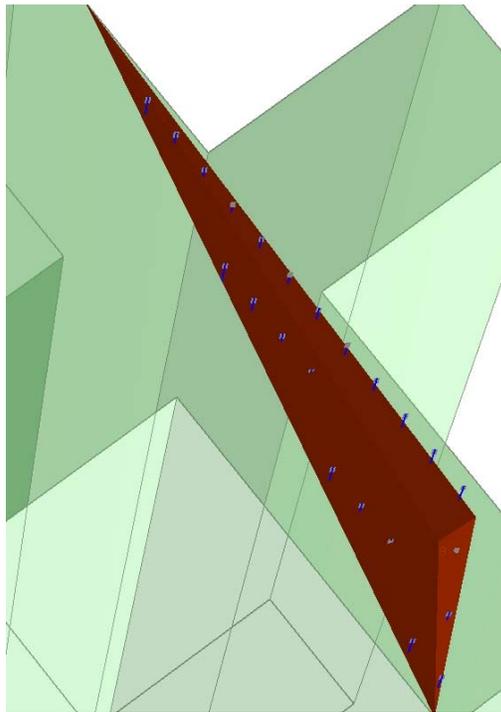


Figure 4.10 indicates, with regards to support capacity of the maximum theoretical wedge defined by the primary joint and fault orientations in the 4 South mine, that both mechanical and point-anchor resin rebar of 1.5- and 1.8-meter length on 1.5-meter centers are sufficient with safety factors of 1.76 and 2.51 respectively.

The same conclusion is arrived at when the analysis is replicated for mechanical and point-anchor bolts of 1.5-meter length with resulting safety factors of 1.51 and 2.41 respectively.

Figure 4.10: Unwedge Gravity Wedge Analysis for 4 South Mine.



Combination	Required Support Pressure (t/m ²)	Wedge Volume (m ³)
Joints 1,2,4	8.8	510
Joints 2,3,4	7.4	495
Joints 1,3,4	6.0	408
Joints 1,2,3	0.2	1

Ground Support Pattern

Type:	Point Anchor	Mechanical
Tensile Capacity (t)	15	10
Plate Capacity (t)	15	10
Anchor Capacity (t)	-	10
Bond Strength (t/m)	45	-
Bolt Length (% of Length)	30	-
Bolt Length (m)	1.8	1.8
Bolting Pattern (mxm)	1.2 m ² , 1.5 m ²	1.2 m ² , 1.5 m ²
Support Pressure (t/m ²)	12.5, 10	8.3, 6.7

Trend	Wedge Weight (t)	Point Anchor Resin Rebar		Mechanical	
		FS (1.5 x1.5 m)	FS (1.2 x1.2 m)	FS (1.5 x1.5 m)	FS (1.2 x1.2 m)
0	137	2.51	3.12	1.76	2.21
10	127	2.40	3.13	1.81	2.21
20	99	2.42	4.31	1.71	3.03
30	45	3.88	6.28	2.70	4.42
40	42	4.32	6.12	2.88	4.08
50	39	4.68	6.18	3.12	4.16
60	35	4.24	6.37	3.11	4.24
70	43	3.82	6.42	2.55	4.40
80	46	4.14	6.14	2.59	4.31
90	137	2.51	3.70	1.76	2.79

4.3.2 Rock Load Height Design

The rock load height concept is a slightly more sophisticated version of the deadweight theory. Originally proposed Terzaghi (1946), the theory predicts the load on the supports based on the rock quality and by the roof span. Unal (1983) defined the rock load height for coal mining:

$$h_t = \left(\frac{100 - RMR}{100} \right) w_e \quad (4.7)$$

where, h_t = Rock load height, m
 RMR = Rock Mass Rating
 w_e = Entry width, m

This equation implies a maximum rock-load height equal to span. It can further be shown that the rock load is proportional to the cube of the span. When the geology is not uniform, and roof falls are truncated by an overlying self-supporting strong bed, the rock load height may be constant regardless of the intersection span. In this case, the rock load increases in proportion with the square of the span (Mark 2000).

Unal developed equations for support design based upon his research of coal mine roof support in the US. The equations were summarized as design charts for 6-meter wide entry and corresponding 4-way intersections. The governing equations are listed below. Unal's design charts are provided in Figures 4.11 and 4.12

For point-anchor bolts:

$$S = \frac{\sqrt{C}}{SF \times \lambda \times h_t}, L = \frac{h_t}{2}$$

For full-column resin bolts:

$$S = \frac{\sqrt{C}}{SF \times \lambda \times h_f}, L = \frac{\sqrt{B^2 \times h_t}}{300}$$

where: γ = unit weight (lbs/ft³)

B = span (ft)

h_t = rock load height (ft)

C = bolt capacity (lb)

L = bolt length (ft)

S = bolt spacing (ft)

SF = safety factor

Unal's roof support guidelines for entries and 4-way intersections in the 4 South mine are summarized below in Table 4.3.

4.3.3 Empirical Roof Support Design

Empirical roof support designs are based on case histories from active and inactive mining operations. It is imperative that in the application of empirical design guides, the reference database is reviewed for congruence with the site of its application and furthermore, that a site-specific database be developed.

There are many empirical roof support guidelines developed for application in hard rock; however, guidelines based on coal mine case histories are less common. Hartman et al (1992) summarizes the empirical rules of Hoek and Brown (1980), the US Corps of Engineers (Anon, 1980), and the US Bureau of Mines (Lang and Bischoff, 1982) regarding bolt length and spacing which have been applied in coal mining.

Figure 4.12: Roof Support Guidelines for 4-way Intersections Formed by 6-meter Wide Entries; after Unal (1983).

ROOF ROCK CLASS	ROCK MASS RATING (RMR)	ROCK LOAD HEIGHT, H _r (FT)	SUPPORT SPECIFICATIONS		ALTERNATE SUPPORT - PATTERNS					
			MECHANICAL BOLTS	RESIN BOLTS	MECHANICAL BOLTS/POSTS			RESIN BOLTS/POSTS		
VERY GOOD	90	2.8	L = 2.5' S = 5' x 5' G = 40 φ = 3/4" C = 8.8 tons	L' = 3.0'/3.0' S = 5' x 5' G = 40 φ = 3/4" C = 9.8 tons						
			L = 3.0' S = 5' x 5' G = 40 φ = 5/8" C = 6.2 tons φ _P = 4"	L' = 4.0'/4.0' S = 5' x 5' G = 60 φ = 1" C = 23.7 tons						
GOOD	70	8.5	L = 5.0' S = 5' x 5' G = 40 φ = 3/4" C = 8.8 tons φ _P = 5"	L' = 3.0'/5.0' S = 5' x 5' G = 60 φ = 3/4" C = 13.2 tons φ _P = 4.5"						
			L = 6.0' S = 5' x 5' G = 40 φ = 5/8" C = 8.8 tons φ _P = 6"	L = 3.5'/6.0' S = 5' x 5' G = 60 φ = 3/4" C = 13.2 tons φ _P = 5.5"						

They are as follows:

1. *Minimum bolt length:*

Greatest of: (a) 2 x the bolt spacing.

(b) 3 x the width of critical and potentially unstable rock blocks defined by the average discontinuity spacing in the rock mass.

(c) For spans less than 6 m, bolt length of 0.5 x span. For spans 6 m to 18 m, interpolate between 3 m and 5 m lengths, respectively. For excavations higher than 18 m, sidewall bolts are one-fifth of wall height.

2. *Maximum bolt spacing:*

Least of: (a) 0.5 x bolt length.

(b) 1.5 x the width of critical and potentially unstable rock blocks.

- (c) 2 m; greater spacing than 2 m makes attachment of wire mesh difficult.

3. *Minimum bolt spacing: 0.9 m*

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

The above empirical rules applied to the 4 South sandstone roof recommend 3.0-meter bolt lengths on 0.6- to 0.9-meter centers. This design is considered conservative and operationally, unrealistic for the 4 South mine where 1.8-meter torque-tension rebar on 1.2 meter centers has performed exceptionally well in a roof support role.

Farmer and Shelton (1987) developed an empirical bolt design for rockmasses with up to three discontinuity sets and excellent joint properties. The sections of this guideline applicable to the 4 South mining conditions are shown in Table 4.4.

Farmer and Shelton recommend 1.8-meter long fully grouted rebar bolts on 0.9-meter centers. The close spacing of the bolts recommended is very conservative for massive sandstone roof, which is generally self-supporting at development spans up to 8.5 meters wide.

Table 4.4: Roof Bolt Design Guideline; after Farmer and Shelton (1987)

Excavation Span (m)	Number of Discontinuity Sets	Bolt Design	Comments
<15	≤2 inclined at 0-45° to horizontal	L = 0.3B S = 0.5L (depending on thickness and strength of strata). Install bolts perpendicular to lamination where possible with wire mesh to prevent flaking.	The purpose of bolting is to create a load-carrying beam over span. Fully bonded bolts create greater discontinuity shear stiffness. Tension bolts should be used in weak rock; subhorizontal tensioned bolts where vertical discontinuities occur.

4.3.4 Analytical Support Design

Dolar et al. (2001) collected geotechnical, mining, geologic and other roof bolting factor data from 37 coal mines in the US and developed an empirical roof bolt design

guideline for coal mines. The design first requires quantification of the coal mine roof rating, pre-mining stress level and mining-induced stresses. Next, an appropriate diagonal intersection span (I_s) is calculated by the following equation:

$$(I_s) = \frac{9.5 + (0.2 \times CMRR)}{2}, (m) \quad (4.8)$$

Next, a bolt length formula, derived from a modification of the Unal (1983) rock load height equation, and based on the beam building mode is applied as detailed below:

$$L_B = 0.12(I_s) \log_{10} (3.25H) \left[\frac{100 - CMRR}{100} \right] (m) \quad (4.9)$$

where (I_s) = diagonal intersection span, m

(H) = depth of cover, m

Finally, $PRSUP_m$, a parameter consisting of the support capacity times a dimensional factor calculated as the length of bolts divided by the entry width times the spacing of bolts, is determined with the following equation:

$$PRSUP_m = \frac{Lb \times Nb \times C}{Sb \times We} \quad (4.10)$$

where, Lb = length of bolts (m)

Nb = number of bolts per row

C = Capacity of bolts (kN)

Sb = Spacing of bolts (m)

We = entry width (m)

The minimum recommended PRSUP is approximately 43.5. The suggested value of $PRSUP_m$ for shallow cover is determined as:

$$PRSUP = 225 - 3.33CMRR \quad (4.11)$$

The Dolinar et al. method described above applied to the 4 South mine, recommends 4 x 1-meter long rock bolts on 1.5-meter centers with a capacity of 15 tonnes. The suggested value for shallow cover is below the minimum recommended value of 43.5 for PRSUP.

Cain (1999) presented a limit equilibrium analytical equation based on roof beam theory. The numerator consists of the Daws (1988) equation developed to calculate the value of support offered based on the increase in strength developed in the rock mass as a result of the confinement offered by roof bolting and the thickness of the roof beam. The denominator consists of the Bieniawski formula for required support. The Cain (1999) equation is shown below:

$$SF = \frac{\tan^2\left(45 + \frac{\phi}{2}\right) \cdot \frac{\sigma_b \cdot A_b}{S_a \cdot S_b} \cdot t}{\frac{100 - RMR}{100} \cdot B^2 \cdot \gamma} \quad (4.12)$$

where: ϕ = internal angle of friction of the rock mass = $5 + RMR/2$

σ_b = yield stress of the bolt (tonnes/m²)

A_b = cross section area of bolt at threads (m²)

S_a, S_b = bolt spacing across and along the roadway (m)

t = reinforced beam thickness (defined as the rock bolt length minus the resin anchorage length minus 50% of the threaded length at the end of the bolt)

RMR = Geomechanics rock mass rating (%)

B = roadway width (m)

γ = unit weight of rock (tonnes/m³)

Cain suggests the rock bolt pull out load be substituted for the terms $\sigma_b \cdot A_b$ in the above equation if it is less than the yield load of the bolts. The equation is only valid if the Coates (1981) reinforced beam criteria is satisfied; specifically, that the bolt length, L is two times the maximum of the greatest of the bolt spacing:

$$L / \max imum(S_a, S_b) > 2$$

A recommended safety factor of 1.3 for temporary roadways and 1.5 for permanent openings is stated for this analysis.

Using the Cain equation, a safety factor of 1.4 is calculated for a 6-meter wide roadway with a reinforced beam thickness of 1.3 meters, bolts of 1.8-meter length on 1.5-meter centers, a 15-tonne bolt yield strength, and a RMR of 70%. The Coates criteria, however requires a bolt length of 3.0 meters, similar to the empirical guidelines of Hoek and Brown (1980), the US Corps of Engineers (Anon, 1980), and the US Bureau of Mines (Lang and Bischoff, 1982) regarding bolt length. A 3.0-meter length rock bolt is too long for installation at the 4 South mine.

4.4 ROCK BOLTS

Rock bolts are an intrinsic type of roof support installed into the roof. Roof bolts are loaded as the roof deforms, and they interact with the rock to reduce bed separation by confinement. The selection of rock bolts available to an underground coal mine operator is astounding. Figure 4.3.1 below presents the results of a 2005 US underground coal mine ground support usage survey in the by Tandolini et al. (2006).

The design, properties and performance of a rock bolt can be summarized by the following key characteristics:

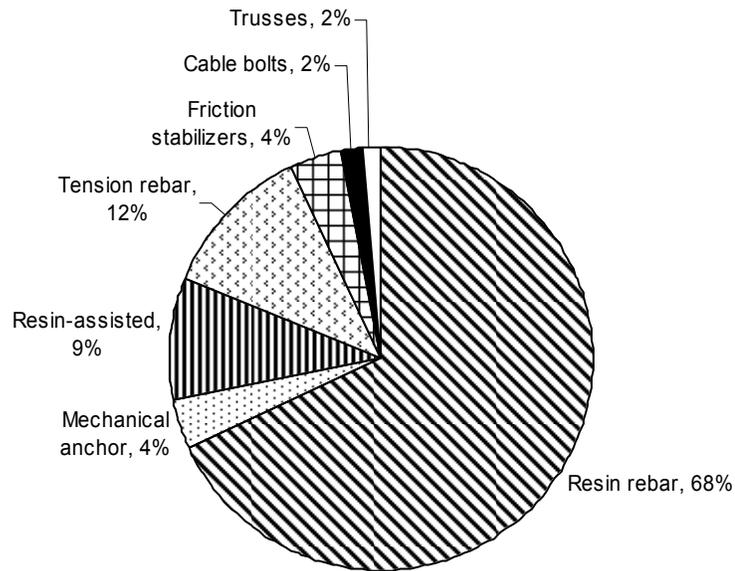
Passive or active: Passive bolts load the strata immediately upon installation. Active bolts require the strata to move before the rock bolt loads the strata.

Point-anchored or full-column: Point-anchored bolts typically have anchorage lengths less than 30% of their length. Full-column bolts are anchored along their entire length, or very close to. Full-column resin installations can be hampered due to high insertion pressures up to 45 MPa and the resulting loss of resin to fractures and glove fingering (Giraldo et al. 2006).

Stiffness: Stiffness increases in direct proportion to the area or the square of the bolt diameter. A 22-mm diameter bolt is twice as stiff as a 16-mm diameter bolt, all other things being equal. Also, as the free bolt length below the anchorage length increases, the stiffness of the bolt decreases, meaning that longer bolts have a softer response and

allow more roof movement to occur for the same for the same increase in bolt load. Fully grouted bolts are normally considered to be stiffer than point-anchor bolts.

Figure 4.13: Distribution of Ground Support Use in US Underground Coal Mines in 2005; after Tandolini et al. (2006).



Capacity: Higher capacity bolts can carry more broken rock and are also capable of producing more confinement and shear strength in the rock, and they may be pre-tensioned to higher levels. The capacity C of a roof bolt, defined as the point of yield of the bolt, is normally determined by the bolt diameter (D) and the grade of the steel (G):

$$C = \frac{\pi}{4} GD^2 \quad (4.13)$$

For rebar, the diameter is usually given as a number, where #5 rebar is 16mm in diameter, #6 is 19mm. The grade of the steel is normally given in thousands of psi, where a grade 40 steel is 40,000 psi (280 MPa), etc. The grade and the diameter, and some other information including the bolt length, are typically stamped on the head of the bolt.

Roof rock bolts should be installed as soon as possible after a cut is mined to counteract excessive movement of the roof. Excessive roof sag can reduce the strength of the roof rock mass by reducing the frictional resistance on bedding planes and other discontinuities. The degree of roof sag and potential damage from delayed bolting

depends chiefly on the stress level, the span and the roof quality. A comparison of three rock bolts and five different installation types are shown below in Table 4.5.

Table 4.5: Rock Bolt Comparison of Mechanical and Point-anchor and Full-column Resin Tensioned and Forged-head Rebar Bolts.

BOLT	INSTALL TYPE	ACTIVE/ PASSIVE	STIFF/ SOFT	CORROSION RESISTANCE	EASE OF INSTALLATION	PULL OUT RESISTANCE	SHEAR RESISTANCE	RELATIVE COST
Mechanical	Point-anchor	Passive	Soft	Medium	Good	Medium	Poor	Cheap
Tension	Point-anchor	Active	Soft	Medium	Medium requires training	Very Good	Medium	Cheap
	Full Column - single resin		Stiff	Good			Very Good	Expensive
	Full Column - dual resin	Passive	Stiff	Good			Very Good	Expensive
Forged head	Point-anchor	Passive	Soft	Medium			Medium	Cheap
	Full Column		Stiff	Good			Very Good	Expensive

Molinda et al. (2000) reported on roof quality and roof bolt performance from interviews and reconnaissance at 41 US coal mines and found eighty percent of underground development was supported with untensioned bolts while the other 20% was tensioned. All untensioned bolts were full grouted, while nearly all tensioned bolts were ungrouted or partially grouted. The most common bolt length was 1.5 meters (38%) followed by 1.8 meters (30%) and 1.2 meters (22%).

Mechanical bolts, tensioned rebar and forged-head rebar in lengths of 1.2-, 1.5- or 1.8-meters were chosen for consideration at the 4 South mine.

4.4.1 Mechanical Rock Bolts

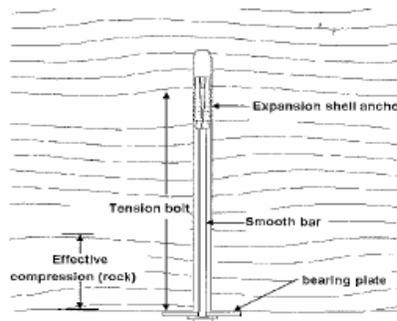
The mechanical bolt is an active fixture that loads the strata immediately upon installation. The mechanical bolt is one of the oldest ground support fixtures. A mechanical anchor consists of a steel tendon with an expandable shell which is inserted into the hole. When the bolt is rotated, the shell expands. Once the shell is fully expanded, the bolt is tensioned by either tightening the nut against the washer at the free end or by further rotation of the bolt in the case of fixed head bolts. Because of the long free length of the steel tendon, mechanical anchor bolts can stretch when load is applied. It is therefore soft support, even though it is active by virtue of its pre-tension.

The mechanical bolt can be installed quickly and anchorage capacities up to 12 tonnes have been achieved. The rock strength, more specifically shear strength, always controls

the anchorage and the amount of bolt tension that can be applied by the mechanical anchor. The critical issue of this bolting system is the “bleed-off” or loss of tension after installation. Frittering of the roof underneath the washer, slippage of the anchor and vibrations cause mechanical bolts to lose pre-tension quickly. Installed tensions can be up to 50% less than those required to ensure proper bolt installation and subsequent performance (Maleki et al., 1985). In certain mines corrosion is also a problem.

Mechanical anchors are relatively cheap and easy and quick to install and ideal for short term applications (less than 1 year). They are also suitable for suspending massive roof layers. Mechanical anchors are generally avoided in long term applications like main development, wet areas and thick, weak roof situations where beams have to be created. Figure 4.14 illustrates the components and installation of a mechanical rock bolt.

Figure 4.14: Mechanical Anchored Roof Bolt; after Tandolini et al (2006).



4.4.2 Tensioned Rebar

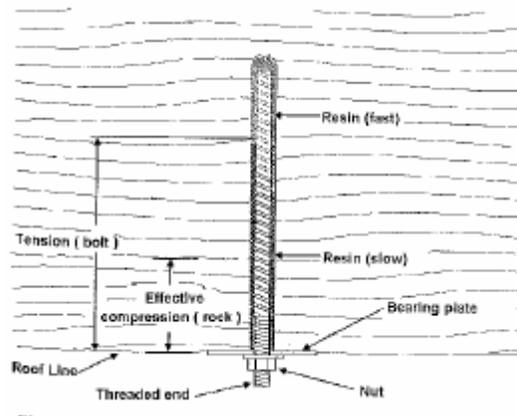
Tensioned rebar bolts are active fixtures in all applications except for full-column single resin. The support system consists of rebar that is threaded on the end to allow a nut to be installed and tightened against the bearing plate.

The point-anchored, tensioned resin rebar is like a mechanically anchored bolt, except that the expandable shell of the latter is replaced by a resin anchor. It remains soft active support, same as a mechanical anchor. A primary disadvantage of point-anchored rock bolts is that if the collar is disturbed, and plate contact is lost with the roof, the bolt is ineffective. Point-anchored tensioned resin rebar also require more time and care to

install than mechanical bolts. The only real advantage is that the anchor resistance can be increased by making the anchor longer.

Full-column, tensioned resin bolts can be installed with single or dual speed resin systems. Maximum stiffness or complete benefit of the tension rebar bolt system is attained when two speeds of resin are used. To achieve a full-column of resin and still be able to tighten or tension the bolt, the top portion is anchored with a fast setting resin and the bottom with a slower resin. The high stiffness of fully grouted bolts is attributed to the ability of the full column resin to quickly transfer the developed loads, via the bolt, back into the rock mass. Consequently, a significant resistance to rock movement is developed both axially and laterally. Loads that are developed along the bolt, if roof separations occur, are transferred back into the rock and the movement is resisted at the parting or separation. Without a bearing plate, fully grouted bolts can still function and resist rock movements, however at a reduced capacity. Adequate plates help resist roof movements in the lower 0.6 meters of the roof and are an important support element in roof reinforcement. Figure 4.15 illustrates the concept of the tensioned full-column bolt with dual-speed resin.

Figure 4.15: Tensioned Full-column Bolt with Dual-speed Resin; after Tandolini et al. (2006).



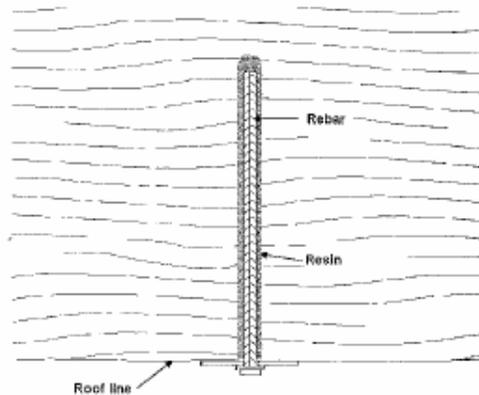
4.3.2 Forged-head Rebar

Forged-head resin-rebar bolts are passive fixtures. Some operators argue that the practice of applying an upward thrust force to the forged-head bolt during installation throughout

the resin set time, “locks” in tension in the installed bolt. The plate load of these bolts is low to nonexistent. Anchorage is good and some yield is present in the stretch available in the unanchored portion of the rebar. Due to their passive nature, they must be placed close to the active mining cycle with mining to assure that the deformation of the rock strata will load them.

Full-column resin installation is preferred with forged-head bolts to increase their effectiveness and guard against disturbances. A primary advantage of forged head bolts in lower seam heights is the relatively small amount of the fixture that protrudes after installation. Threaded rebar with threads cut into the rebar rather than via the cold-roll process, will have a slightly less ultimate load in comparison to a forged head rebar with all other properties being equal, due to the reduced diameter of the bar at the threads. Figure 4.16 illustrates an installed forged-head resin rebar bolt.

Figure 4.16: Forged-head Rebar Bolt; after Tandolini et al. (2006).



4.5 ROCK BOLT PULL TESTS

Rock bolt pull tests initiated in 1997 at the 4 South mine intended to evaluate alternative, less intensive and costly ground support options amenable to the good roof conditions in 4 South. Before and during this work, 1.8-meter long point anchor rebar were installed on 1.2-meter centers, strongly believed to be excessive during the time. In 1997, Quinsam Coal performed 3 standard pull tests on 1.8-meter long, Grade 60, #6 point-anchor resin rebar and 4 x 1.8-meter long, C1060 Grade #5 mechanical bolts. In the same year, shortly

thereafter, Thiessen assisted in the testing of 13 additional mechanical bolts with 1” FIF expansion shells.

In 2003, the author organized and conducted 28 standard pull tests on #6 forged-head point-anchor and fully grouted bolts of lengths 1.2, 1.5 and 1.8 meters. Five short-encapsulation pull tests were also performed. Compton et al. (2005) suggested the results of the SEPT tests are highly conservative relative to overcored standard pull tests. He added the SEPT test is still useful to confirm adequate anchorage and as an index test to compare relative anchorage strengths. Table 4.6 summarizes the record of pull tests for the 4 South mine. The location of the 2003 tests in the 4 South mine is shown in Figure 4.17.

Table 4.6: Summary of Rock Bolt Pull Test Details by Year at the 4 South Mine.

Year	Lead	# of tests	Type	Rockbolt		
				Details	Length [m]	Diameter [mm]
1997	QCC	4	M	Grade C1060, RH	1.8	16
1997	QCC	3	PA,TT	Grade 60 RH	1.8	19
1997	Thiessen	13	M	Grade C1060, RH	1.8	16
2003	QCC	8	PA, FH	Grade 60 RH	1.8	19
2003	QCC	4	TT,SEPT	Grade 60 RH	1.8	19
2003	QCC	10	FG,FH	Grade 60 RH	1.2	19
2003	QCC	10	FG,FH	Grade 60 RH	1.5	19
2003	QCC	1	FH,SEPT	Grade 60 RH	1.8	19

M = Mechanical FH = Forged Head SEPT = Short Encapsulation Pull Test
 PA = Point Anchor TT = Torque Tension FG = Full grouted

All tests were conducted with a standard pull test kit consisting of two assemblies:

1. a mechanical assembly which includes a claw, stand, and wing-nut
2. a hydraulic assembly which includes a 30-ton hollow core ram, hand pump, force-indicator gauge and hose.

Each rockbolt was pre-fitted with a 50mm mechanical collar to accept the “claw” adapter which couples the rockbolt to the pull-test assembly. All rebar rock bolts used for the 2003 pull test program were manufactured by Jenmar Corporation USA. Minova Lokset A-series resin was used in all rockbolt installations. The annuli for all tests was approximately 4mm. Tandolini (1998), in a US study found that annuli ranging from 2.5-6.5 mm all provided acceptable results in strong rock. All holes were overdrilled 2” to

allow room for the plastic encasing the resin. A torque of 150 to 200 ft-lbs - 2.7 to 3.6 tonnes using a 40:1 torque-tension ratio – was applied.

Figure 4.17: 2003 Rock Bolt Pull Test Locations.



For the 2003 series of pull tests, rock bolts were pre-loaded to one tonne. Bolt displacement was measured from the hydraulic ram displacement. A standard wooden ruler was used for measurements. A zero reading was taken prior to the start of the test. An average loading rate of 4.5 tonnes/min was maintained. On average six to eight readings were taken per test up to the yield load of the bolt. No bolts failed during testing. Prior to the start of the pull tests, safety chain was used to secure the pull test ram assembly to the roof, using the installed roof bolts and screen.

The procedure for short encapsulation pull tests (SEPT) involved installing each tested bolt with a partial 0.3-meter section of resin to achieve a resin encapsulation between 0.3 and 0.45 meters. This length provided enough material to ensure adequate mixing but

more importantly minimized the possibility of the bolt yielding prior to the bond failure occurring (Mark, 2000). The depth of encapsulation was determined by sliding a steel wire down the hole and measuring the depth to the resin. The purpose of the SEPT was to determine the effectiveness of load transfer between the resin and the borehole wall, called the anchorage factor. The anchorage factor is commonly termed the bond strength; however this is actually a misnomer because there is no adhesion between the resin and the rock, just mechanical interlock [Karabin and Debevic 1976]. The anchorage factor is determined by dividing the applied pulling load by the anchorage length.

4.5.1 Mechanical Rock Bolts

Seventeen mechanical rock bolt tests were performed in 1997 at the 4 South mine. All tests were completed in C-Road, 14 cross-cut in No.1 Mains. In two instances only the wedge pulled through the shell, both at 11.3 tonnes and a displacement of about 100 mm. In all other tests, no anchor failure occurred. In all other tests, the bolts reached their yield point and could not be failed. The yield point was reached within the first 10 mm of displacement at a load between 8.6 and 10.4 tonnes. In the 13 Thiessen tests, the load reached at the termination of the test ranged between 10.9 and 13.2 tonnes. The response of the mechanical bolts tested by Quinsam Coal achieved relatively high final loads ranging from 18 to 38 tonnes at a displacement of 50 to 55 mm. The reason for the stiffer response of these tested mechanical bolts is unknown. A graphical summary of all pull test results for mechanical bolts at the 4 South mine is shown below in Figure 4.18.

The use of mechanical rock bolts at the 4 South mine has been recommended by several researchers. Mraz (1997) performed a simple analysis based on the following parameters derived from the pull test results:

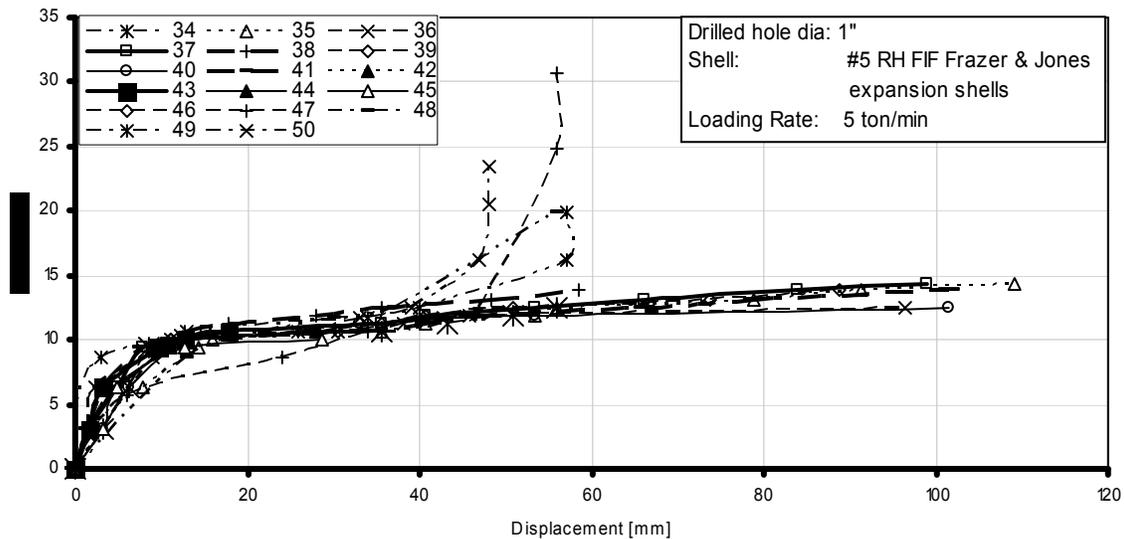
Rock bolt yield load: = 8.6 tonnes

Anchor failure load: = 11.2 tonnes

Yield Displacement: = 100 mm

Bolt Capacity: = 8.6 tonnes

Figure 4.18: 1.8-meter Long Mechanical Rock Bolt Pull Test Results.



In his report, he concluded:

“...the tested 16 mm x 1.8 m grade C 1060 bolt with mechanical expansion shells can be used on a spacing of 1.5 m x 1.5 m. A torque between 175 and 225 ft-lbs should be applied to each bolt. During the first three months of usage, no less than 50 bolts per month should be tested by torque wrench at random and results of this testing should be reported in writing.

...In faulted zones, extra bolts must be installed as required and additional support, such as post or cribs, must be used if necessary.”

Cullen (1996) performed a preliminary assessment of roof support requirements for 4 South mine based on empirical and analytical methods. The input data for the analysis was obtained from drill core logging and rock outcrop mapping at the 4 South mine portals. His recommendation for primary ground support were as follows:

- Bolt spacing: 1.5 m x 1.5 m
- Bolt length: 1.5 m minimum
- Bolt type: point anchor resin or mechanical
- Bolt capacity; 9 tonnes

Pakalnis (2001) also provided an endorsement of mechanical bolts, noting the following:

“The use of the mechanical bolt option should be successful within the sandstone back of the 4S for a test production panel. The specifications must be reviewed and assessed in terms of bolt strengths/plates/shells/lengths with respect to patterns similar to 2N. Quality control of the bolts must be assessed and tested on an ongoing basis. They are not to be used in structurally controlled/faulted ground.”

4.5.2 Forged-head Bolts

A total of 29 pull tests were conducted on forged-head bolts of bolt lengths 1.2, 1.5 and 1.8 meters. About 1 meter (66% of bolt length) of resin anchorage was achieved for all tests. A loading rate of about 4.5 tonnes/min was maintained during all tests. A summary of the pull test results for full-column resin forged-head bolts of 1.2-meter length is shown below in Figure 4.19: No bolts pulled out or failed for all ten tests. The yield load ranged between 12 and 14.5 tonnes at 7 mm displacement. The maximum displacement of 32 mm was achieved at a load of 21 tonnes.

A summary of the pull test results for full-column resin forged-head bolts of 1.5-meter length is shown below in Figure 4.20. No bolts pulled out or failed during testing. The yield load ranged between 12 and 16 tonnes at 8 to 12 mm displacement. The maximum displacement of 27 mm was achieved at a load of 20 tonnes.

A summary of the pull test results for full-column resin forged-head bolts of 1.8-meter length is shown below in Figure 4.21. No bolts pulled out or failed for all tests. The yield load ranged between 14 and 18 tonnes at 5 mm displacement. The maximum displacement of 25 mm was achieved at a load of 19 tonnes.

The short-encapsulation pull test achieved a yield load of 18.5 tonnes at 65 mm displacement. The anchorage length was 26mm (12.3”) corresponding to a minimum anchorage factor of 18 tonnes/ft (50 tonnes/m) in massive sandstone. Excellent anchorage capacity is indicated for forged head bolts in the immediate roof.

Figure 4.19: 1.2-meter Long Full-column Forged-head Rock Bolt Pull Test Results.

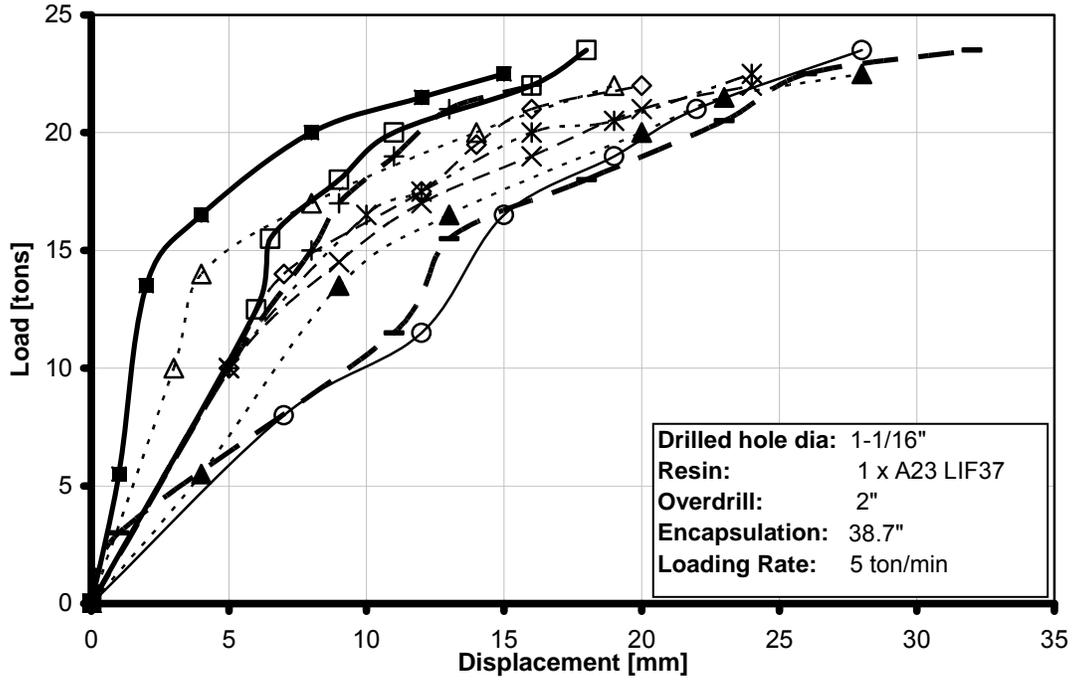


Figure 4.20: 1.5-meter Long Full-column Forged-head Rock Bolt Pull Test Results.

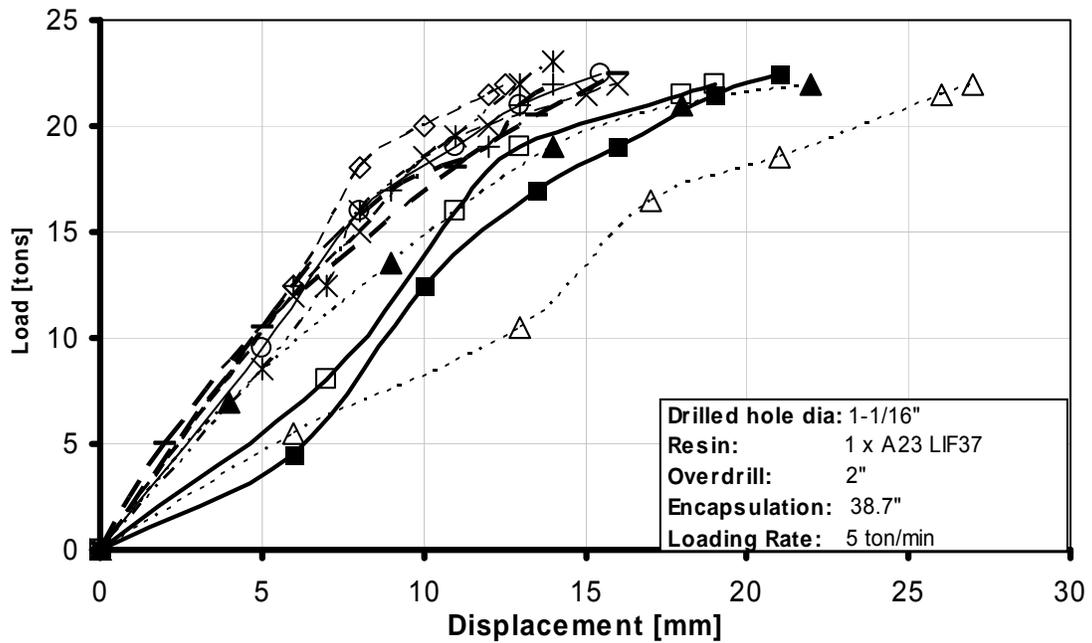
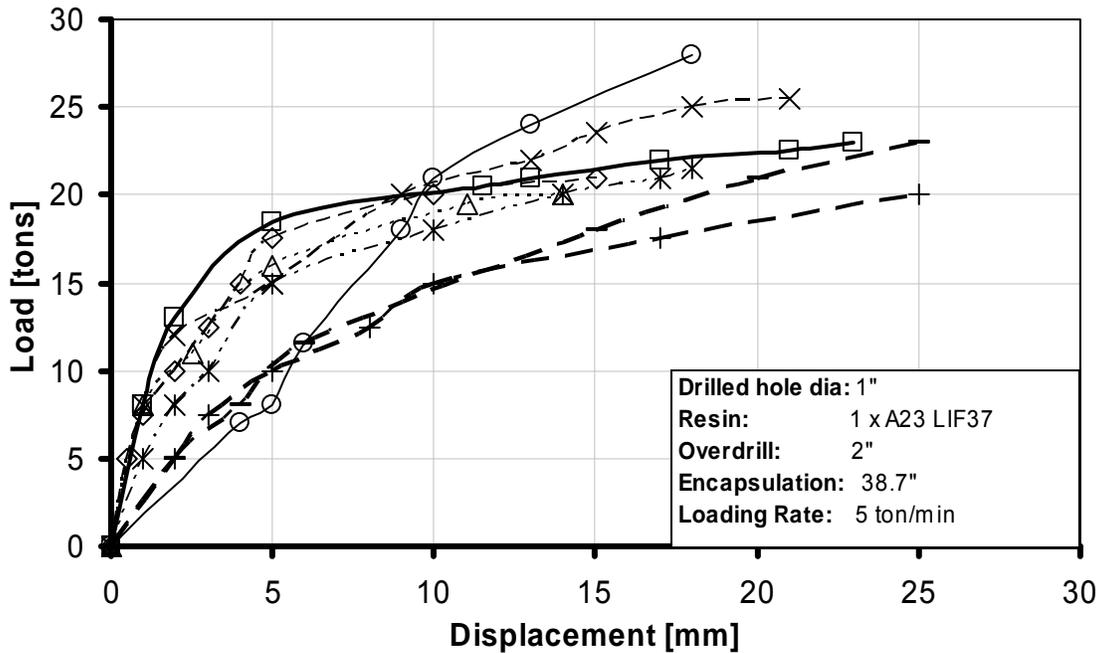


Figure 4.21: 1.8-meter Long Full-column Forged-head Rock Bolt Pull Test Results.



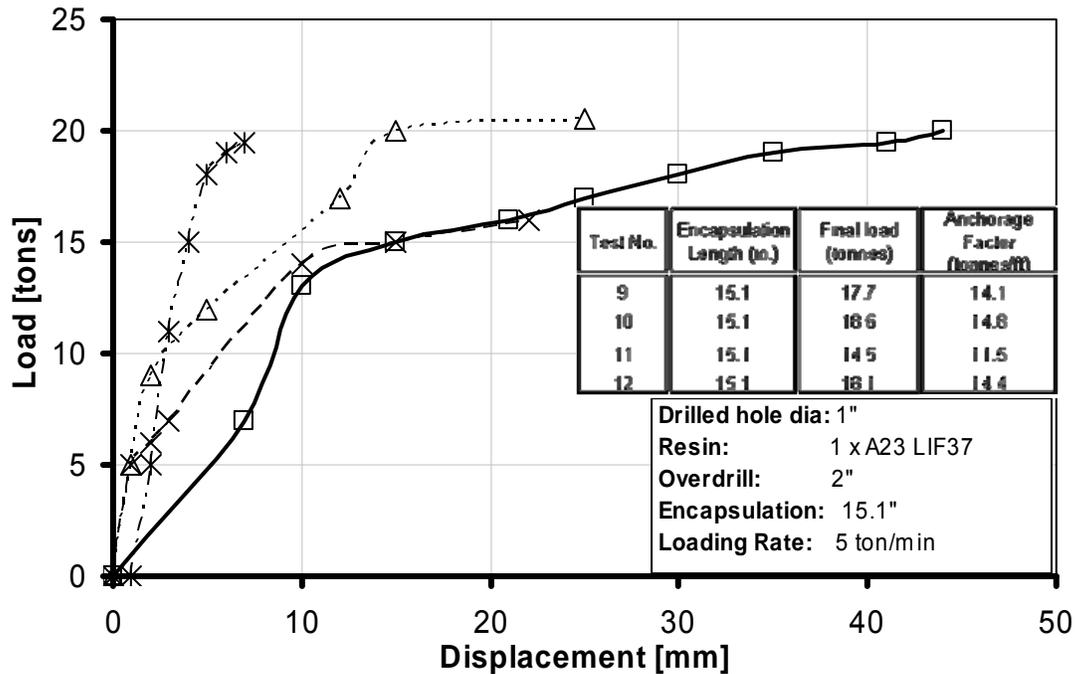
4.5.2 Tensioned Rebar Bolts

Four pull tests (No.9-12) were conducted on tensioned rebar of 1.8-meter length. Resin anchorage was set at 0.38 meters for all tests. A summary of the SEPT results for tension rebar bolts of 1.8-meter length is shown below in Figure 4.22. The yield load varied from 13 to 18 tonnes at 5, 10 and 15-mm displacement. Maximum displacement was 44 mm at about 18 tonnes. The anchorage factor ranged from 11.5 to 14.8 with an average of 13.7 tonnes/ft (45 tonnes/m). Mark (2000) reported typical anchorage factor for hard sandstone in Australia between 25 and 63 tonnes/ft.

4.6 CONCLUSIONS AND RECOMMENDATIONS

This section intended to determine the safest, productive and cost effective ground support strategy by examining the support density, rock bolt length, and optionally, type of permitted ground support for the 4 South mine. Currently the ground support system for 6-meter roadways consists of #6, 1.8-meter length point-anchored resin rebar on 1.2-meter centers with standard Grade 30 dome plates and no screen.

Figure 4.22: 1.8-meter Long Tensioned Rock Bolt SEPT Results.



The chief conclusions drawn from the investigation reported in this section are:

1. Rock bolt corrosion has been limited to thin surface rust films on the installed plates and exposed rebar. Reflection on a case history and reference to empirical estimates of rock bolt longevity strongly suggest that corrosion will not measurably affect the rock bolt strength until 15 years post installation date.
2. Elastic beam theory of rock bolt support function is the best general model for the 4 South massive sandstone roof. Where vertical discontinuities extend through the bolted roof thickness, Voussoir beam theory may be more appropriate.
3. Stereonet wedge support analysis, facilitated with Rocscience Unwedge software, has excellent application at the 4 South mine for assessing the capacity of the primary roof support to stabilize roof wedges.
4. Pyramid idealizations of intersection wedge geometry are most appropriate for massive sandstone roof.
5. A critical wedge was determined to be defined by joints angled at 30° above the roof line and a slab of rock formed with a joint angled at 18° based on

sandstone roof supported with #6 1.5-meter point-anchored rock bolts on 1.5-meter centers. The corresponding critical angles for wedges and slabs supported with 1.8-meter point-anchored rock bolts on 1.5-meter centers are 35 and 20° respectively. Wedges and slabs formed by joints angled more steeply will require supplemental support assessment.

6. Empirical estimates of support requirement for the 4 South sandstone roof suggest a conservative range of rock bolt lengths and pattern support spacing and a greater support load density and anchorage depth that has successfully been provided by the current installed ground support. Methods based on beam theory require excessive bolt lengths of 3.0 meters for a 1.5-meter center to center bolt spacing.
7. The Unal (1983) analytical support predictions prescribe #6 1.5 meter long mechanical and point-anchored rebar on 1.5-meter center to center spacing - a design well suited to the roof conditions in 4 South.
8. Farmer and Shelton (1987) and Cain (1999) prescribe 1.8 meter long rock bolts on 0.9- and 1.5-meter center to center support spacing. Dolinar et al. (2001) recommend 1.0-meter long #6 point-anchored rebar on 1.5-meter center to center spacing.
9. Resin rock bolts are finding increasing application in underground coal mines, particularly untensioned, forged headed rebar, with 1.5-meter length being the most popular.
10. 1.8-meter length, 16mm diameter Grade C1060 mechanical bolts on 1.5-meter centers are recommended as primary roof support by Mraz (1997) based on pull tests and the resulting bolt minimum bolt capacity of 8.6 tonnes at 100 mm displacement. A program of quality control torque testing is also prescribed within the first three months of usage at a minimum rate of 50 bolts per month.
11. Cullen (1996) reviewed empirical and analytical roof support design methods and recommended either 9-tonne capacity, 1.5-meter minimum length point-anchored resin rebar or mechanical bolts on 1.5-meter centers as roof support for the 4 South mine.

12. Pakalnis (2001) acknowledged the good match between the sandstone roof and mechanical rock bolts based on underground observations; however, he cautioned that quality assessment and control must be part of their application, and that they would not be appropriate in structurally controlled/faulted ground.
12. Forged head bolts performed very well in underground pull tests. The average yield load and corresponding displacement for rebar lengths of 1.2, 1.5, and 1.8 meters are shown below in Table 4.7. The bond strength from a single SEPT pull test on a 1.8-meter length was 50 tonnes/meter.

Table 4.7: Summary of Pull Test Results – Forged Head Bolts.

Length (m)	Avg. Yield Load	Avg. Displacement at Yield load (mm)
1.2	13	7
1.5	14	10
1.8	16	5

13. The average bond strength of SEPT tests on 1.8-meter torque tension rebar was 45 tonnes/meter at an average yield load of 15.5 tonnes.

Based on the conclusions itemized above, recommendations for ground support at the 4 South mine are summarized below in Table 4.8.

The Type 1B support system recommendation is based on the presented results of other researchers and requires updating to before implementation in the 4 South mine. The material properties of currently available mechanical bolts, including the anchorage capacities of the shells should be tested thoroughly before the Type 1B standard is adopted into the 4 South primary ground support plan. The torque-testing protocol recommended by Mraz (1997) should be followed until a site-specific QA/QC protocol is developed at 4 South for mechanical bolts. Supplementary support consisting of No.9 gauge screen for surface spalling control, 4 m long passive cablebolts for wedge support

and timber cribs and posts for rib and roof support should be reviewed for inclusion on a case by case basis or according to design rules.

Table 4.8: Ground Support Design – 4 South Mine.

TYPE 1A	
Steel Grade:	60
Yield Load/Capacity (tonnes):	15
Bolt Type:	Fully grouted Forged-head resin rebar
Resin (m):	1 x 4' x A23 LIF37 resin or equivalent
Length (m):	1.5 meters
Bolts per row:	4.00
Rib to rib-side bolt spacing (m):	0.75
Bolt to bolt spacing (m):	1.50
Row spacing:	1.50
Plate:	3/8" Grade 30 flat plate
Screen:	Not required
TYPE 1B	
Steel Grade:	C1060
Yield Load/Capacity (tonnes):	15
Bolt Type:	Mechanical bolt w/ hardened steel washers
Resin (m):	None
Length (m):	1.8 meters
Bolts per row:	4.00
Rib to rib-side bolt spacing (m):	0.75
Bolt to bolt spacing (m):	1.50
Row spacing:	1.50
Plate:	3/8" Grade 30 flat plate
Screen:	Not required
SUPPLEMENTAL SUPPORT	
Faulted/ Structurally Controlled Ground	
12' Passive Cable bolts	Deeper suspension
No.9 Gauge wire mesh	Surface ravelling control
Posts	Roof Span reduction/support

CHAPTER 5 COAL PILLAR DESIGN OPTIMIZATION

5.0 INTRODUCTION

Roof bolts support the vertical roof load above an underground excavation on a localized scale while coal pillars support the overburden load on a much larger scale. Coal pillars are intentionally left in place to support the overburden and maintain the integrity of openings. They are usually square or rectangular in shape and define the location of roadways and cross-cuts in an underground coal mine.

Coal pillars are a form of extrinsic ground support whose failure can contribute to a myriad of ground control problems, some very serious. Rib failures may result in increased entry spans which, in turn, can contribute to impaired floor and roof stability. Redistribution of stresses caused by pillar failures can disrupt confinement along bedding planes and other discontinuities in the roof, leading to wedge failures, and in some cases, large scale roof failures. The deformations and changes in rock mechanical properties accompanying pillar failure may also adversely influence stresses and deformations in the surrounding strata.

In addition to serving a support function in underground coal mine workings, coal pillars serve other important functions including:

1. restricting surface subsidence locally and regionally,
2. protecting critical service development from the effects of high load and abutment stress, e.g. conveyor/transport/ventilation roads.
3. minimizing fracturing of the roof strata and the formation of hydraulic connections to aquifers, surface water bodies and water-filled gobs, and
4. serving as ventilation partitions.

The design of pillars is basically approached by the traditional material strength equation:

$$\sigma_p > (SF)(\sigma_v)$$

where: σ_p = pillar strength, MPa

σ_v = stress imposed on the pillar, MPa

SF = safety factor, depends on rock conditions and mining requirements.

Traditional strength-based pillar design first requires an estimate of pillar stress and then an estimate of pillar strength. The factor of safety for the pillar is then evaluated by dividing the pillar strength by the induced pillar stress. An acceptable safety factor depends on the tolerable risk of failure. A safety factor of 2 is typical for pillars in main development headings or panels during advance mining while a safety factor of 4 and above is typical of barrier pillars and mains pillars that serve long term support roles.. Safety factors of 1.1 to 1.3 are typical for panel pillars in retreat mining.

Coal pillar system design is a function of many parameters including geology, discontinuities, seam strength, time, weathering, loading rate and the surrounding strata properties. Where one of these parameters becomes dominant, or new factors which were not present in the original area of study are encountered, the stability of the pillar system may not be predicted by the current pillar design methodology.

Peng (1986) listed four items that must be considered in the design of pillars:

1. Expected load history, including pre-mining pillar loads and abutment loads,
2. stress distribution within the pillar,
3. pillar strength, and
4. interaction between the roof, pillar, and floor.

This chapter intends to present the foundation of pillar design theory and tools for underground coal mines including pillar load estimation, pillar strength determination, and pillar design. In section one, the theory and limitations regarding the application of Tributary area theory (TAT) to estimating coal pillar loading will be presented. Abutment loading of coal pillars sourced from adjacent mined-out workings will also be discussed. Section two presents the assessment of pillar strength and the consideration of size, shape and discontinuity effects in pillar strength determination. Section three details empirical, analytical and numerical tools and guidelines for pillar design. Correctional formulas for

evaluating the strength of rectangular pillars is provided as well as guidelines for barrier pillar design. The NIOSH program Analysis of Retreat Mining Pillar Stability (ARMPS) is also discussed. Section four provides pillar design recommendations for primary pillar types for the 4 South mine. Section five summarizes the conclusions of the chapter.

5.1 PILLAR LOADING

In underground coal mines there are two types of loading conditions: development and abutment loading. The loading on the development pillars is generally governed by the tributary area theory (TAT). In this theory, the base assumption is that each pillar is responsible for bearing the weight of the overburden above it and the tributary area at a distance one-half the width of roadways and cross cuts on each side pillar side - that is that each pillar, regardless of position within the assessed mining area, is subjected to the same load. The prerequisites for using the TAT are that the layout must be regular and that the panel width must be greater than the depth below surface. TAT disregards the span of the mining area, the properties of the overburden and the presence of barrier pillars. Consequently, one can expect that it will overestimate pillar loads. Van der Merwe et al. (2003) provide the following reasons for the overestimation of pillar loads by TAT:

- It assumes that pillars are equally loaded, irrespective of their position in a panel,
- It ignores the stiffness of the overburden,
- It ignores the continuity of overburden rock, assuming that the deadweight of each rock column rests on the underlying pillar, and
- It ignores the load borne by inter panel pillars and other abutments.

Robert et al (2002) reported that tributary area theory is less valid at pillar width-to-height (w/h) ratios less than 1.25 and especially for extraction ratios greater than 65 per cent.

Stress levels within pillars according to TAT, can be determined using equation 5.1 (Brady and Brown, 1985). Figure 5.1 illustrates tributary area theory.

$$\sigma_a = \sigma_o \frac{(w+r) \times (l+r)}{w \times l} \quad (5.1)$$

where: σ_a = average post mining vertical stress (MPa)

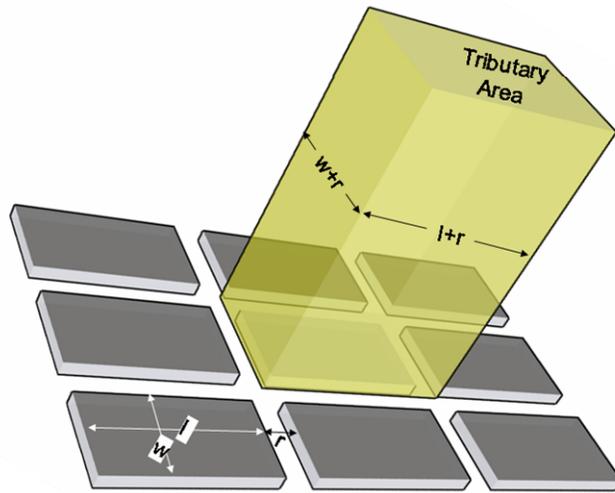
σ_o = pre-mining vertical stress (MPa)

w = pillar width (m)

l = pillar length (m)

r = roadway width (m)

Figure 5.1: Illustration of Tributary Area Theory Loading of Pillars.



The extraction ratio is a geometric characteristic which analyzes the relationship between the area of a pillar and the area of the adjacent opening along the horizontal plane. The extraction ratio for perpendicular intersections is determined by the following relationship:

$$e = \frac{(w+r) \times (l+r) - w \times l}{(w+r) \times (l+r)} \quad (5.2)$$

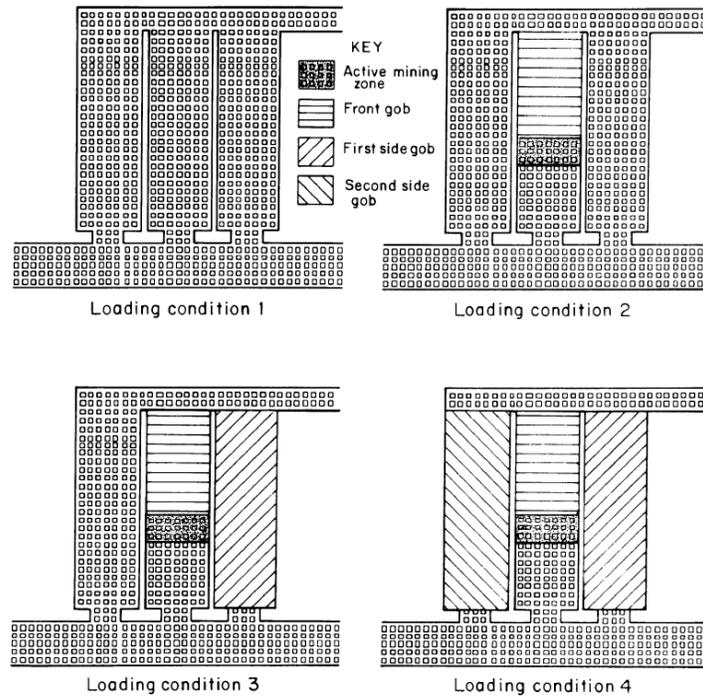
As the extraction ratio increases the pillars receive higher levels of stress as pillars become more slender. If the extraction ratio is known, the tributary stress can be calculated using equation 5.3:

$$\sigma_a = \frac{\sigma_o}{1 - e} \quad (5.3)$$

Modifications to the TAT formula have been made to account for the effects of horizontal stresses on pillars in a dipping seam by Pariseau (1982) and Hedley and Grant (1972); however, they will not be discussed in this study.

The load imposed by full extraction mining on the development pillars is the second type of loading scenario for coal pillars, termed the abutment load. Chase et al. (1994) present loading scenarios for retreat pillar mining development with front and side gobs as shown in Figure 5.2 below.

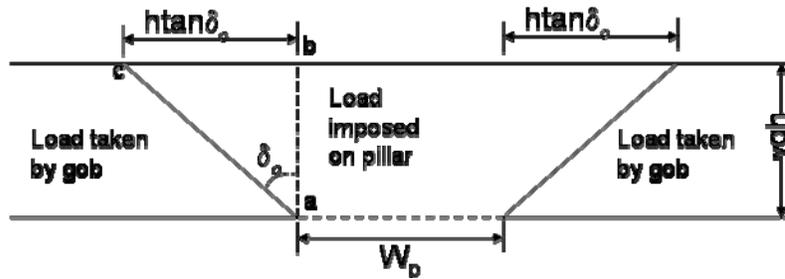
Figure 5.2: Retreat Pillar Mining Loading Scenarios; after Chase et al. (1994).



Peng (1978) reported that the magnitude of the abutment pressure ranges from 0.2 to 6.4 times that of the overburden weight, depending on the characteristics of the strata and their sequence.

Figure 5.3 illustrates the abutment loading concept reported by King and Whittaker (1971) which proposes that the abutment load imposed on a pillar is equal to the volume of rock defined by the vertical line ab and inclined line ac from the edge of the pillar. The internal angle between these two lines is termed the angle of draw, δ_o . The rockmass outside the triangular area is assumed to be supported by the gob material.

Figure 5.3: Abutment Load Imposed on Pillar from Adjacent Gobs; after King and Whittaker (1971).



Equation 5.4 below can be used to determine the pillar loading, P , on square pillars, when $W > 2h \tan \delta_o$, where W is the panel width:

$$P = \lambda h \left(W_p + \frac{W_o}{2} + \frac{h \tan \delta_o}{2} \right) (W_p + W_o) \quad (5.4)$$

where: W_o = entry width (m)

Equation 5.4 is only valid for pillars flanked on one side only.

Mark et al. (1992) reported alternative equations for the abutment load implicit in the Analysis of Retreat Mining Pillar Stability (ARMPS) program developed by NIOSH. The equations are as follows:

If, $GEXT > 2h \tan \phi$, $GEXT$ is the extend of the gob in meters:

$$LA = 0.5 \cdot \gamma \cdot D^2 \cdot \tan \phi \cdot EFW \quad (5.5)$$

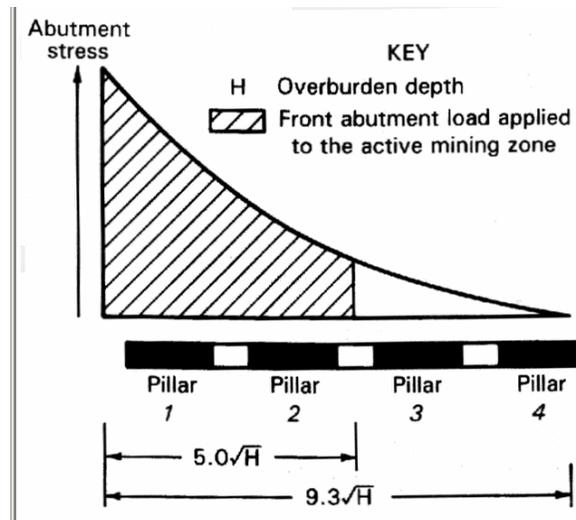
If, $GEXT < 2h \tan \phi$::

$$LA = \gamma \cdot \left(0.5 \cdot D \cdot GEXT - \frac{GEXT^2}{8 \tan \phi} \right) EFW \quad (5.6)$$

where: LA = abutment load
 γ = unit weight of rock
 EFW = extraction front width
 ϕ = angle of draw

Mark (1992) reports that abutment loads are distributed over pillars inbye the gob line a distance of $9.3\sqrt{h}$ and that 90% of the abutment load is imposed on pillars within a distance of $5\sqrt{h}$, where h is the depth of overburden. Figure 5.4 illustrates this concept.

Figure 5.4: Abutment Load Applied to Pillars Inbye the Gob Line; after Mark (1990)



The stress, σ_x at a distance, X from the edge of the gob is calculated using equation 5.7 below:

$$\sigma_x = \left(\frac{3LA}{9.3\sqrt{D}} \right) (9.3\sqrt{D} - X)^2 \quad (5.7)$$

5.2 PILLAR STRENGTH

The strength of a coal pillar is chiefly dependent on the size and shape of tested specimens. Lind (2002) reported on the following factors affecting pillar strength:

1. Pillar dimensions, including mining height,
2. jointing effects

3. loading system, “geological” and “local”
4. contact conditions
5. k ratio (ratio of pre-mining horizontal to vertical stresses)
6. stability of the roof and floor
7. pillar length to width ratio
8. dip
9. creep and other time effects

Testing of intact coal specimens in the laboratory is typically limited to 50-100mm cylindrical and/or cubical samples collected underground or from drill core. Laboratory test samples typically over-estimate rock material strength values because larger flaws or fractures are exhibited as specimen size increases. The point at which the specimen size becomes sufficiently large so that further reductions in material strength become insignificant is often referred to as rock mass strength. Bieniawski (1984) reported that cubical coal specimens from 0.9 to 1.5 meters are of this critical size and are representative of rock mass characteristics. Wilson (1981) suggested a factor of strength reduction applied to the laboratory value, should be based on the rock type. He suggests a factor of reduction of 0.2 for coal and unity for strong massive unjointed rock. Others have used reduction factors ranging from 40 to 80% to determine material strength from uniaxial strength values

5.2.1 Size Effect

The size effect refers to the difference in strength between the small-sized specimens tested in the laboratory and the in-situ mine pillars. Equation 5.8 is reported to adequately characterize the scaling of properties from laboratory measured data to in situ:

$$\sigma_1 = \frac{k}{\sqrt{h}} \quad (5.8)$$

where: σ_1 = scaled in-situ pillar strength (psi)

k = scaling factor (determined from in-situ material)

h = height of cubical pillar (inches)

The height, h , of the in-situ pillar to be analyzed should be set to 36 for $h \geq 36$ inches.

The scaling factor, k , can be determined using equation 5.9 developed by Gaddy (1956):

$$k = \sigma_c \sqrt{D} \quad (5.9)$$

where: σ_c = UCS of laboratory sample (psi) of diameter or cubic-size, D (inches)

The use of D for cylindrical or cubical specimens is valid for samples between 50 to 100 mm diameter or edge length.

Equation 5.9 predicts a scaled coal pillar strength of 5.9 MPa for No.3 seam coal. Mark and Barton (1997) reported that 6.2 MPa is a better approximation than can normally be obtained from coal testing, adding an adjustment the in situ coal strength is warranted if back-analysis of site-specific case histories indicates that it is significantly different from this value. The scaled pillar strength predicted with Equation 5.9 supports the approximation reported by Mark and Barton (1997).

5.2.2 Shape Effect

The shape effect refers to the ratio of diameter or width to the height of the specimens. The strength and behavior of a coal pillar can vary significantly depending on the shape of the pillar. Mark (1999) reported on three broad categories of pillar behavior defined by a range of width to height (w/h) ratios. They are summarized in Table 5.1.

Table 5.1: Pillar Classification of Failure Behavior at Ultimate Capacity by W/H Ratio; after Mark (1999).

Type	w/h	Failure behavior at ultimate capacity
Slender	<3 to 4	fail completely
Intermediate	4 to 8	do not shed entire load, non-violent pillar squeeze
Squat	10 +	strain-hardening

The shape effect is inherent in most of the empirical formulas proposed to determine the average ultimate strength of coal pillars. They can all be derived from the following generic formula:

$$\frac{\sigma_p}{\sigma_c} = A + B \left(\frac{W^\alpha}{H^\beta} \right) \quad (5.10)$$

where, σ_p = pillar strength in psi.

σ_c = UCS of a cubical pillar, psi (use equation 5.8)

W = pillar width, ft.

H = pillar height, ft.

There have been numerous pillar strength formulas developed over the years, specific to coal fields and valid for a range of width to height ratios. Table 5.2 provides a summary of these formulas referenced to equation 5.2 for comparison.

Table 5.2: Summary of Pillar Strength Formulas Referenced to Generic Pillar Strength Equation 5.10.

Year	Investigator	Constants				Sample Size	Coal or seam tested	Country
		A	B	α	β			
1911	Bunting	1.75	0.75	1	1	Small	Anthracite	United States
1911	Bunting	0.7	0.3	1	1	-	(²)	-
1912	Griffith	0	(³)	1	0.5	Large	Pittsburgh	-
1928	Zern	0	1	0.5	0.5	-	-	-
1939	Greenwald	0	0.7	0.5	0.5	-	-	-
1941	Greenwald	0	2.8	0.5	0.85	-	-	-
1954	Stear ¹	0	1.04	1	1	Small	Natal	Rep. South Africa
1956	Gaddy	0	(³)	0.5	1	-	Beckley and Pittsburgh	United States
1964	Holland	0	(³)	0.5	0.5	-	-	-
1966	Evans	0	(³)	-0.32	0	-	Deep Dufryn	England
1966	Evans	0	(³)	-0.17	0	-	Barnaly Hards	-
1967	Salamon	0	1	0.46	0.66	Large	Bituminous	Rep. South Africa
1967	Obert and Duvall	0.778	0.222	1	1	-	-	-
1968	Bieniawski	0	1.1	0.16	0.55	-	Witbank	-
1969	Bieniawski	0.4	0.22	1	1	-	-	-
1972	Wilson	0	448	1	1	-	-	England
1974	Wagner	1	0.58	1	1	-	Ustua	Rep. South Africa
1974	-	0	1.6	0.5	0.5	-	-	-
1974	Van Heerden	0.704	0.296	1	1	-	-	-
1976	Hustrulid	0	(³)	1	0.5	Small	Existing data	-
1976	Wardell	1000	20	2	2	Large	Newcastle	England
1977	Skelly ¹	0.78	0.22	1	1	-	Pocohontas	United States
1978	Sorenson, Parisez	0.983	0.307	1	1	-	-	-
1981	Bieniawski	0.64	0.36	1	1	Large	-	United States
1987	Sheorey et al.	0	1	0.5	0.86	-	-	-

¹ Used σ_c as cubical strength, while all others used $\sigma_c=1$

(²) From a different mine

(³) Constant that changes with different seams

5.2.3 Discontinuity Effect

There are three primary types of discontinuities in coal: cleat (minor) joints, bedding planes, and major slips. The frequency, orientation and shear strength of these features will affect the in-situ strength of coal mine pillars. Assuming the coal rock mass strength to be isotropic can result in significant errors in the estimation of the strength of pillars.

The effect of discontinuities on pillar strength was recently investigated by Taylor (2003) who, relating South African coal pillar data to a fracture/frequency classification developed by Laubsher (1990), found the relationship between the frequency of discontinuities and in-situ pillar strength could be described by a polynomial function.

Ramamurthy et al. (2000) developed equations based on laboratory strength tests to account for the fact that pillar strength is a minimum when joints are inclined at 45° to the major stress vector. A parameter was introduced to account for the fact that the effect of joints diminishes as the width to height ratio of the pillar increases. The pillar height was introduced in the equations so that the pillar size would be accounted for.

Esterhuizen (1997) developed joint classification system for coal, finding that coal from the South African coal fields could be readily classified into six classes, or combination of classes depending of the nature of discontinuities in the coal as shown in Figure 5.5. Esterhuizen's classification system was developed in recognition of the difficulties in mapping discontinuities in coal and is commonly applied as a measure of comparison between coal seams.

The effect of discontinuities on pillar strength is commonly accounted for in design by reducing the calculated pillar strength by an amount equal to the reduction in the seam strength predicted by classification methods. However, Esterhuizen, who conducted numerical model studies, reported that the reduction in the strength of coal pillars due to the presence of jointing is not constant for all width-to-height ratios and effect of jointing becomes less pronounced as the width-to-height ratio increases.

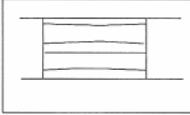
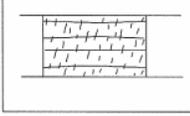
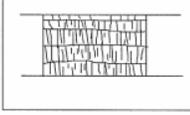
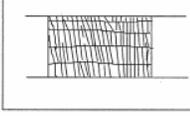
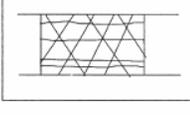
Singh et al. (1997) suggested the following expression for rock mass strength based on Q based on back analyses of underground coal mine tunnels:

$$\sigma_{cj} = 7 \cdot \gamma \cdot Q^{1/3} = 7 \cdot \gamma \cdot \left(e^{(RMR-44/9)} \right)^{1/3} \quad (5.11)$$

where: σ_{cj} = uniaxial compressive strength of jointed rock (MPa)

γ = unit weight of rock (g/cm^3)
 Q = Q rockmass quality
 RMR = Rock mass rating (1976)

Figure 5.5: Discontinuity Classes in Coal Seams; after Esterhuizen (1997).

Class		
	1	Massive coal , no visible cleats/joints longer than 5cm. Coal may be horizontally layered.
	2	Massive coal, irregular cleats/joints, less than 30cm long, less than 10 per metre.
	3	Blocky coal, regular cleats/joints in bands, typically less than 1m long with frequency of more than 10 per metre.
	4	Highly disturbed coal, continuous cleats/joints more than 1m long with frequency of more than 10 per metre.
	5	Jointed coal, smooth planar joints, usually inclined with infilling. Joints are not limited to individual coal layers, typically longer than 1m. One or more sets may be present.
	6	Coal contains major slips with undulating surfaces, continuous over several tens of metres; may extend into roof or floor of the seam.

Barton (2002) modified equation 5.11 and included the uniaxial compressive strength, σ_{ci} and suggested the following expression:

$$\sigma_{cj} = 5 \cdot \gamma \cdot \left(\frac{Q \cdot \sigma_{ci}}{100} \right)^{1/3} = \sigma_{cj} = 5 \cdot \gamma \cdot \left(\frac{(e^{(RMR-44/9)}) \cdot \sigma_{ci}}{100} \right)^{1/3} \quad (5.12)$$

where σ_{ci} = uniaxial compressive strength of intact rock (MPa)

The joint-reconciled UCS of No.3 seam coal suggested by Equations 5.11 and 5.12 is 6.5 and 2.5 MPa respectively. Equation 5.11 predicts a rock mass strength for No.3 seam coal

that supports a design value of 6.2 MPa as the rock mass strength for coal mine pillars at 4 South.

5.2.4 Time Effect

The duty life of a coal mine pillar is governed by the temporal stress-strain response of the pillar rockmass. The slow, time dependent deformation of a coal pillar under a constant load is termed creep. The stand-up time of a coal mine pillar can be increased by:

- increasing pillar stoutness (increase pillar area and decrease pillar height),
- reducing road entry widths,
- increasing pillar confinement (rib bolting, stowing), and
- increasing local roof stiffness and self-supporting capacity (passive and active roof support installations)

Taylor (2003), who performed back-analyses on 34 South African collapsed pillar cases, found a correlation between the duty life of each pillar and the pillar in-situ strength, w/h ratio, triaxial stress factor and average TAT load, governed by the following equation:

$$YTF = \frac{IPS^k \left(\frac{0.5P_w}{h} \right)}{\sigma_{avps}} \quad (5.13)$$

where: YTF = years to failure

IPS = in-situ pillar strength (MPa)

P_w = pillar width (m)

h = pillar height

σ_{avps} = average tributary stress acting on pillar (MPa)

k = triaxial stress factor

Equation 5.13 predicts the duty life of final coal panel pillars in the 4 South mine to be approximately 4 years at an average depth of cover of 60 meters; however it is known that these pillars have remained intact for over 12 years.

Scaling of coal pillars takes place to some degree which reduces the pillar width and, consequently, the safety factor. Van der Merwe (1993), investigated underground coal mine pillars in the Vaal basin of South Africa, and proposed a rough empirically derived index relating the rate of reduction pillar width to the mining height as shown in Equation 5.14

$$R = 0.015h^{3.7} \quad (5.14)$$

where R = rate of reduction in pillar width (m/year)

h = the mining height (m)

Van der Merwe suggested that pillar failure is preceded by scaling and occurs when the pillar safety factor is reduced to 0.3. Van der Merwe states that the scaling rate is possibly a function of several variables including climatic conditions, absence or presence of sidewall support, and chemical composition of coal and easily quantifiable variables including pillar stress, mining depth and mining height. Negligible pillar scaling is observed for coal mine pillars at 4 South with the exception of pillars intersected by major faults where pillar slough averages 0.3 to 0.6 meters in depth.

5.3 PILLAR DESIGN

Classic coal pillar design requires calculation of the pillar loading, pillar strength and safety factor. Empirical based methods, based upon empirical pillar strength formulas, use this design methodology. Software tools like ARMPS have augmented empirical pillar strength design by incorporating abutment loading and modifications to the pillar strength formula to account for rectangular pillars. The increased strength of rectangular pillars can also be addressed by calculating and using the square-equivalent dimensions of the pillar in design. The design of barrier pillars is facilitated with empirical rules for acceptable widths. All of the above is discussed in further detail in this chapter

Analytical and numerical codes based upon mechanical models of pillar behavior represent advanced tools for coal pillar design ultimately required to model coal pillar creep, and pillar to floor and roof interactions and other complex pillar stress/strain

behaviors. Mark (2006) reported on the value of numerical models for coal pillar design as stated by Gale (2005) with the following points:

1. Post-failure simulation of the “strain softening” process.
2. Simultaneous assessment of shear, tensile and bedding plane failure within the material, together with the effect of joints and structural weakness.
3. Adequate simulation of the material properties and stress distribution within the ground, and
4. Ability to simulate failure of strata above and below the pillars, and to simulate the correct stress path within the pillars.

Analytical and numerical design methods are discussed in greater detail in this chapter.

5.3.1 Empirical Pillar Design

Given empirical pillar strength formulas are an essential aspect of empirical pillar design, it is important in their application to understand the guidelines governing their use in design. Table 5.3 illustrates five popular empirical pillar strength formulas used in the United States and guidelines for acceptable width to height ratios and safety factors in their application.

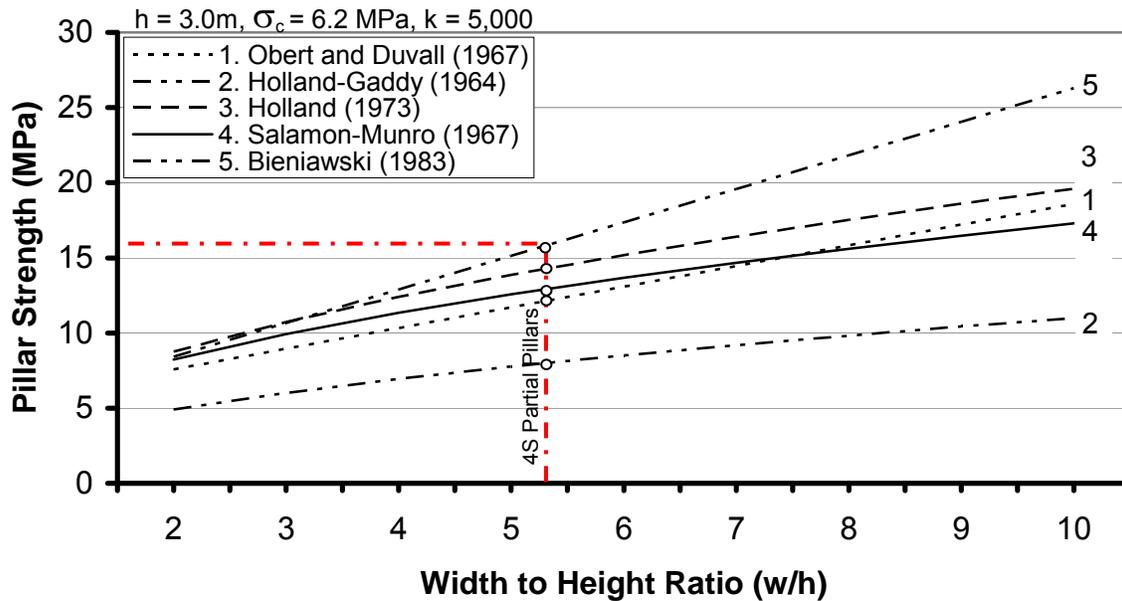
Table 5.3: Five Common US Pillar Strength Formulas with Recommended Range of w/h Ratio and Safety Factor.

Researcher	Formula	w/h ratio	Safety Factor	Comments
Obert and Duvall(1967)	$\sigma_p = \sigma_c \left[0.778 + 0.222 \left(\frac{w}{h} \right) \right]$	0.25 to 4.0	Short term: 2 Long term: 4 Conditions known: 1.5	Assuming gravity loading
Holland-Gaddy (1964)	$\sigma_p = \frac{k\sqrt{w}}{h}$	2 to 8	Range: 1.8 - 2.2 Recommend: 2.0	
Holland (1973)	$\sigma_p = \sigma_c \sqrt{\frac{w}{h}}$	-	Recommend: 2.0	
Salamon-Munro (1967)	$\sigma_p = 7.2 \frac{w^{0.46}}{h^{0.66}}$	up to 5	Range: 1.31 - 1.68 Recommend: 1.6	Applicable to South African conditions
Bieniawski (1983)	$\sigma_p = \sigma_c \left[0.64 + 0.36 \left(\frac{w}{h} \right) \right]$	up to 10	Short term: 1.5 Long term: 2.0 Recommend: 2.0	Based on large scale in-situ tests

Where: σ_p = pillar strength (MPa), σ_c = UCS of cubical coal specimen (MPa)
 w = pillar width (m) h = pillar height (m)

Figure 5.6 illustrates a comparison of the five empirical pillar strength formulas.

Figure 5.6: Five Common US Pillar Strength Formulas with Recommended Range of w/h Ratio and Safety Factor.



The Bieniawski (1983) pillar strength formula is widely accepted based on its representation of the rapid increase in pillar strength realized at width to height ratios greater than five. Specifically, coal pillars with width to height ratios greater than 10 are known to be indestructible and no current theory indicates this. The Holland-Gaddy (1964) formula provides the most conservative predictions of pillar strength. The Holland-Gaddy formula predicts approximately half the pillar strength of the other formulas.

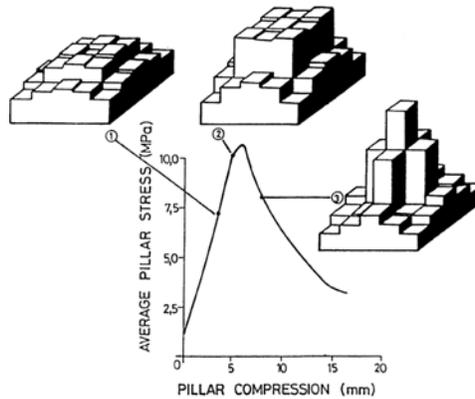
In-situ tests performed by Wagner (1974) showed that the failure of a pillar is a gradual process with failure initiating at the pillar skin and progressing towards the pillar core. Figure 5.7 illustrates Wagner's model of pillar load distribution. The confined core of the pillar accepts a greater proportion of the pillar load, owing to its increased stiffness. Wilson reported that the centre portion of a pillar was capable of withstanding extremely high stresses, even when the pillar has been loaded beyond its ultimate strength.

5.3.2 Rectangular Pillars

In underground coal mines, both square and rectangular pillars are used. The majority of empirical coal pillar design equations are intended for square pillars. The use of

rectangular pillars requires either equations developed specifically for their use, or alternatively, the conversion of square pillars to rectangular pillars of equivalent strength.

Figure 5.7: Stress in Coal Pillar Versus Pillar Compression; after Wagner (1980).



Wagner (1974) recommended the first approach to consider rectangular pillars, using an equivalent width, w_e , in the equation for strength of a pillar with cross section area A and circumference c , calculated as follows:

$$w_e = \frac{4A}{c} \quad (5.15)$$

This equation indicates that the maximum equivalent width for a pillar with infinite length is twice the physical width of the pillar.

Salamon and Oravec (1976) suggested a different conversion factor:

$$w_e = \sqrt{wl} \quad (5.16)$$

Investigation into the relative influence of the pillar length on pillar strength reports that pillar strength is primarily dependent on pillar width, not pillar length. According to equation 5.12 the effective width of an infinitely long barrier pillar is twice its width.

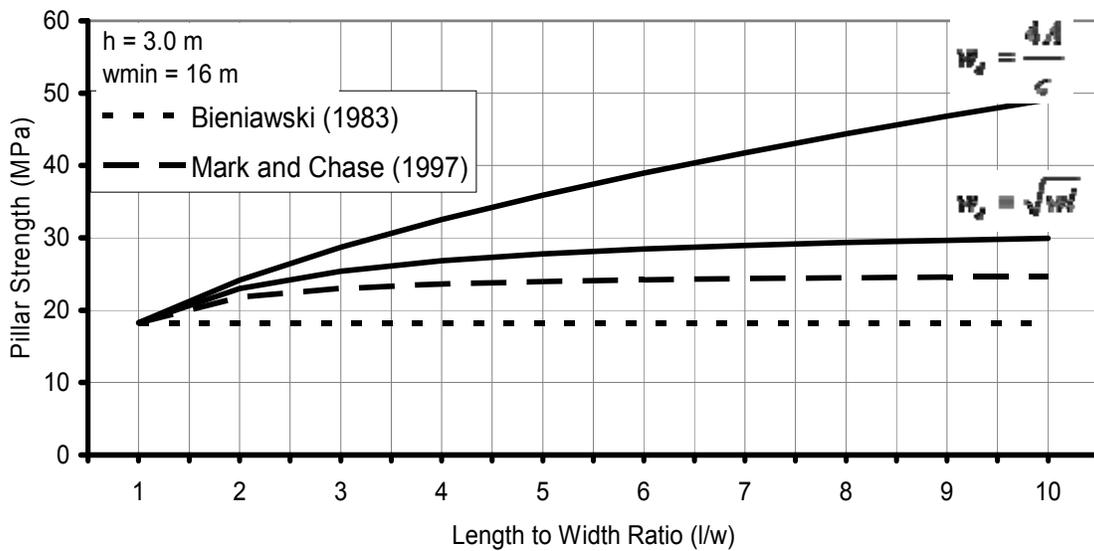
Mark and Chase (1997) developed the Mark-Bieniawski formula to account for their finding that the strength of rectangular pillars exhibits a dependence on the width-to-height and width-to-length ratios. The Mark-Bieniawski formula is used in the ARMPS program to calculate the pillar strength values. Equation 5.14 shows this formula:

$$S_p = S_1 \left[0.64 + 0.54 \left(\frac{w}{h} \right) - 0.18 \left(\frac{w^2}{l \times h} \right) \right] \quad (5.17)$$

where: l = pillar length (m)

Figure 5.8 illustrates the relationship between pillar strength and length to width ratio (l/w) using equation 5.15 and 5.16 with the Bieniawski (1983) pillar strength formula and the Mark and Chase (1997) formula. Equation 5.15 predicts the highest pillar strength while the Bieniawski (1983) “square” pillar formula predicts the lowest strength. The difference between the formulas increases with increasing length to width ratio. The Mark and Chase formula predicts about 16% higher pillar strength in comparison to the Bieniawski formula at a length to width ratio of two.

Figure 5.8: Effect of Length to Width Ratio of the Strength of Coal Pillars.



5.3.3 Barrier Pillars

Barrier pillars are generally designed using rule-of-thumb type of formulas based on experience. Two of the more common formulas are the Mine Inspector’s formula and the British Coal Operator’s formula.

The Mine Inspector’s Formula is (Holland, 1973):

$$W_{bp} = 20 + 4h + 0.1H \quad (5.18)$$

where: W_{bp} = width of barrier pillar (ft)
 h = seam thickness (ft)
 H = thickness of overburden (ft)

The British Coal Operator's formula disregards mining height if the determination of the barrier pillar width as shown below:

$$W_{bp} = 45 + 0.1H \quad (5.19)$$

Equation 5.18 and 5.19 recommend barrier pillar widths ranging from 20 to 36 meters. At Barrier pillars at the 4 South mine are currently sized at 30 to 36 meter width with provision for slab cuts on either side of up 8m depth into the pillar. A reduction of the barrier pillar width to 24 meters for the 4 South mine is recommended where partial pillar extraction design is shown to provide longer term support to the immediate roof and the risk of caving is low. Slab cuts would have to be reviewed on a case by case basis, taking into account the risks of water and gas transmission through the barrier pillar, as well as the presence of major structure.

5.3.4 Analytical and Numerical Pillar Design

Analytical methods for pillar design are employed to understand the more complex rock mechanical behavior of pillars. In their application, good estimates of roof, floor and coal rock mechanical properties are required. Yield pillar design for longwall and other full-extraction mining methods in soft rock typically employ analytical methods.

Wilson (1982) proposed the “confined core” model proposed by Wilson (1983), which uses the Mohr-Coulomb failure criterion applied to coal to derive an expression for the vertical stress gradient within the yield zone σ_v , based on the assumption that confinement develops as a continuous function of the distance (x) from the nearest rib:

$$\sigma_v = kp' \left(\frac{2x}{h} + 1 \right)^{k-1} \quad (5.20)$$

where $k =$ triaxial strength factor $= \frac{1 + \sin \phi}{1 - \sin \phi}$,

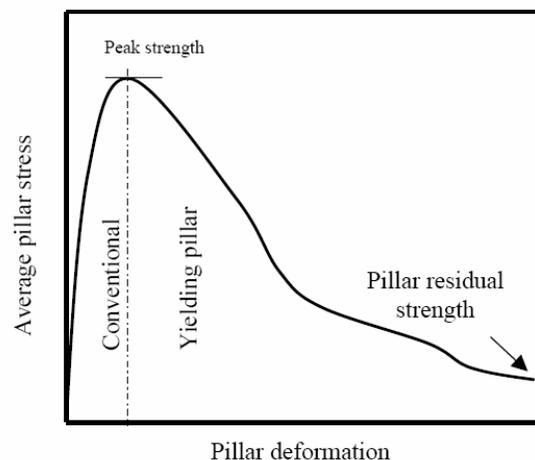
and $\phi =$ internal friction angle

$p' =$ residual strength of the unconfined, failed coal at the pillar edge

Wilson recommends values of $p' = 14$ psi and $\phi = 30^\circ$ to 37° .

Salamon (1992) also derived analytical methods based on the confined core model which account for the interface properties of the coal and rock. Both the Wilson (1983) and Salamon (1992) models consider the strain-softening behavior of coal mine pillars, illustrated in Figure 5.9. Detailed analysis of post-peak strength behavior of coal mine pillars is also achieved with numerical methods.

Figure 5.9: Stress-Deformation Curve for Conventional and Yield Pillars; after Schissler (2002).



The analysis of stresses and displacements around underground excavations is an important tool in rock mechanical analysis and design of underground structures. Numerical modeling techniques like the Boundary Element Method (BEM), Displacement Discontinuity Methods (DDM) and Finite Element Methods (FEM) have evolved and are finding increasing application in rock mechanical analysis in coal mining.

The application of numerical modeling for coal mine pillar design has been advanced and popularized with the availability of powerful and inexpensive personal computers and user-friendly interface design for software packages running numerical codes.

Mark (2006) reported on the value of numerical models for coal pillar design as stated by Gale (2005) with the following points:

1. Post-failure simulation of the “strain softening” process.
2. Simultaneous assessment of shear, tensile and bedding plane failure within the material, together with the effect of joints and structural weakness.
3. Adequate simulation of the material properties and stress distribution within the ground, and
4. Ability to simulate failure of strata above and below the pillars, and to simulate the correct stress path within the pillars.

Higher stress predictions are associated with two-dimensional models as three-dimensional models consider stress relief effects out of plane. 2D models assume plane strain conditions for elastic solutions and require, as a rule, a pillar length-to-width ratio of about 3 or more to be acceptable Hartman (1992). The overestimation of the resulting stress field around an opening, results in a conservative estimate of induced stresses. 2D models are good when rapid analysis is required, and often precedes the use of 3D models.

Two software packages have excellent application for coal mine pillar design. *Phase2*, a two-dimensional elasto-plastic finite element program for calculating stresses and displacements around underground openings, is versatile software with the ability to model staged excavations, ground support and jointing explicitly in the model.

Examine^{TAB} is a pseudo 3D displacement discontinuity program for calculating elastic stresses and displacements within tabular ore bodies. It assumes linear elasticity and that all excavations lie on the same plane and are of the same height.

5.3.5 Analysis of Retreat Pillar Mining Stability (ARMPS)

ARMPS is a National Institute for Occupational Health and Safety (NIOSH) computer program for use in the design of pillars for room-and-pillar retreat mining developed by the US Bureau of Mines (Mark, et al., 1995). ARMPS calculates a stability factor for the

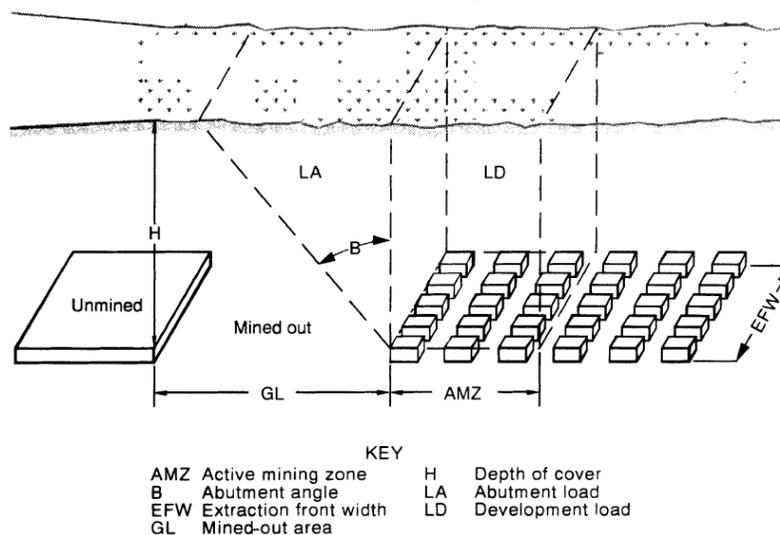
Active Mining Zone (AMZ) which is the product of the estimated load bearing capacity of the pillars in the AMZ divided by the estimated load on those pillars. The pillar load is the sum of TAT development load and abutment load. The calculation and theory governing abutment loads can be referenced in section 5.1. Figure 5.10 graphically illustrates the ARMPS model of the AMZ and abutment load.

ARMPS has several advantages over other methods of pillar design analysis, some of which have been touched upon in previous sections:

1. It considers increased load bearing capacity of rectangular pillars over square pillars of same width,
2. it allows for an analysis of the stability of pillars in the Active Mining Zone (AMZ) during development, retreat and with gobs one or both sides, and
3. it considers the effect of depth on abutment loading.

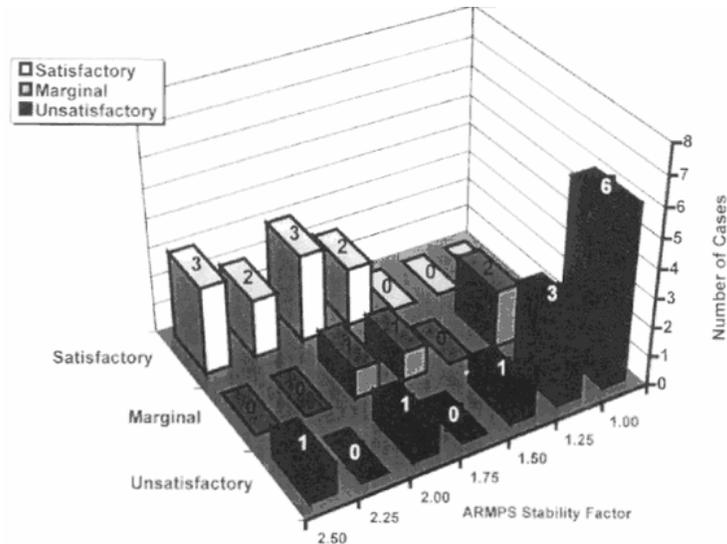
ARMPS is a very flexible method of analysis. The software allows the user to input all of the major parameters relating to the mining layout including the exteny and number of adjacent gobs areas around the active mining zone. The simplicity of its use and widespread application in the underground coal mines of the United States make it a very useful tool in pillar design, particularly when retreat pillar mining is practiced.

Figure 5.10: Schematic of ARMPS Active Mining Zone; after Mark (1995).



Mark (1999) recommends a minimum pillar stability factor equal to or greater than 1.5 at overburden depths up to 200 meters. This recommendation is corroborated in Figure 5.11, which shows case histories for sandstone roof from a mining complex in southern West Virginia, USA.

Figure 5.11: ARMPS Case Histories for Sandstone Roof in a Southern West Virginia Mining Complex; after Mark (2008)



5.4 4 SOUTH MINE PILLAR DESIGN RECOMMENDATIONS

The consideration of partial and non-caving methods and irregular pillar layouts for the 4 South mine requires the use of a customized pillar design method to develop a safe pillar layout. The adoption of full retreat pillar mining under the massive sandstone roof at 4 South is confounded by the poor caving properties of the sandstone roof. The primary pillar types to be assigned design guidelines can be referred to in Figure 2.4.

Tributary area theory is recommended for quick determination of pillar loads using equation 5.3. The extraction ratio can be determined using AutoCAD or similar CAD program. For design, Examine^{TAB} is recommended to determine elastic normal pillar stresses and total roof displacements. The Bienawski (1983) pillar strength formula will be used for pillar strength calculations. An in-situ coal strength of 6.2 MPa for No.3 seam coal is recommended for use in pillar design. Design guidelines for pillars at 4 South mine are discussed in this section.

5.4.1 Panel Pillars

Panel pillars are required to support the overburden during development and delay or minimize the risk of roof caving during pillar extraction. Current panel pillars are sized at 16 x 30 meters. As a recommended design guideline, panel pillars at the 4 South mine should:

1. have a minimum safety factor of 1.5,
2. have a minimum width to height ratio of 4, and
3. be reviewed for signs of instability due to the presence of structure or overstress during depillar mining.

5.4.2 Final Coal Pillars

Final coal pillars are what remain after pillar extraction has been completed as per design guidelines. Final coal pillars are currently sized to have a minimum effective diameter of 16 meters. As a recommended design guideline, final coal pillars at the 4 South mine should:

1. have a minimum safety factor of 1.5,
2. have a minimum width to height ratio of 4, and
3. be reviewed during depillar mining for modifications in cut sequencing to mitigate the daylighting of geologic structure in the roof.

5.4.3 Barrier Pillars

Barrier pillars are used to separate mining panels and shield them from stress transfer from adjacent mined out panels. Barrier pillars also serve to control water and gas transfer from mined out panels to adjacent developing panels. Retreat pillar mining is not being considered, therefore the importance of and requirement levels for stress bearing capacity and shielding in barrier pillar design is minimal. As a recommended design guideline, barrier pillars at the 4 South mine should :

1. have a minimum safety factor of 2,
2. have a minimum width to height ratio of 6,
3. be reviewed during depillar mining for modifications in cut sequencing to mitigate the daylighting of geologic structure in the roof.

5.4.4 Stump Pillars

Stump pillars are used to indicate roof sag and promote intersection stability during pillar extraction. In the 4 South mine, stump pillars or point pillars are the small wedges of coal that remain, in addition to the final coal pillars, of panel pillars after planned extraction cuts into the pillars have been completed. As a recommended design guideline, stump pillars at the 4 South mine should:

1. have a minimum edge length of 2.5 meters,
2. be reviewed during depillar mining for modifications in cut sequencing to mitigate the daylighting of geologic structure in the roof.

5.5 CONCLUSIONS AND RECOMMENDATIONS

The important variables affecting the design of coal mine pillars have been discussed and presented in this chapter. Tributary area theory (TAT) is recommended for evaluating pillar loads where irregular pillar layouts are designed and extraction ratios do not exceed 65%. The Bieniawski (1983) pillar strength formula is recommended for estimating pillar strength as part of empirical design. Consideration of discontinuity effects and rectangular pillar correction in pillar strength calculations is good practice. When regular pillar layouts are designed for, ARMPS should be used to design pillars using the empirical guidelines for acceptable stability factors. Examine^{TAB} is also recommended to determine pillar loading for irregular pillar layouts, particularly at extraction ratios higher than 65%. In the event that pillar yielding is indicated in the pillar stability analysis, the Rocscience Software, Phase2 should be employed to evaluate the pillar stress/strain response and ground support capacity.

The discussion in this chapter supports the use of empirical and numerical design methods for pillar design of 4 South mine. Specifically, tributary area theory and the Bieniawski (1983) pillar strength formula will be used in conjunction with Examine^{TAB}. A value of 6.2 MPa for in-situ coal strength is recommended for design. No correction factors to the in-situ coal strength are required for analysis.

CHAPTER 6 EXCAVATION STABILITY

6.0 INTRODUCTION

The design of safe excavation spans promotes safety for underground personnel, mining equipment, services and infrastructure and increases operational efficiency. Excavation spans strongly governs roof behavior, and consequently the design of safe roof spans is also a cornerstone of effective ground support design. Furthermore, the maximum roof span that a coal mine roof can support strongly governs the cavability of the mine roof and the amenability of full-extraction or partial caving mining methods. Massive sandstone roof in underground coal mining environments is widely known to challenge rock engineers in designing safe excavation spans and predicting roof behavior during pillaring. Specifically, full pillar extraction is difficult to safely implement with massive sandstone roof conditions due to the large spans required to initiate caving, the unpredictability of caving processes, and the potential for large windblasts in the case of massive roof failures.

Section one intends to investigate the cavability of the massive sandstone from available empirical and classification methods drawn primarily from empirical studies in the coalfields of India where massive sandstone is the dominant geology of coal mine roof. Section two details the prediction of the critical span of coal mine roof using empirical and analytical studies. Analytical methods for estimating the critical span of mine roof in this discussion are based on beam and plate theory. Section three discusses mine roof convergence monitoring and introduces a novel mine roof convergence instrument developed for underground coal mines. The results of an in-situ test in the Quinsam Coal 2 North mine with this instrument will be detailed. This chapter will be concluded with a summary of the chief conclusions and recommendations.

6.1 CAVABILITY

Caving is an important aspect of strata control for partial and full-extraction mining methods. Controlled caving can be associated with overall mine stability, as it relieves

stresses on abutments and local barrier pillars. Cavability is usually expressed in terms of a pressure arch, a circular, parabolic or rectangular zone in the rock above an opening in two dimensions that has low radial compression stress and where the rock sags and ultimately collapses under self weight at a critical unsupported span Hartman (1992).

The characteristics of discontinuities in the rock greatly influence the cavability of a rock mass, which should be reflected in the rock mass classification method employed. Hartman (1992) reports that the onset of fracturing for a given roof span has been considered by Obert and Duvall (1967) and Wright (1973) but with little success.

Early approaches to characterize the cavability of roof strata were based on arching and bulking theory as detailed by Terzaghi (1946) and echoed by Farmer (1985). The theory states that for caved strata to bulk sufficiently to support upper layers, the span must facilitate the process of caving “choking” off. The analysis requires the roof be classified according to Terzaghi’s classification system in Table 6.1 below.

Table 6.1: Relationship between Cavability and Rock Classification Systems; after Terzaghi (1946) Deere (1963) as reported by Hartman (1992).

Terzaghi classification	Rock behavior and possible causes of instability	Approximate stand-up time	Deere classification	Rock breakage height, m
1 Hard and intact	Stable excavation unless induced stress greater than rock strength	Many years	Excellent: RQD 90-100	0
2 Hard stratified and schistose	Bed separation with time; surface spalling Immediately stable.	1 year	Good: RQD 75-90	0.25 B
3 Massive, moderately jointed	Detachment of blocks, progressively releasing further blocks Immediately stable.	1 week		
4 Moderately blocky and seamy	Detachment of blocks, progressively releasing further blocks Immediately fairly stable.	1 week	Fair: RQD 50-75	0.7 B
5 Very blocky and seamy and shattered	Surface dilation of rock due to rapid block detachment Local roof falls during excavation, Rapid peripheral dilation	1 day	Poor: RQD 25-50	1.5 B
6 Completely crushed	Immediate collapse	1 hour		
7 Sand and gravel	Rapid yielding and deformation	0	Very Poor: RQD 0-25	2 B
8 Squeezing: moderate depth			Squeezing and swelling ground	

The following equation is proposed to estimate the span required to support the caving roof:

$$B = \frac{M}{(\beta - 1)x} \quad (6.1)$$

where: B = roof span (m)

M = excavated thickness (m)

β = bulking factor

x = rock breakage height coefficient from Table 6.1

Wagner et al. (1997) reported on the strength and bulking properties of rock formations in South African Coalfields as shown in Table 6.2 below.

Table 6.2: Typical Strength and Bulking Properties of Rock Formations in South African Coalfields; after Wagner et al. (1997).

Rock type		Uniaxial compressive strength MPa	Flexural strength MPa	Bulking factor, k_i
Sandstone	Fine	70–120	5	1,3–1,5
	Medium	50–70	3	
	Coarse	30–50	2	
Shale		40–80	2	1,1–1,2
Dolerite		250–390	10	?
Coal		15–40	1,5	1,3

For the 4 South sandstone roof, if $\beta = 1.4$ and $x = 0.5$ and $M = 3.5$ meters, a minimum span of 17.5 meters is suggested provide enough rockmass bulking to terminate caving.

Singh (2005) placed emphasis on the fragmentation of caved rock as criteria for cavability, stating that the caved rock must be suitably fragmented so that it fills the gob solid. Large blocks in the gob will not result in the necessary consolidation to support the roof.

Singh et al. (1999) reported on the work of Sarkar et al (1988) who, on the basis of statistical analysis of the data from a large number of case studies, developed an empirical formula for the cavability index, I , to characterize the behavior of the immediate roof strata of Indian coal mines. Considering compressive strength as a

parameter of strength and average length of core as a parameter of massiveness, the cavability index is defined as:

$$I = \frac{C \cdot L^n t^{0.5}}{5} \quad (6.2)$$

where: I = cavability index

C = uniaxial compressive strength (kg/cm²)

n = 1.2 in the case of uniformly massive rocks with a weighted average of RQD of 80% and above; otherwise = 1.

L = average length of core (cm)

t = thickness of bed (m)

A classification for cavability was developed based on equation 6.2 and is shown below in Table 6.3:

Table 6.3: Classification of Roof Cavability based on the Cavability Index; after Singh et al. (1999).

Category of roof	Range of cavability index
1 Easily cavable	< 2,000
2 Moderately cavable	2,001 - 5,000
3 Cavable with difficulty	5,001 - 10,000
4 Cavable with substantial difficulty	10,001 - 14,000
5 Cavable with extreme difficulty	>14000

Equation 6.2 classifies the 4 South mine sandstone as cavable with substantial difficulty. The Singh (1999) classification system does not assign a risk factor which becomes important when implementing caving methods with a coal mine roof capable of bridging very large spans. Singh (2005) reported on the relationship between the UCS and cavability of roof, stating that roof with a compressive strength of less than 50 MPa is considered to be cavable. Caving of the 4 South mine roof would be possible based on this guideline.

Zamarski as reported Siska (1972) classified the cavability of roof rock from the Ostrava Karvina Coalfield on the basis of the length of unbroken cores (K_j) obtained by drilling in the roof. Unbroken core lengths greater than 10.5 cm indicate poor cavability.

Bieniawski (1982) reported on the cavability classification system of Unrug and Szwiliski (1983) used in coal mines in Poland. This classification, depicted in Table 6.4

is based on assessing the Roof Quality Index (RQI) which includes a calculation of the in-situ compressive strength of the rock strata.

Table 6.4: Cavability Classification System; after Unrug et al (1983).

Roof Class		Roof Quality Index ^b	Allowable Area of Roof Exposure (m ²)
I	Very weak	$L < 18$	1
II	Little stable	$18 < L < 35$	1-2
III	Medium stable	$35 < L < 60$	2-5
IV	Stable	$60 < L < 130$	5-8
V	Very Strong	$L > 130$	>8

^b Roof quality index $L = 0.016\sigma_M d$

$$L = 0.016\sigma_M d \quad (6.3)$$

where: σ_M = in-situ compressive strength of rock strata (kg/cm²) = $\sigma_c K_1 K_2 K_3$

σ_c = uniaxial compressive strength,

K_1 = 0.4 (coefficient of strength utilization)

K_2 = 0.7 (coefficient of creep)

K_3 = 50% (coefficient of moisture content)

d = mean thickness of roof strata layers (cm)

The Unrug et al. (1983) classification system applied to the 4 South mine categorizes the sandstone roof as very strong with an allowable area of roof exposure greater than 8 m². Poor cavability is clearly indicated.

Singh (2005) reports that solid stowing is the best solution for gob control in bump-prone seismic conditions under a strong roof with poor cavability. Stowing involves packing the gob solid via one or a combination of the following packing methods: manual, gravity stowing, mechanical stowing, pneumatic stowing or hydraulic stowing. Stowing distributes roof stresses, reduces convergence and minimizes the risk of spontaneous combustion. Stowing is capital and manpower intensive and rarely employed at underground coal mines, except in the most extreme circumstances.

6.2 CRITICAL SPAN

The critical span of a coal mine roof is an important and often overlooked parameter of coal mine design. The critical roof span directly impacts the selection of mining method, pillar sizing and spacing and ground support requirements among other things. Advanced numerical methods now provide rock engineers with powerful tools to evaluate the limits of underground span development, stand-up time and failure mechanics. While numerical methods are highly suited to evaluate critical spans for coal mine roof, they require expertise not readily available and affordable to most coal mines like Quinsam Coal mine. Empirical relationships are very useful to smaller-scale operators in this respect as a tool for mine design.

Massive sandstone roof is able to span large distances. Lind (2002) stated that underground mines with massive roof strata in New South Wales developed spans between 60 to 70 meters before first caving occurs. He added that massive energy releases causing windblasts are common occurrences after the first major cave of the massive strata and that consequently, partial extraction of pillars is considered to avoid the potential roof fall risks associated with full caving practices.

6.2.1 Empirical Span Prediction

Lang (1994) analyzed 172 case histories to relate the RMR_{76} to the unsupported critical span at two hard-rock cut and fill mines in the Canadian Shield. The original database was based on RMR records between 60 and 80 percent. Ouchi et al. (2004) updated this database with 120 case histories to cover a wider range of rock mass quality. Particularly the weak rockmass range between 30 and 50 percent. The critical span is defined as the diameter of the largest circle that can be drawn within the boundaries of the exposed back as viewed in plan. This exposed span is then related to the prevailing rock mass of the immediate back to arrive at a stability condition. Figure 6.1 shows the updated span curve.

The critical span curve does not include sufficient data from underground coal mines to be regarded as a recommended tool for assessing the critical span of coal mine roof.

Figure 6.1 predicts safe unsupported spans of 15 to 25 meters for the 4 South mine roof.

Bieniawski (1989) related the rock quality to stand-up time and span in as shown in Figure 6.2. The predicted stable unsupported span for the 4 South mine roof is approximately 20 meters with a stand-up time of about 1 month. Both predictions are likely conservative based on known unsupported spans of 24 meters being developed safely in the 4 South mine.

Figure 6.1: Updated Span Curve; after Ouchi et al (2004).

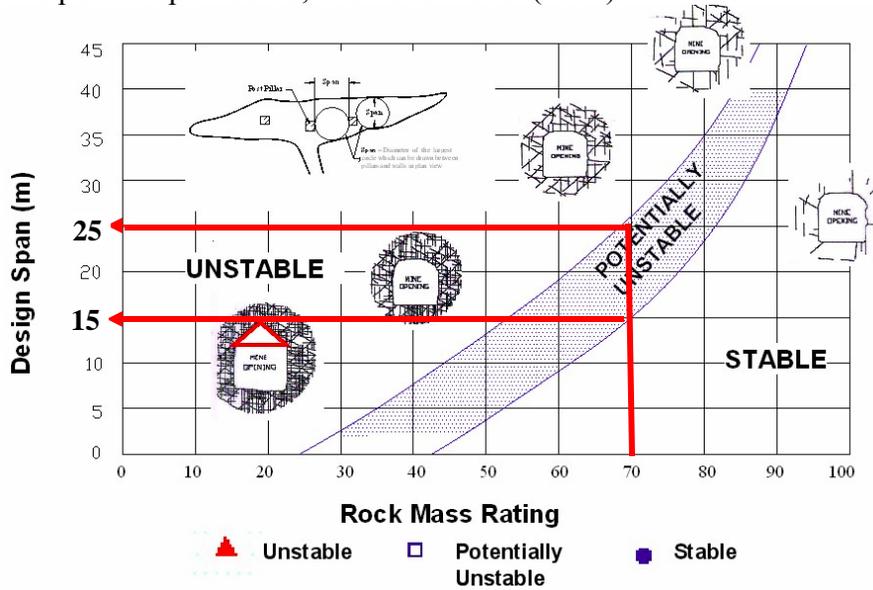
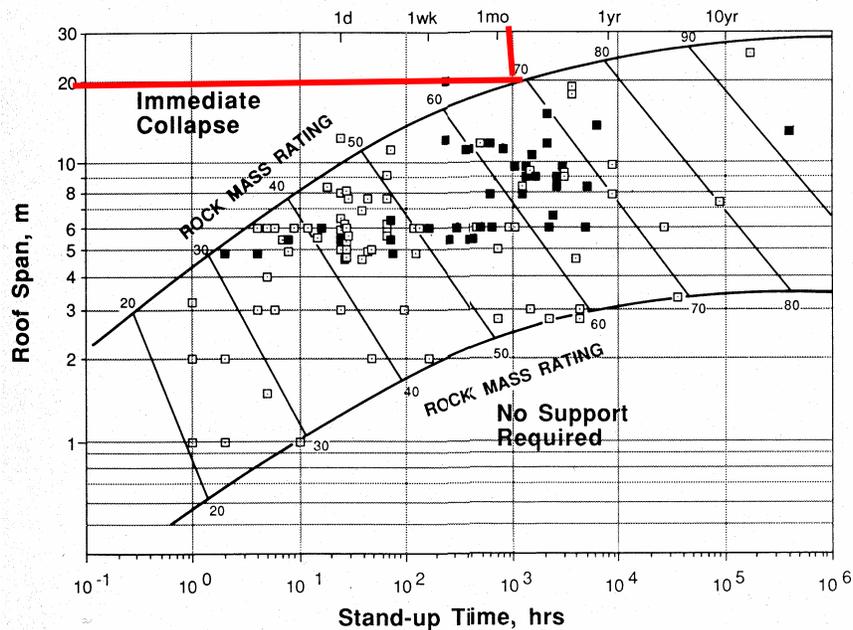


Figure 6.2: Relationship between Stand-up time, span and RMR classification; after Bieniawski (1989).



Barton et al (1974) related the maximum span, to the NGI-Q rock quality as depicted in equation 6.4.

$$S_c = ESR \cdot 2(Q)^{0.4} \quad (6.4)$$

where: Q = NGI-Q rock quality $\exp((RMR-44)/9)$

ESR = excavation support ratio = 3-5 for temporary mine openings

Carter (2000) related the critical span of mine roof to Q rating as follows:

$$S_c = 3.3(Q)^{0.43} \cdot \sinh^{0.0016}(Q) \quad (6.5)$$

where: S_c = critical Span (m)

Q = NGI-Q rock quality = $\exp((RMR-44)/9)$

The hyperbolic sine term in the above expression was introduced to account for the non-linear trend to increasing stability recognized for very good quality rock masses

Equation 6.4 predicts a stable unsupported span for the 4 South massive sandstone roof between 19 and 32 meters. Equation 6.5 predicts an unsupported span of about 15 meters.

6.2.2 Beam and Plate Theory Span Prediction

A simple means of determining the maximum span using beam theory is to assume clamped end conditions. Failure by flexure will occur when the maximum tensile stress induced by flexure of the clamped beam exceeds the tensile strength of the sandstone.

Whittaker et al. (1989) referred to the clamped beam equation below, to estimate of the self-supporting spans of different types of rock strata:

$$\sigma_t = \frac{\rho g L^2}{2t_s} \quad (6.6)$$

where, σ_t = maximum tensile stress (N/m²)

ρ = density of the rock strata (kg/m³)

g = gravitational acceleration (9.81 m/s²)

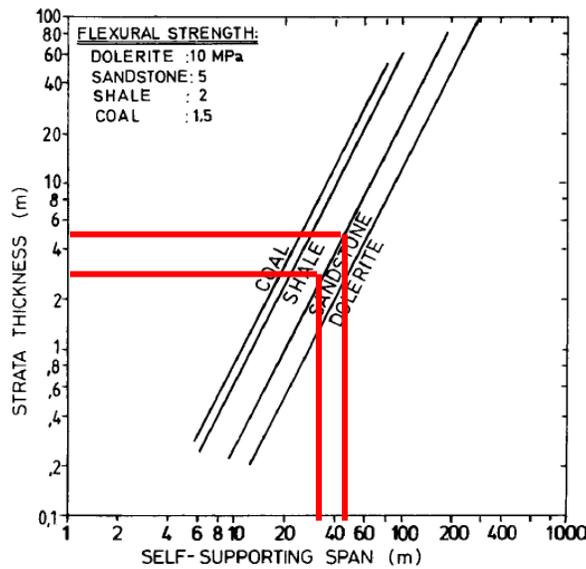
L = Unsupported span (m)

t_s = thickness of sandstone beam (m)

Figure 6.3 depicts the relation between the roof strata thickness and the self-supporting span for common rock formations found in South African coalfields as predicted by equation 6.6. For sandstone, the results are based on a flexural strength of 5 MPa. The predicted self-supporting span of the 4 South mine roof is 35 to 45 meters.

Equation 6.6 is valid for the situation where there is no horizontal stress and the idealized beam is unjointed. When jointing is present and continuous through the beam thickness, the beam behaves as a cantilever, similar to longwall roof behind the cutter head.

Figure 6.3: Typical Self-Supporting Spans for the Rock Formations Found in South African Coalfields; after Wagner et al (1991).



The following equation can be used to determine the unsupported span of a cantilever beam:

$$\sigma_t = \frac{3\rho g L^2}{t_s} \quad (6.7)$$

The results indicate that a cantilever roof has 1/6 the tensile strength of an unjointed clamped beam roof. The predicted unsupported span for the 4 South sandstone roof ranges from 14 to 18 meters. Figure 6.4 illustrates the model of both roof types.

Diedrichs et al (1999) reported normalized stability charts based on parametric modeling of voussoir beam and plate theory equations for prediction of maximum unsupported span. Lines for the critical crushing and snap-thru limits are provided for the intact rock UCS and rockmass modulus normalized by an effective specific gravity term, $S.G.' :$

$$S.G.' = S.G. \cos \alpha \quad (6.8)$$

where: $S.G.$ = specific gravity (for flat dipping beams $S.G.' = S.G.$)
 α' = dip of rock beam

Figure 6.4: Clamped Beam and Cantilever Beam Model.

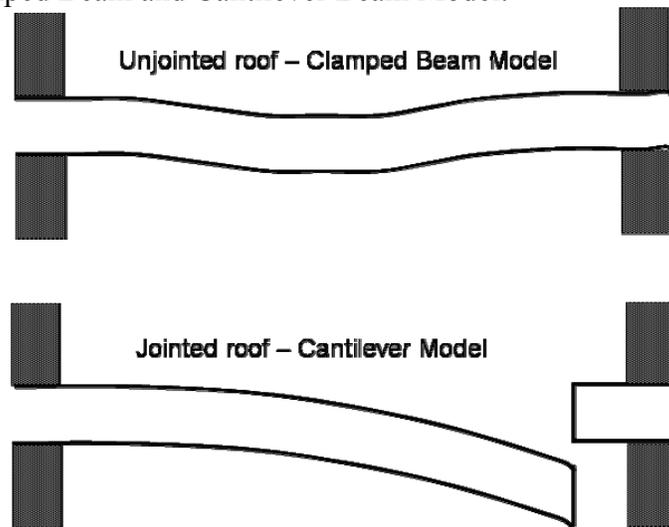


Figure 6.5 shows curves developed by Diedrichs (1999) for jointed rock beams and jointed rock plates. The predicted unsupported span for the 4 South sandstone roof based on the snap-thru limit (the limiting failure condition) is 45 meters for tunnel span and 72 meters for square span.

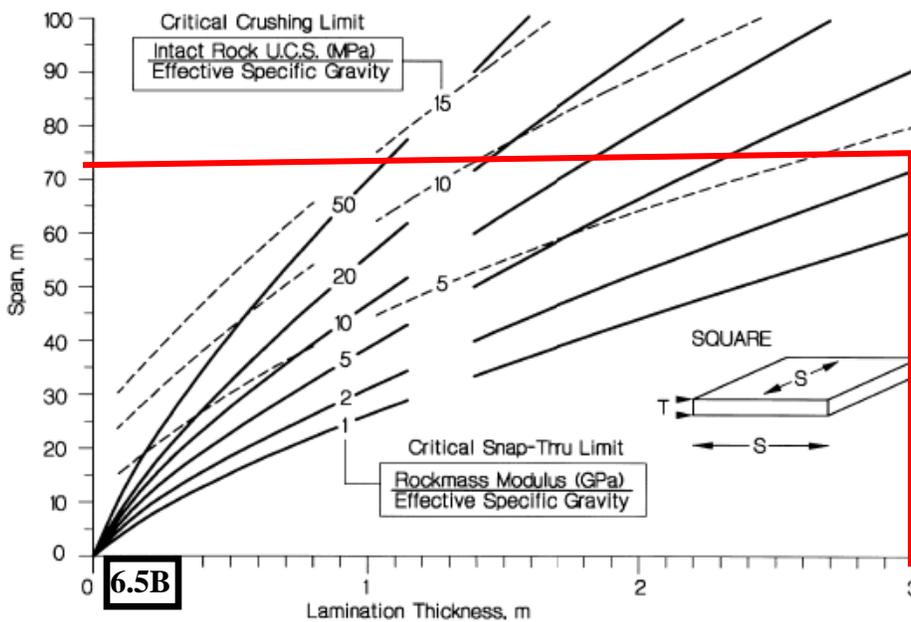
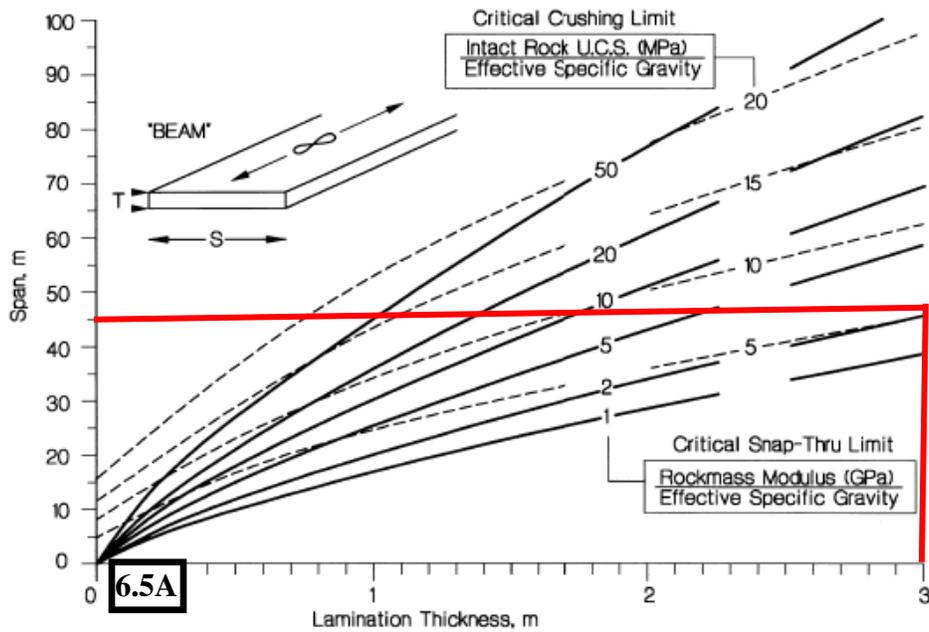
6.2.3 Mine Span Stability – 4 South Mine

The largest unsupported roof spans were created in the north in-line pillar panel off No.1 Mains from 6 to 14 cross cut. The average span created in this area was 23 meters. The largest single roof span created was 40 meters at the north extent of cross cuts 7A to 8A at an overburden depth of 40 meters.

This area was not observed to have caved during pillaring operations. Reports of non-specific caving events in this mining section have been recorded (Morely et al., 2008) in

the presence of geologic roof structure. The majority of intersection spans in the north in-line pillar panel were stable throughout pillar extraction. A minimum critical span of 23 meters is assumed from this information. No sandstone roof failures below a span of 23 meters have been observed in the mine. A value of 40 meters is suggested for massive sandstone roof at the 4 South mine.

Figure 6.5: Stability Guidelines for Jointed Rock Beams (6.5A) and Jointed Rock Plates (6.5B); after Diedrichs et al (1999).



6.3 CAVING OF MASSIVE SANDSTONE ROOF

Massive sandstone roof has been shown to bridge large spans. Massive sandstone roof has a high compressive strength and low bending strength under load and with continued loading, it may accumulate energy and rupture suddenly and violently causing air blast and/or bumps. Figure 6.6 shows a conceptual model of the caving of massive sandstone roof, using 4 South mine as an example.

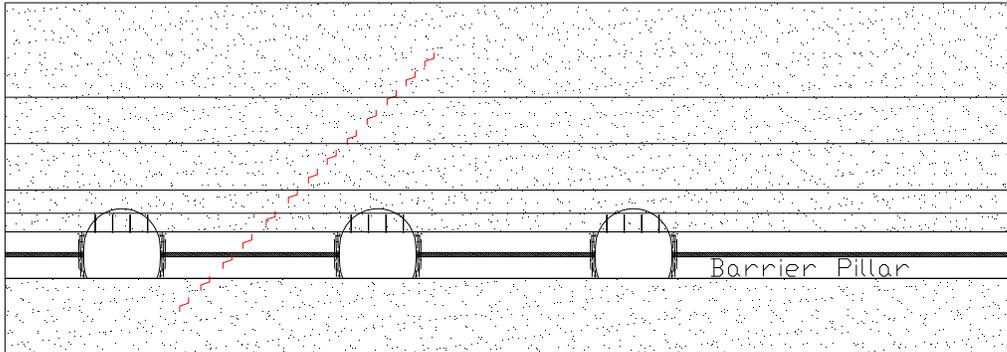
In step one, 6 meter roadways are developed on 22 meter centers with 1.5 meter rock bolts on a 1.5 meter spacing. Zones of relaxation are shown above the excavations in response to mining. Additionally, a fault is shown dipping over the workings. Bedding planes on two to three meters thick are shown which correspond to depositional cycles.

In step two, a pillar has been mined and the sandstone roof bridge begins failing. A zone of relaxation extending one-third the span into the roof has formed and the sandstone beds within this zone begin to sag. Tensile cracks begin to form at the edge of the immediate sandstone roof and in the center where tensile strains are greatest and bending moments are realized. Induced mining stresses concentrate in the barrier pillar and adjacent pillar. The presence of the fault is shown to distort the stress vector acting on the mine pillar.

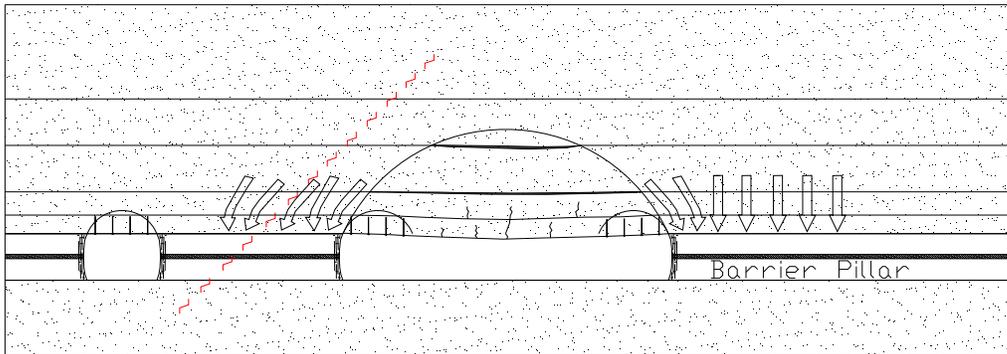
In step three, tensile cracks in the immediate roof compromise the integrity of the sandstone beds and lead to failure. Large blocks rotate out and create a cantilever beam anchored along the rockmass adjacent to the fault. A zone of compressive failure occurs at the fulcrum of the sandstone beam and tensile cracks continue to develop within the sandstone roof beams.

In step four, the cantilever roof beam fails violently followed by the remaining sandstone roof within the zone of relaxation. A stable roof arch is created and the broken roof rock augments the stability of the mine pillars by providing confinement to the rib. A large scale fault like the one illustrated can significantly diminish the stability of the immediate roof and lead to continued caving. As mentioned earlier, large scale roof failures can lead

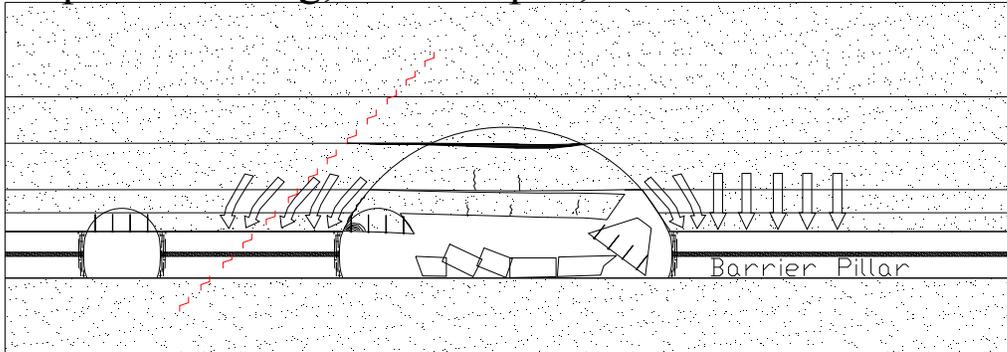
Figure 6.6: Conceptual Model of the Caving of Massive Sandstone Roof



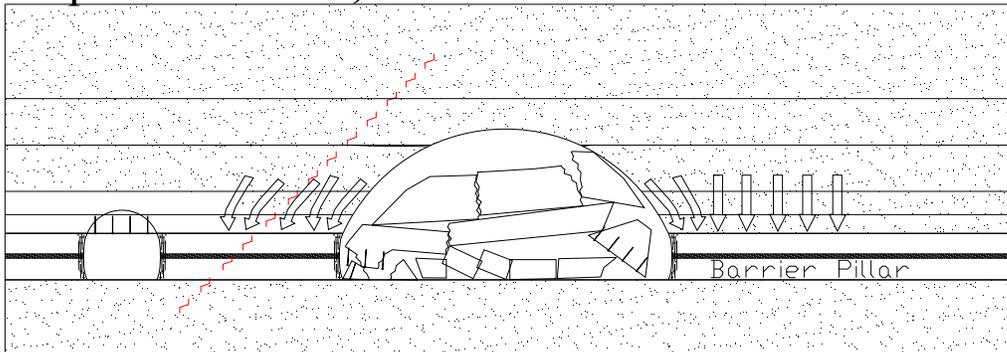
Step 1. Roadway development.



Step 2. Pillaring, critical span, stress redistribution.



Step 3. First cave, cantilever roof.



Step 4. Cantilever roof failure.

to air blast and bumps. The force of the wind may cause injury to personnel, damage mining equipment and ventilation infrastructure and can increase the hazard of explosion by expelling methane from the gob and mixing it with the raised coal dust to form a potentially explosive mixture.

Mark et al. (1997) present a case study of a shallow depth (53 to 66 meters of overburden cover) retreat room and pillar mine in Logan County, West Virginia characterized by fine-grained, semi-laminated sandstone with a CMRR of 64%. Three meter high roadways were developed on 15 meter centers with 5 meter splits for an overall extraction ratio between 76 and 86%. During pillaring operations, a massive collapse of fenders occurred in the gobbed-out area. The roof bolter operator was knocked to the floor by the resulting air blast and 103 stoppings were destroyed.

Lind (2002) highlighted full extraction operations in New South Wales that resulted in fatalities and/or injuries associated with windblasts resulting from massive roof strata failure. Lind adds that hydrofracturing – the application of high pressure water injection or blasting in boreholes preceding the line of mining to induce breaking of the massive strata to encourage early caving – has been applied successfully in longwall operations under a massive roof. Hydrofracturing is not generally applied in New South Wales because a full risk analysis and mitigating control measures are required if caving of massive roof is planned.

Systematic and controlled roof caving with mudstone/siltstone strata as practiced at the 2N/3N mine at Quinsam coal and other mines globally. The mudstone/siltstone strata is amenable to full extraction methods due to its lower RMR of 40 to 50 percent, considerably greater joint density and weathering sensitivity. In pillar operations, mudstone/siltstone strata caves readily at roof spans of approximately 20 meters and tends to fragment into smaller rock assemblages.

6.4 ROOF STABILITY MONITORING

Falls of ground can happen unexpectedly and progress rapidly after the first observable signs of instability. Observational techniques in coal mining include the following:

1. Visual inspection
 - a. Changes in joint dilation (through use of wooden wedges)
 - b. Rock dust and rock debris falling from roof
 - c. Bending of rock bolt plates (ideally before the onset of the rock bolt yield point)
 - d. Failure of rock bolts
 - e. Slabbing or sloughing of ribs indicating reduction of the rockmass strength
2. Sudden changes in the penetration rate of roof drilling
3. Sounding of the roof rock or rock bolts for relaxation

While observational techniques are an important means of determining roof stability, their effectiveness can be extended by monitoring the movement of the mine roof in boreholes using mechanical tools. Iannacchione (1999) list four types:

1. *Scratch tools*: can detect separations and provide an indication of loose rock layers or roof beam deflection.
2. *Telltails*: are rigid bars, possibly just a roof bolt, anchored into the roof. A small section of rod protruding from the borehole is covered in three bands of reflective tape: green, followed downward by yellow and then red. As the roof deflects downward, the roof line can easily be seen to move through the green, yellow and red tape zones.
3. *Mechanical extensometers*: consist of a top and bottom anchor, steel wire or rigid tubing, and some kind of micrometer or dial gauge.
4. *Electronic extensometers*: Include sonic probes that allow for up to 20 permanent anchors up to a 6 m height. The probes have the added benefit of

being remotely read by portable devices or by connection to a data acquisition system. For use in coal mines, they must be approved intrinsically safe devices.

Critical roof deformation levels and responses need to be identified based on the rock mechanical conditions and deformation characteristics at the mine site. Measured deformations will be a contribution of deflection of the roof skin and internal deformation within the roof rockmass.

Roof deflections of roadways can be predicted by conventional gravity loaded beam theory as the length of roadways exceeds twice their width. The maximum deflection formula is shown below:

$$\eta = \frac{\rho g L^4}{32 E t^2} \quad (6.9)$$

where: η = Maximum deflection (m)

ρ = density (kg/m³)

g = gravity (m/sec²)

E = Modulus of Elasticity (N/m²),

t = thickness of layer (m)

L = span width (m)

The amount of roof sag is proportional to the fourth power of road width. This means that small increases in roadway width translate into significantly larger maximum roof deflections. The effect is more pronounced for intersections where deflections are theoretically about 4 times as large. Intersection span increases due to rounding of pillar corners that are rounded for machine travel (turn-outs) or rib spalling require constant vigilance. In 1996, there were 2,105 non-injury reportable roof falls. More than 71% of these occurred in intersections, despite the fact that intersections probably account for less than 25% of all development underground (Mark et al. 2001). Equation 6.9 predicts maximum deflections of 1.8 to 5.1 mm for 6-meter width roadways and 7.0 to 19.5 mm for intersections in the 4 South mine.

Ghosh and Ghose (1992) attempted to establish a relationship relation between the maximum ground movement in a roadway with the rock mass rating, the roadway width and the rock dry density and presented the following relation based on case histories from eight different coal mines in India:

$$C_m = 40B^{0.5}\gamma^{0.3}(1 - R/100)^3 \quad (6.10)$$

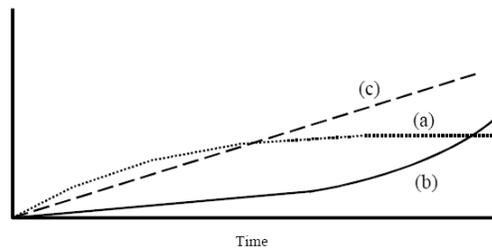
where: C_m = maximum ground movement (mm)
 B = roadway width (m)
 γ = rock dry density, (kg/m³)

Equation 6.10 predicts a maximum ground movement of 3.5 mm for 4 South sandstone roof in 6 m wide roadways.

6.4.1 Critical Convergence Rate

The rate of movement in the immediate roof has been shown to be a reliable measure of roof instability. Maleki (1988) proposes the rate of movement is favorable to other metrics of instability because a) the rate does not depend on the entire history of roof movement and b) it indicates a change in the stability of the whole mining system. Van Der Merwe (1998) reported on three common displacement-time behaviors for mine roofs as illustrated in Figure 6.7. In this figure, curve (a) represents stable roof requiring monitoring at long intervals, (b) acceleration, typical of imminent failure, and (c) steady deformation, where failure occurs when the maximum magnitude of displacement is reached.

Figure 6.7: Displacement vs. Time Behavior of Roofs; after Van Der Merwe, J.N (1998).



Ghosh and Ghose (1995) instrumented the intersections of several underground coal mines in India to develop a relation between the critical convergence velocity, the

roadway width and the rock dry density. Equations 6.11 and 6.12 show the empirical formulas they developed for critical convergence and maximum convergence velocity respectively:

$$V_R = 2.25B \left(\frac{\Gamma}{1000} \right)^{0.66} \left(\frac{100 - R}{100} \right)^6 \quad (6.11)$$

where: V_R = critical velocity (mm/d)

B = roadway width (m)

Γ = rock dry density, (kg/m³)

$$V_{R_{\max}} = 3.3B^{0.55} \left(\frac{\Gamma}{1000} \right)^{0.36} \left(\frac{100 - R}{100} \right)^{3.3} \quad (6.12)$$

Equations 6.11 and 6.12 are based on roadways of width 3.0 to 4.8 meters and rock mass ratings between 19 and 50%. These formulas predict a critical convergence and maximum convergence velocity of 0.018 mm/day and 0.23 mm/day respectively for the 4 South mine roadways. Pakalnis (2009) assumes a critical convergence rate of 1mm/day for consulting work in hard rock mines. Cullen (2002) found a critical convergence velocity of 0.03 mm/min (432 mm/day) for the 2N/3N mine based on instrumented studies. These results indicate the critical convergence velocity for a massive sandstone roof is more than two orders of magnitude less than for a weaker mudstone/siltstone roof. Convergence measurements in-field at the 4 South mine have not been completed and should be considered for future work. A critical convergence velocity of 1 mm/day is recommended for a massive sandstone roof.

6.4.2 Convergence Measurements in the 2N/3N Mine

A convergence measurement rod was designed and constructed to test in the 4 South mine; however, due to inactivity in the mine, the test program was moved to the 2 North / 3 North mine to determine the effectiveness of the instruments in active mining areas. Figure 6.8 shows a photo of the convergence rod.

Figure 6.8: Air Pump Convergence Stick



The instrument is made almost entirely of polyvinyl chloride (pvc) plastic. The outer pipe is 2.1 meters long x 32mm diameter and fitted with (a) a rubber o-ring inset about 12mm from the open end of the pipe (b) a solid pvc plug sealed into the bottom of the tube and (c) a standard tire valve secured at about 38mm from the bottom of the pipe. The inner pipe is 2.1 meters long x 25 mm diameter and fitted with an end cap and scaled at 5mm increments. The instrument weighs approximately 2 kg and is safe for use in underground coal mines. The average materials and fabrication cost per instrument is \$35CAD.

The instrument as fabricated can be installed without props in heights up to 3.2 meters. Props can be added to the end of the instrument to increase its operating height to a maximum of four meters. The instrument is activated with a standard manual air pump with installation pressure ranging from 10 to 15 psi. Wooden blocks can be used to increase the bearing surface of the base where necessary to increase the stability and accuracy of the instrument in soft floor conditions.

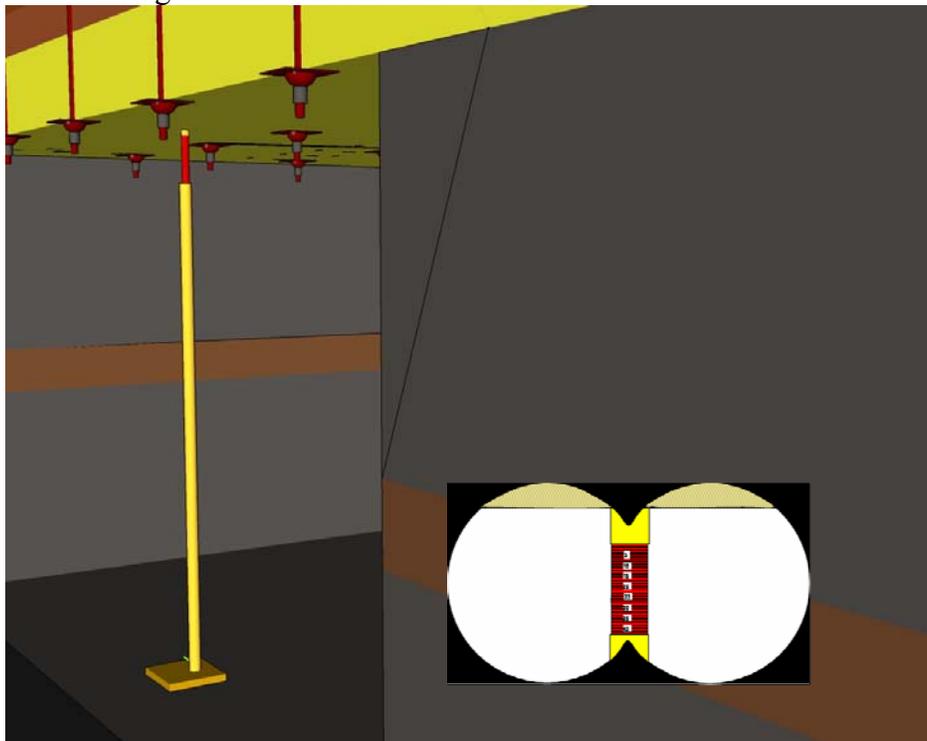
The instrument can be read manually in safe installation locations, or remotely via binoculars. For this study, 20 x 50 Barska wide angle binoculars were used for remote reading of instruments in gob areas. Figure 6.9 illustrates a typical instrument setup

Three instruments were installed inbye the 16 Panel gob at (1) the intersection of A-road/ 14 cross-cut, (2) C1-road/ 13 cross-cut and (3) C-road/ 13 cross-cut. Instrument No.1 was installed on August 2, 2004 approximately 20 meter inbye the active gob and was active for three days. A convergence reading of 10 mm was read on the last day of service. Instrument No.2 was installed on August 3, 2004 and rendered inactive on August 6th. On August 5th, the first convergence reading was taken at a value of 10 mm with the active gob approximately 12 meters north of the instrument. A further 10 mm of displacement was recorded without any noticeable progression of the gob on August 6th.

Instrument 3 was installed on August 7th and was knocked over on August 10th. The frequency of convergence measurements was increased to 3 per day for this installation. On August 7th, the first convergence reading was taken at a value of 5 mm with the active gobs about 15 meters inbye to the north and 12 meters inbye to the east on the instrument. A total displacement of 33 mm was recorded over the service period. A convergence rate of 10 mm/day was recorded. Figure 6.10 depicts the instrument setup locations and convergence readings.

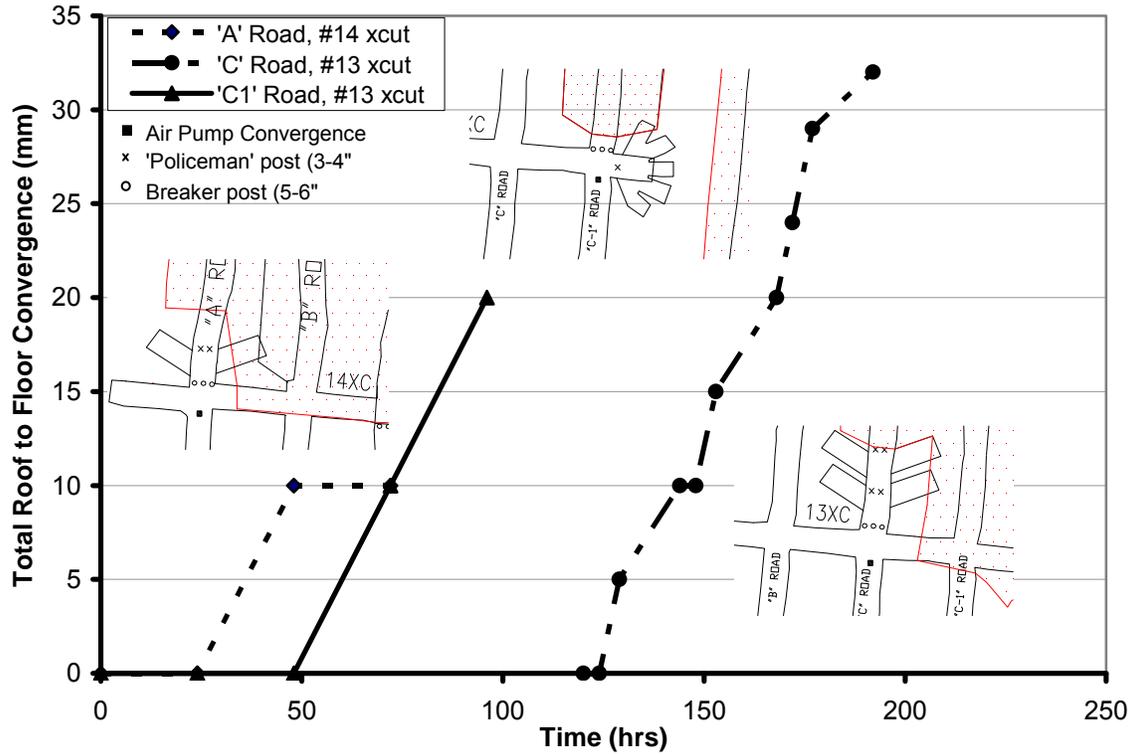
Convergence rates of 10mm/day were recorded by the instruments, which represents double the recommended critical convergence value of Cullen (2002). Floor displacement is assumed to be contributing to the total roof-to-floor convergence in these trials.

Figure 6.9: Convergence Rod Installation.



The use of the convergence rod in the 4 South mine as a primary roof convergence measurement tool is inconclusive, due in part to the inability to directly test them at the 4 South mine, and also due to the inability to detect changes in the roof convergence rate with the instruments. Further trials are necessary to make a decision in this regard.

Figure 6.10: 2N/3N Convergence Instrumentation Results.



6.5 CONCLUSIONS AND RECOMMENDATIONS

The cavability of massive sandstone roof has been shown to be very low based on the classification systems presented. At the 4 South mine, caving of the massive sandstone roof was difficult and unpredictable. Massive sandstone roof has been shown to bridge spans of up to 40 meters at the 4 South mine. The literature indicates that spans up to 70 meters may be possible in massive sandstone strata. Table 6.5 summarizes suggested critical span for the 4 South sandstone roof by reference in this chapter.

Table 6.5: Critical Span Prediction for 4 South Sandstone Roof by Chapter Reference.

Source	Researcher	Suggested Critical Span (m)
Updated Span Curve	Ouchi et al. 2004	15 to 25
Stand-up time vs Rock mas quality	Bieniawski 1989	20
Maximum span vs NGI-Q rock quality	Barton 1974	19 to 32
Critical span vs NGI-Q rock quality	Carter 2000	15
Self-Supporting span vs Rock Formations in S.A. coalfields	Wagner 1991	35 to 45
Cantilever roof beam model		14 to 18
Stability guidelines for Jointed Rock Beams (tunnel span)	Diedrichs et al 1999	45
Stability guidelines for Jointed Rock Plates (square span)	Diedrichs et al 1999	72

A conceptual model for caving of massive sandstone roof has presented to show the progression of caving and the dangers associated with the massive blocks of roof rock that rotate out and potentially initiate airblasts/bumps. A comparative review of the caving of mudstone/siltstone roof strata that it is more amenable to caving with a critical convergence velocity orders of magnitude greater than that for a massive sandstone roof. A rate of 1 mm/day is recommended for the massive sandstone roof at 4 South mine.

Data is required to determine reliable critical roof convergence rates for massive sandstone roof. In-field trials of the air pump convergence sticks are recommended. Alternative convergence monitoring instruments should also be considered. Empirical relationships to determine the critical and maximum convergence velocity for underground roadways investigated in this chapter are based on low rock mass quality sandstone and not applicable to the 4 South mine.

CHAPTER 7 ENVIRONMENTAL RISK MANAGEMENT

7.0 INTRODUCTION

The Quinsam Coal Mine, which is comprised of the current 2 North / 3 North mine in the North mining area and the 1, 2 and 3 South open pits, and 2 South, 4 South and 242 mines in the South mining area, is surrounded by fish bearing lakes which empty into the Quinsam River, a heritage river for the community of Campbell River that is used for fish stocking and rearing in support of the valued local fishing industry.

Effective environmental management must be planned for and demonstrated to support new mining permit applications and maintain a legal and social license to operate. The future of the 4 South mine weighs heavily upon diligent management of environmental issues relating to sulphate concentrations in the underground mine water discharge and heavy metals in the coarse coal refuse produced from processing No.3 seam coal. Future mining plans for 4 South mine consequently, must consider the environmental implications of mining activity with the intent to minimize environmental risk and liabilities and safeguard the ecosystem and watershed.

This chapter intends to relate both the underground water discharge and coarse coal refuse waste quality and chemical characteristics to mining plans for the 4 South mine. The first section will introduce the water management system handling the 4 South mine water discharge and follow with a discussion on water quality monitoring data and the implications on mining. Section two will discuss coarse coal refuse monitoring quality data and follow with details on leach pad tests conducted on mixes of 2 North and 4 South coarse coal refuse. Finally, conclusions and recommendations will be presented.

7.1 WATER MANAGEMENT

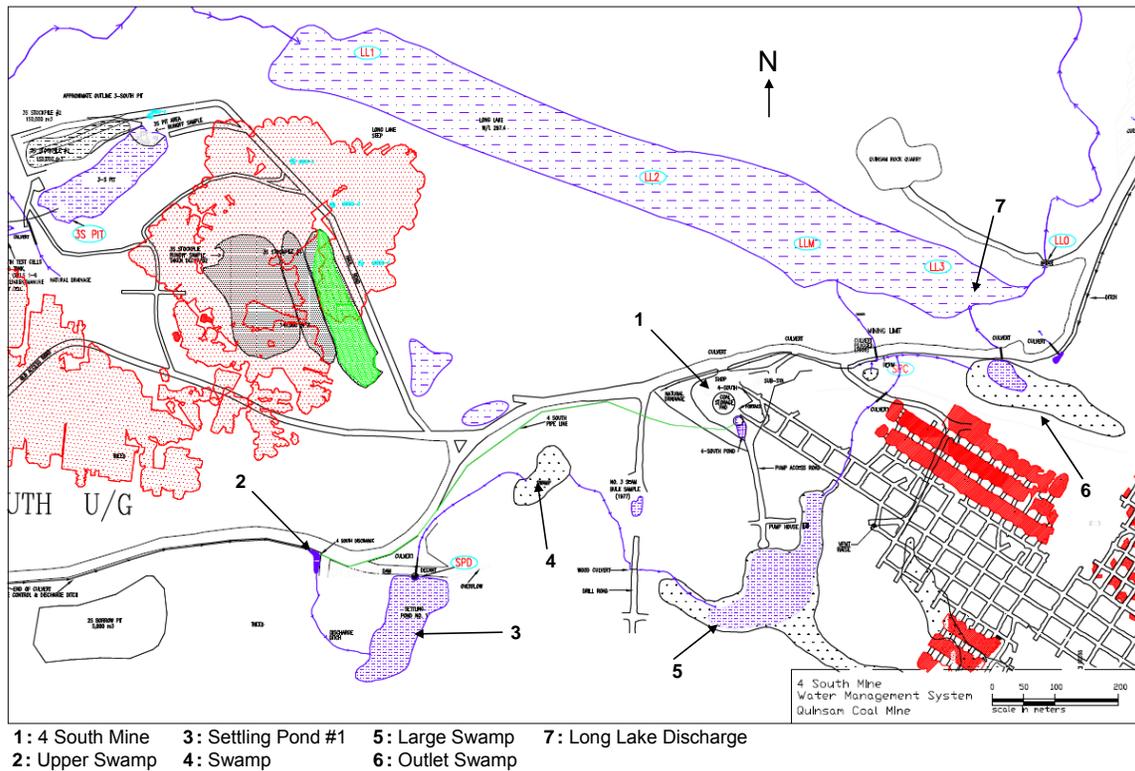
Quinsam Coal Corporation is aware that water management is required to meet effluent discharge flow restrictions and water quality limits set out in Mine Permit PE-07008. Long Lake, the receiving environment for 4 South mine water discharge, has sulphate concentrations that at depth, exceed the British Columbia Water Quality guidelines for

sulphate in receiving environments. Any mining activity in the 4 South mine must consider the effect of the mine discharge water on Long Lake and ultimately, the Quinsam River which accepts outflow from both Long Lake and Middle Quinsam Lake.

7.1.1 Water Management System

Figure 7.1 illustrates the water management system for 4 South Mine discharge water.

Figure 7.1: 4 South Mine Water Management System.



Mine discharge water from 4 South is pumped from underground settling sumps using a 65HP Flygt pump into the 4 South Pond - a temporary settling sump for underground mine water located outside the underground mine portals at the south end of the 4 South Mine pad (1) - prior to being pumped to the Upper Swamp (2). Approximately 4 m³/h of water is pumped to the Upper Swamp during the summer months and 83 m³/h during the winter months. Water from (2) reports to Settling Pond #1 (SPD) – the primary settling pond for the South mining area and point of compliance for water quality – identified as location (3). The effective catchment area of Settling Pond #1 is 68.4 ha. Water exits (3) through an orifice plate at the north end of the settling pond and reports to swamp (4), (5) and (6) respectively before discharging into the east end of Long Lake at (7).

7.1.2 Water Quality Monitoring Data

Effluent water quality samples are collected at designated monitoring stations around the mine site. Water quality monitoring is performed daily, monthly or bi-monthly as specified by environmental permits which can vary depending on flow rates. Laboratory analyses are carried out on effluent samples for the following water quality parameters as per the requirements of Permit PE-07008:

- pH
- conductivity
- total suspended solids (TSS) and total dissolved solids (TDS) (mg/L);
- alkalinity (mg/L as CaCO₃);
- hardness (mg/L as CaCO₃);
- sulphate (mg/L);
- ammonia nitrogen, nitrate/nitrite nitrogen (mg/L);
- total and dissolved phosphate (mg/L)
- total and dissolved metals (mg/L)
- oil and grease (for stations SPD and WD only) (mg/L); and
- rainbow trout bioassays (for stations SPD and WD only).

Water quality samples are typically sent to ALS Environmental in Vancouver for analysis.

Three primary water quality monitoring stations are used to monitor the impact of 4 South mine water discharge: 4 South Pond, SPD, and SPC. Two stations, 4 South Upper and 4 South Lower, are used to determine the effect of the 4 South mine coal storage pad on a natural surface stream flowing underneath the coal storage pad via a culvert.

The water quality parameter of most interest in the South mining area is sulphate. Sulphate occurs naturally in minerals such as gypsum (CaSO₄·2H₂O) and as a result of

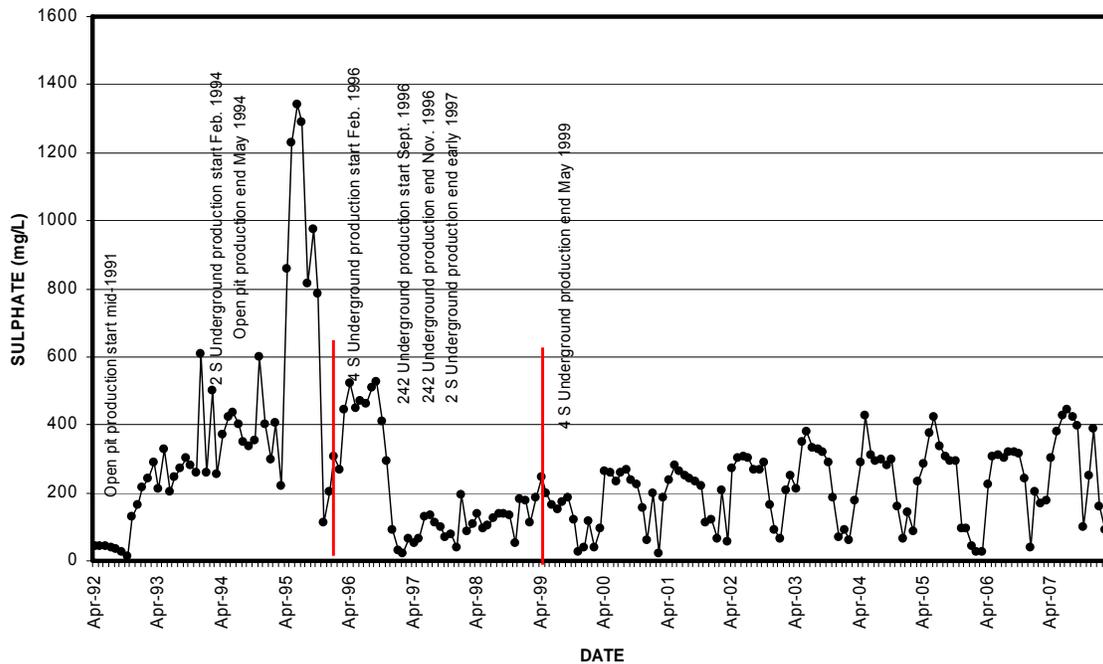
oxidation of sulphide sulphur in minerals such as pyrite (FeS_2) and other metal sulphides (Greschuk 2008).

In 2000, the BC Ministry of Environment developed an interim water quality guideline for the protection of aquatic life for sulphate of 100 mg/L (as dissolved SO_4). This guideline value is considered to be a maximum that should not be exceeded at any time. In addition, the BC Ministry of Environment presented an “alert level” of 50 mg/L, above which occasional monitoring of the health of aquatic mosses should occur. The scientific rationale for this guideline rests primarily on aquatic toxicity data for three aquatic species shown in historical laboratory exposures to be very sensitive to sulphate: *Fontinalis* sp., an aquatic moss; *Hyaella azteca*, an amphipod; and larval striped bass. However, the validity and relevance of the data obtained from these sulphate-exposure tests, have been questioned in recent studies. For aquatic moss and *Hyaella*, particularly, historical results indicating high sulphate toxicity have been linked to methodological flaws in these tests, such as inappropriate sulphate salts or test solutions.

The 4 South Pond represents the water quality being delivered to Settling Pond #1 (SPD). Monthly samples have been collected from the 4 South Pond since October 2001. Sulphate concentrations at 4 South Pond ranged averaged 414 mg/L in 2007/08, up from the previous years’ average concentration of 287 mg/L.

Sulphate concentrations at Settling Pond #1 (SPD) are shown below in Figure 7.2. Sulphate concentrations at SPD spiked in mid-1995 with the start-up of the 2S underground mine at 1350 mg/L, and then decreased to an average concentration of 10 mg/L in late 1996, shortly after the start of production from the 4 South mine in February 1996. Sulphate concentrations continued to trend upward following the significant decrease in production from the 4 South mine in May, 1999. High sulphate concentrations appear to be linked with open pit operations in the South mining area and the 2S underground mine, and underground production from the 4 South mine. It is believed that the ongoing increase in sulphate at Settling Pond #1 is linked to the oxidation of sulphate bearing rock on surface.

Figure 7.2: Sulphate Concentrations (1992- 2007) at Settling Pond #1 (SPD); after Greschuk (2008).

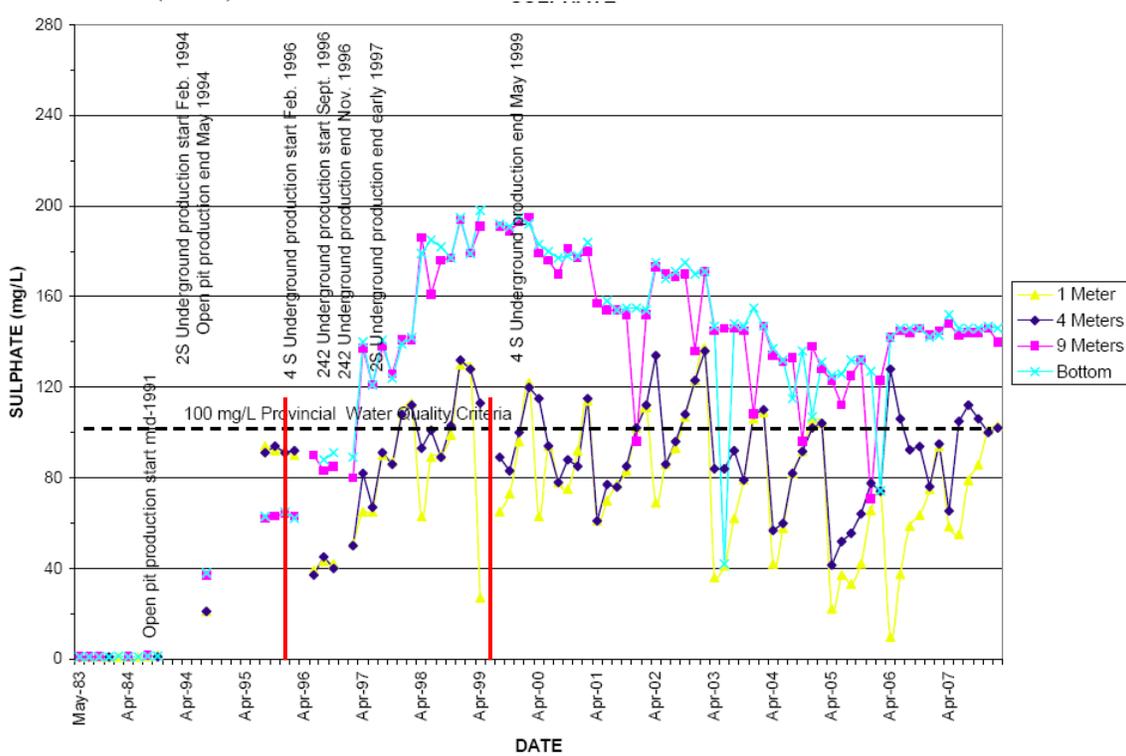


The culvert into Long Lake (SPC) is used to monitor the quality of mine discharge water before it reaches Long Lake. SPC had an average sulphate concentration of 121 mg/L for 2007/08 and averaged 119 mg/L over the previous 4 years. The decrease in sulphate along the flow path suggests that sulphate may be partially removed as it travels through a series of wetland meadows to the culvert and location of the sampling point.

The 4S Upper and 4S Lower monitoring stations are regularly sampled by QCC and submitted for analysis of sulphate. To date, the results indicate that the 4 South coal pad and coal stored upon it have not had an impact on pH and concentrations of acidity, alkalinity and sulphate in this stream.

The primary location for water quality compliance for sulphate concentrations in the South mining area is Long Lake. Lake water samples are collected for the months of April to September at depths of 1 meter, 4 meters, 9 meters, and 1 meter above lake bottom. Figure 7.3 shows the historical sulphate concentrations by depth for Long Lake.

Figure 7.3: Sulphate Concentrations (1994- 2007) at Long Lake; after Greschuk (2008).



The results clearly indicate the 4 South mine has been contributing to the increase in sulphate levels in Long Lake, particularly at depth where concentrations peaked at approximately 190 mg/L.

7.1.3 Implications for Mining

The 4 South mine has been shown to contribute a sulphate loading to Long Lake from underground mine water. The mine water flow rates to Long Lake are seasonal, being higher in the winter months and lower in the summer months. A ground water source to Long Lake from the underground workings is assumed, but flow rates are not known.

Caving mining methods for the 4 South mine exacerbate the amount and quality of mine water managed by increasing the frequency and extent of fractures in the overburden and consequently the hydraulic conductivity of the overburden. This increase translates into increased groundwater flow to the underground and sulphate loadings to the receiving environment. A non-caving method provides an abundance of water management storage

underground, minimizes disturbance of the mine roof and consequently minimizes the amount of mine water discharge and sulphate loading to the receiving environment.

7.2 COARSE COAL REFUSE MANAGEMENT

Coarse coal refuse (CCR) is a byproduct of coal washing and consists of interseam ash and rock mined during coal extraction. The heavy media bath circuit, consisting of Eagle Washers, uses a mixture of fine magnetite and water to exploit the density difference between rock and coal and selectively extract CCR from the ROM coal. CCR is conveyed from the wash plant into 65-tonne CAT haul trucks for final placement.

Quinsam Coal Corporation is currently permitted to use CCR in the construction of the outer shell of the fine tailings impoundment at the mine site only if, as writ in the Mine Permit C-172, the CCR is analyzed and found to be non potentially acid generating (NPAG). Bi-weekly Acid-Base accounting (ABA) analyses on composite coarse reject samples are analyzed by Sturm Environmental Services of Bridgeport, West Virginia, U.S.A. CCR analyzed in this way must have a Sobek neutralizing potential ratio (NPR) – the ratio of neutralizing potential to acid-generation potential – greater than two. When this condition is not satisfied, the material is classified as potentially acid generating (PAG) and must, under the terms of mine permit C-172, be deposited sub-aqueously in the 3 South pit located in the South mining area.

For the first 17 months of production at 4 South Mine (February 1996 to May 1999), 4 South and 2 North CCR was blended and deposited in the 2 North dump in one meter lifts, with a half meter thick till cap applied every three meters or third lift. Phillipone et al. (1999) reconstructed the depositional history of the 2 North dump and estimated a ratio of 2 North to 4 South CCR (by mass) of 20:1 for 1996 and between 3:1 and 5:1 for 1997. The purpose of the capping was to limit precipitation and surface runoff infiltrating the material. Phillipone et al. (1999) reported on the ARD potential of the material in-situ to determine whether the blending covering coarse coal refuse would be effective option to mitigate acid rock drainage (ARD) and metal leaching (ML). Although this report concluded that the ARD potential for the entire blended mass was considered to be low,

there were documented occurrences of sulphide oxidation within the dump containing the blended CCR. The depositional history of 4 South CCR post August 1997 is vague; however it is strongly believed than surface blending of 4S and 2N CCR continued up until May 1999.

Blending and covering of coarse coal refuse was not endorsed by the BC Ministry of Mines Energy and Petroleum Resources (MEMPR) to manage coarse coal rejects from 4 South mine in light of its potential for acid drainage and metal leaching. MEMPR directed Quinsam Coal to investigate sub-aqueous management instead, which was viewed as an acceptable and environmentally responsible long-term solution to mitigating ARD/ML.

In February 2000, Keystone Environmental Ltd. was retained by Quinsam Coal Corporation to perform a water balance study for the 3-South Pit. The water balance study was required by the Ministry of Employment and Investment (MEI), Mine Review and Permitting Branch to “establish the necessity of the water management works in the south pit areas for the maintenance of permanent flooded conditions of potentially metal leaching/ARD generating coarse refuse (Cameron et al., 2000).” Since early 2000, CCR from the 4 South mine has been deposited sub-aqueously in the 3 South Pit. To date, sub-aqueous disposal of PAG CCR is the only permitted disposal option available to Quinsam Coal Corporation.

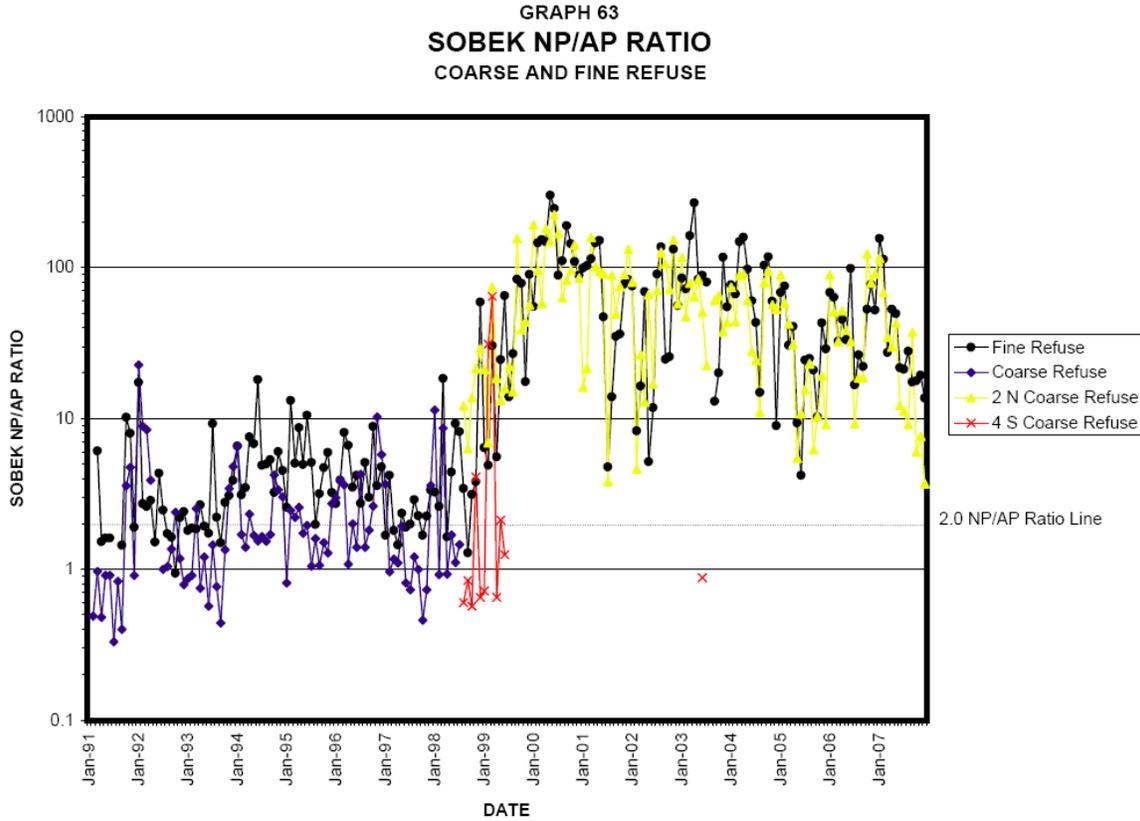
7.2.1 Monitoring and Quality Data

Figure 7.4 illustrates the historical Sobek NP/AP Ratio for both fine and coarse coal refuse.

The No.3 Seam coal mined at 4 South has a high in-situ sulphur pyritic sulphur content of three to four percent which contributes to the poor CCR quality. The 4 South coarse coal rejects have a characteristic high acid potential and low neutralizing potential which also appears characteristic of coarse coal rejects from ROM coal mined from the South

mining area. After production from the 4 South mine ceased in May 1999, CCR quality improved considerably, with NPR results 10 to 100 times higher being recorded.

Figure 7.4: Historical Results for Sobek NP/AP Ratio for Fine and Coarse Coal Refuse at Quinsam Coal Mine (1991 to 2008); after Greschuk (2008).



7.2.2 Leach Pad Tests

In 2003, a request of the Environmental Technical Review Committee (ETRC) was made to construct three coal refuse leach pads to evaluate the chemical composition and total and dissolved metals content of the effluent. Approximately 11 tonnes of CCR at -100mm size were used for each leach pad, emplaced on a 45 mm EPDM liner. The CCR prescription for each leach pad is shown below:

1. 4 South (4S)
2. 2 North (2N)
3. 50/50 mix of 4 South and 2 North (4S/2N)

The CCR leach pads were located alongside the coal preparation plant wash ditch on the west side of the till stockpile in the North mining area. Quinsam Coal Corporation monitors the water quality in the CCR leach pad effluent water on a weekly basis. Golder Associates Ltd. monitors the same on a quarterly basis. Table 7.1 shows a statistical analysis of leach pad effluent water parameters from April 2005 to April 2008 as sampled by Quinsam Coal Corporation.

Table 7.1: Coarse Refuse Leach Pad Water Chemistry – Statistical Analysis of Data from April 2005 to April 2008.

	4S-2N				2N				4S			
	Avg	Max	Min	SD	Avg	Max	Min	SD	Avg	Max	Min	SD
Conductivity (umhos/cm)	4279.2	10700.0	734.0	2402.7	136.4	237.0	53.1	54.5	8363.3	27300.0	2180.0	6765.0
Hardness CaCO ₃ (mg/L)	1035.9	3520.0	56.8	722.9	48.0	86.8	14.3	17.8	845.3	1930.0	238.0	495.9
pH	2.5	2.9	2.2	0.2	7.5	8.8	6.3	0.6	2.1	2.6	1.4	0.3
Total Suspended Solids (mg/L)	20.6	141.0	2.4	34.4	15.6	47.2	3.0	13.8	67.8	521.0	3.7	132.2
Acidity (to pH 8.3)	6373.5	31000.0	165.0	7778.5	4.2	15.2	1.0	3.5	14948.6	57900.0	1570.0	19229.9
Alkalinity - Total CaCO ₃ (mg/L)	1.9	2.0	1.0	0.4	51.7	80.6	21.7	16.9	1.8	2.0	1.0	0.4
Sulphate (mg/L)	4695.0	20500.0	195.0	4874.3	20.2	56.9	5.2	15.2	11946.7	43000.0	1600.0	14743.5

The effluent quality of the 4S CCR, and to a lesser extent, the 4S/2N CCR, is poor with high sulphate, high acidity and low pH. The high hardness of the effluent is ameliorating to some extent. Table 7.2 summarizes the coarse coal refuse leach pad total metals concentrations (in mg/L) where the British Columbia Contaminated Sites Regulation for freshwater environments has been exceeded. Total metal concentrations exceeded CSR standards for arsenic, cadmium, chromium, cobalt, and copper for samples from both the 4 South and 4 South/2 North leach pads. Table 7.3 summarizes acid-base accounting data on CCR samples from the leach pads from 2003 and 2005 as reported by Smith et al (2005).

Table 7.3: 4 South and 2 North Acid-Base Accounting Results (2003, 2005).

Location	Parameter							Year
	Paste pH (s.u.)	Total Sulphur (%)	Sulphide Sulphur (%)	Sulphate Sulphur (%)	AP	NP	NPR (NP/AP)	
4 South	3.60	11.20	8.99	1.74	280.94	179.68	0.64	2003
2 North	7.60	0.10	0.04	0.03	1.16	98.59	84.99	
4 South / 2 North	4.50	2.07	1.45	0.50	45.31	97.09	2.14	
4 South	5.60	6.42	5.05	1.37	157.81	34.60	0.20	2005
2 North	7.40	0.07	0.06	0.01	2.19	29.40	13.40	
4 South / 2 North	5.00	2.44	1.79	0.65	55.94	2.70	0.10	

Table 7.2: CCR Total Metals Concentrations where Parameter Exceed CSR (freshwater) - Average from QCC April 2003 to April 2005, Golder Associates Ltd. from October 2003 to February 2005; after Smith et al. (2005).

Parameter	Aquatic Life CSR-AV (Freshwater) ¹	Quinsam Coal Corporation			Golders Associates	
		4S-2N	2N	4S	4S-2N	4S
Arsenic	0.05	5.88	-	27.65	2.96	46.7
Cadmium	0.0001-0.0006	0.0085	-	0.0054	0.0048	0.0095
Chromium	0.01 ^{VI} , 0.09 ^{III}	0.96	-	0.28	0.54	0.35
Cobalt	0.04	0.98	-	0.64	0.64	1.43
Copper	0.02-0.09	11.22	0.011	3.65	5.96	3.67
Nickel	0.25-1.5	2.36	-	1.01	1.47	2.16
Zinc	0.075-2.4	4.23	0.04	2.42	2.71	5.31

Reference:

1. British Columbia Contaminated Sites Regulation (CSR) [en. B.C. Reg. 324/2004, s.68; am. B.C. Regs. 405/2004; 76/2005; 239/2007, s.7].

Notes:

All concentrations are in milligrams per litre (mg/L) unless otherwise noted

Shaded numbers exceed the CSR Aquatic Life Standard (2007)

CSR Aquatic Life Standards for copper, lead, cadmium, copper, lead, nickel, silver and zinc represent a range based on hardness

4 South coarse coal refuse experienced a net decrease in neutralizing potential, acid potential and sulphur content between 2003 and 2005, believed to be the result of sulphide oxidation and leaching. The 4S/2N coarse coal refuse mixture did not indicate an apparent decrease in sulphur content between the 2003 and 2005 sampling and testing. The results of the ARD assessment are consistent with the effluent quality results from the leach pads, which indicate ARD generation from leach pads 4S and the 4S/2N coarse refuse mixture and slightly basic to slightly acidic drainage from the 2N leach pad.

7.2.3 Implications for Mining

Coarse coal rejects produced from the processing of 4 South Rom coal have been shown to be potentially acid generating. Heavy metal concentrations in excess of the BC Contaminated Sites Regulation guidelines for total metals in freshwater environments have also been shown to be produced from effluent sourced from the coarse coal rejects. Consequently, CCR from the 4 South mine is currently required to be disposed of sub-aqueously in the 3 South pit. Blending of the 4 South and 2 North production does not appear to increase the NPR of the CCR produced, nor reduce the concentrations of metals in effluent from the coarse coal rejects exposed to the air and precipitation.

Reopening of the 4 South mine and maximum profitability would accrue from selling 4 South coal unwashed. If this is not possible, a more substantial and permitted sub-aqueous storage site is required to contain 4 South CCR.

Non-caving mining at 4 South provides an opportunity to store CRR underground in the mined out panels, thereby minimizing environmental risk and promoting the restart of the mine. The economics of underground stowing of CCR and/or capitalizing a new, long-term CCR sub-aqueous disposal site should be reviewed if 4 South mine production is planned to be processed. Environmental risks and liabilities associated with water treatment and CCR management would also have to be considered.

7.3 CONCLUSIONS AND RECOMMENDATIONS

Environmental management of the 4 South mine effluent and coarse coal refuse affects the sustainability and economics of the mine plan. In review of the water management system for the 4 South mine effluent; the following conclusions can be drawn:

- There is a decrease in sulphate along the flow path to Long Lake, indicating a remedial effect to water quality due to the presence of intervening wetland meadows,
- The 4 South mine contributed to the large increase in sulphate concentrations in Long Lake, particularly at depth that currently exceed the BC water quality guideline for sulphate of 100 mg/L,
- Non-caving mining methods for the 4 South mine minimize environmental risk by reducing the volume and sulphate loading of mine water managed, and providing for opportunities to store coarse coal refuse underground.

A review of the coarse reject deposition history found that blending of 2 North and 4 South CCR is not a dependable method for mitigating ARD and is prone to spontaneous combustion processes. Under Mines Act Permit C-172, the Quinsam Coal mine must dispose of PAG CCR in the 3 South open pit, a permitted sub-aqueous disposal site with a finite capacity.

A review of the historical ABA analyses on 4 South CCR found the material to be potentially acid generating (PAG), characterized by a high acid potential and low

neutralizing potential ratio (NPR). Sub-aqueous deposition is current the only approved method of storing 4 South CCR.

Leach pads constructed of 4 South CCR and a 50/50 blend of 4 South and 2 North CCR support the disposal of 4 South CCR sub-aqueously. The average sulphate concentration and pH in the 4 South CCR leach pad effluent was approximately 12,000 mg/L and 2.1 respectively. The same parameters for the 4 South/2 North CCR leach pad were 4,700 mg/L and 2.5 respectively. The standard deviation for both average sulphate concentrations was over 100% of the average value. Total metal concentrations in the leach pad effluent exceeded the CSR freshwater guidelines for arsenic, cadmium, chromium, cobalt and copper for both the 4 South and 4 South/2 North CCR leach pads. Acid-base accounting test results on the CCR used to the construct the 4 South and 4 South/2 North CCR leach pads indicated the material to be strongly PAG.

The checkerboard non-caving mining method is also advantageous to full-extraction caving methods in that it provides an opportunity for the underground disposal of PAG CCR. Production efforts would be greatly facilitated by securing a market for unprocessed No.3 seam coal. If this cannot be accommodated, then a long term PAG CCR disposal site will be required to accommodate the 4 South CCR and efforts should be made to consider the economics of underground disposal of PAG CCR.

CHAPTER 8 MINING METHOD OPTIMIZATION

8.0 INTRODUCTION

Optimizing a coal mining method requires a myriad of considerations including the available equipment, the thickness, height and grade of the coal mining section (including interseam rock partings and rider and basal coal seams), the geotechnical condition of the immediate roof and floor, prevailing geologic structure, the coal production schedule and mine economics. Massive sandstone roof conditions make full extraction pillar mining operationally and technically challenging. As discussed earlier, massive sandstone roof can bridge large roof spans which may promote stress accumulation and chain failure of pillars or fail suddenly in large blocks such that windblasts may occur. At 4 South mine, two mining methods were used to mine out panels: in-line pillar and checkerboard partial pillar. In-line pillar allowed for caving of the roof during pillar extraction; however, difficulties experienced with in-line pillar mining supported the transition to the checkerboard partial pillar mining method which did not allow for caving during pillar extraction.

Full extraction of coal under a massive sandstone roof is very challenging. There is little empirical data to evaluate what the typical range of extraction ratio is for underground coal mines with massive sandstone roof. An extraction ratio of 76 to 85 has been reported by Mark et al. (1997), which resulted in a significant airblast incident. Extraction ratios of 35 to 70 percent are typical of room and pillar mines that do not cave the roof. When full extraction mining methods are employed, near unity extraction ratios are possible.

This chapter intends to review both the in-line pillar and checkerboard pillar mining methods and present an optimized mining method for future mining in 4 South. Section one will review the in-line depillar mining method and present a review of operational compliance to the mining plan and a stress/strain analysis assisted with the rock engineering software Examine^{TAB} developed by Rocscience (2001). Examine^{TAB} will be used to calculate elastic stresses and displacements within coal pillars and the sandstone roof. Section two provides a similar analysis for the checkerboard partial pillar mining

method. Section three details an optimized checkerboard partial depillar mining method for 4 South mine followed by presentation of conclusions and recommendation in section four.

The analysis and recommendations in this chapter are valuable to the world as a part methodology for determining a safe and productive mining method for underground coal mines with massive sandstone roof conditions.

8.1 IN-LINE PILLAR EXTRACTION (ILP)

In-line pillar mining is a partial pillar recovery method which accommodates delayed caving of the roof in order to allow safe withdrawal of personnel and equipment from the pillar extraction operation. In-line pillar recovery at the 4 South mine was first practiced south of the No.1 Mains, cross cuts 12 to 14. Pillars were designed to be a minimum of 12 meters wide based on a maximum seam thickness of three meters. This design was later modified to accommodate up to four meters of extraction height as practiced in the ILP section north of the No.1 Mains, cross cuts 6 to 14. Pillars were resized to 16 meters wide by 30 meters in length.

In the north mining ILP area, pillars were developed on 22-meter roadway centers and 36-meter cross-cut centers to the economic limits of the coal deposit. Multiple roadways were developed (up to 14) to maximize operational flexibility and increase coal recovery. Such a large number of roadways also allows for more effective dissipation of pressures produced during a windblast triggered by large areas of free-standing roof failing concurrently. Pillar recovery was conducted right to left in-line along each cross-cut and retreated back to the Mains in a typewriter fashion. Fan-out cuts of 6-9 meter depth and three meters wide and slab cuts of seven meters depth were taken where safety permitted. During pillar extraction, alternating angled cuts of 30 and 45° up to 9 meters depth were taken. The design extraction ratio for the ILP mining at 4 South mine was 67.8%. Figure 8.1 illustrates the in-line pillar mining method. A design guideline illustrating the relationship between the pillar width, the distance from the intersection to the first pillar cut and the angle of pillar cuts is shown below:

$$D = \frac{w \left(1 + \tan \frac{\alpha}{2} \right)}{2} \quad (8.1)$$

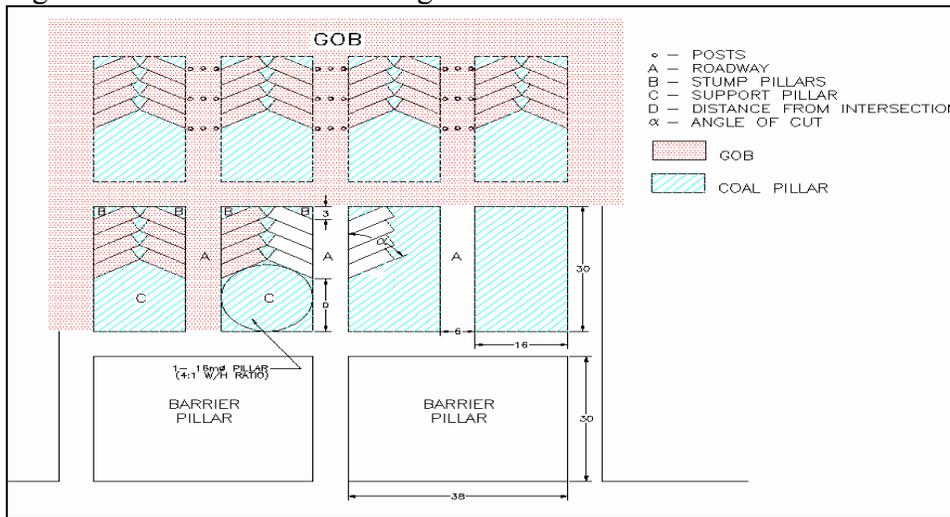
where:

w = width of pillar

D = Distance from intersection, and

α = angle of cut

Figure 8.1: In-line Pillar Mining Method.



Final coal pillars are designed to have a minimum diameter of 16 meters. Furthermore, where roof discontinuities are noted, pillars are to be extended and/or pillar stumps left in place to maintain roof stability during pillar extraction.

8.1.1 Compliance Analysis

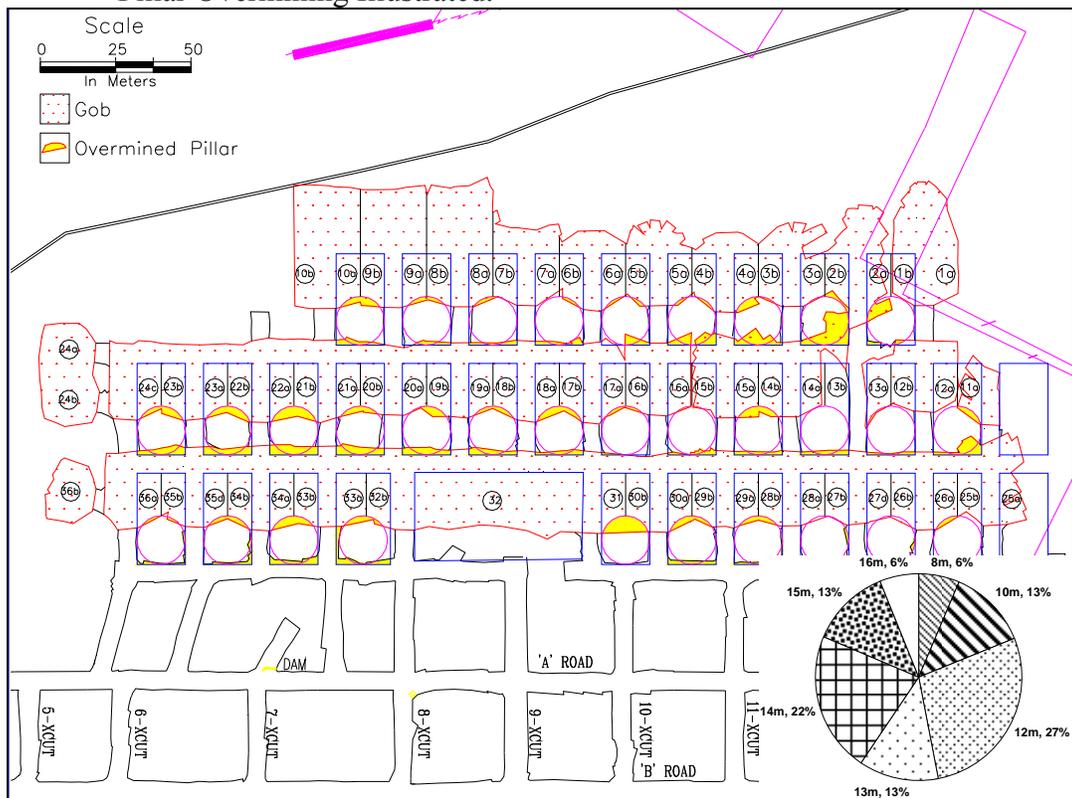
Equation 8.1 and Figure 8.1 require depillar cuts of 60° or less to achieve a minimum pillar diameter of 16 meters. The distance, D has to be at least 12.6 meters from the intersection. In order to maintain an effective pillar diameter of 16 meters, the distance D , would have to change in relation to changes in the angle of the cut α as shown below in Table 8.1.

Table 8.1: Distance from Intersection to Last Pillar Cut Required for Changes in Cut Angle to Maintain Effective Final Coal Pillar Diameter of 16 m for In-line Pillar Extraction.

α	D
45°	11.3
60°	12.6
75°	14.1
90°	16

Figure 8.2 below illustrates the level of compliance on final pillar sizing for the north ILP mining panel. A distribution of final pillar sizes is also shown in the bottom right corner of the figure.

Figure 8.2: North In-line Pillar Extraction Panel with Pillar Extraction Sequence and Pillar Overmining Illustrated.



Only six percent of the pillars in the north ILP mining panel had a minimum pillar diameter of 16 meters. Forty-six percent of pillars were of diameter 12 meters or less. The most severely overmined pillars tended to be grouped together in-line, possibly indicating overmined pillars were referenced by mining operators when starting cuts on adjacent pillars.

Typical mining spans of 22 to 24 meter diameter were made in the north ILP mining panel. The largest mining span of 40 meters was made in the north-west corner of the section, 15 meters beyond the planned limits at cross-cuts 7A, 8 and 8A. There is no record of what areas of this mining area caved and/or remained stable during pillar extraction.

The relative effect of coal pillar overmining can be investigated through the use of Tributary theory and the Bieniawski (1983) pillar formula. Table 8.2 below illustrates the effect of reduced final coal pillar sizes in the north ILP mining panel. The data is based on a three meter mining height, 60 meters depth of cover and a lithostatic stress of 1.5 MPa.

Table 8.2: Reduction of Pillar Bearing Capacities and Safety Factors Resulting From Overmining of Final Coal Pillars in North ILP Mining Panel – 4 South Mine.

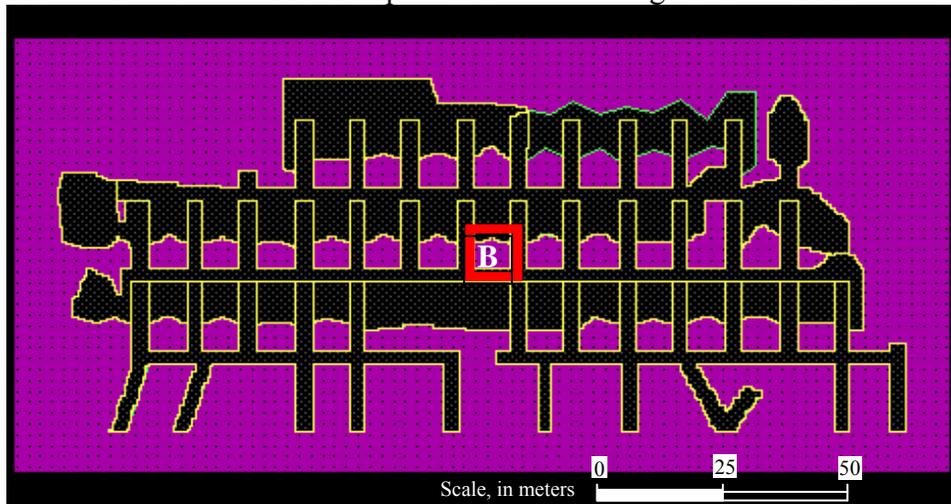
D_{eff} (m)	σ_p (MPa)	Extract. %	Trib.Str. (MPa)	S.F.
16	17.9	68%	4.7	3.8
14	16.2	73%	5.6	2.9
12	14.6	77%	6.5	2.2
10	12.9	82%	8.3	1.5
8	11.2	84%	9.3	1.2

The calculated safety factor for pillars reduced to an effective diameter of 8 meters approaches unity. Reports of unexpected caving of the sandstone roof during the time of pillar extraction (Morely et al. 2008) are deduced to have occurred in the vicinity of undersized pillars.

8.1.2 Numerical Modeling Analysis

Examine^{TAB} was used to determine the normal pillar stresses and roof displacements in the final coal pillars for the North ILP mining section. Figure 8.3 below illustrates the model setup and the location of pillar ‘B’, which was analyzed for pillar stress profiles.

Figure 8.3: Examine^{TAB} Model Setup – North ILP Mining Panel - 4 South Mine.



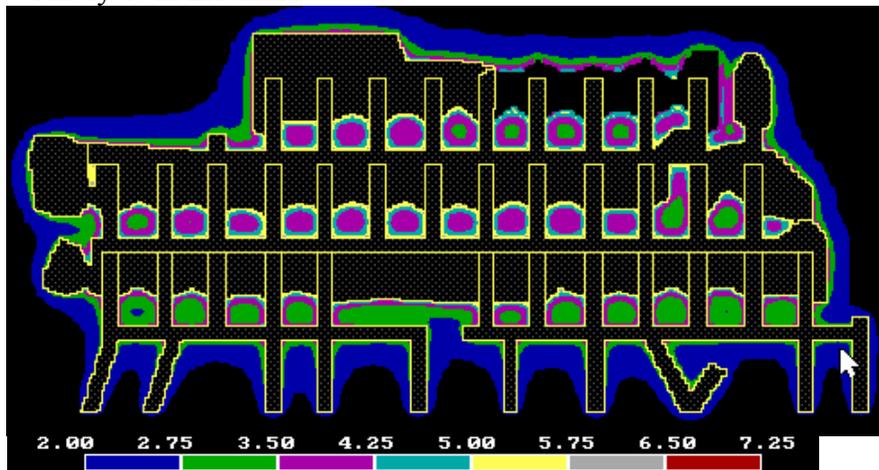
Rock mechanical property and orientation details used in the model are based on rock mass characterization data presented in Table 3.2 and are summarized below in Table 8.3.

Table 8.3: Examine^{TAB} Model Properties.

Rock Properties:	
Host rock modulus (GPa)	11.20
Poission ratio	0.16
Unit weight (MN/m3)	0.026
Pillar rock modulus (GPa)	2.80
Orientation:	
Thickness (m)	4
Depth (m)	60,100,140
Dip (°)	0
Hor to Ver. Stress Ratio	1

Figure 8.4 and 8.5 show the normal stresses in pillars and total roof displacement respectively as determined by Examine^{TAB}.

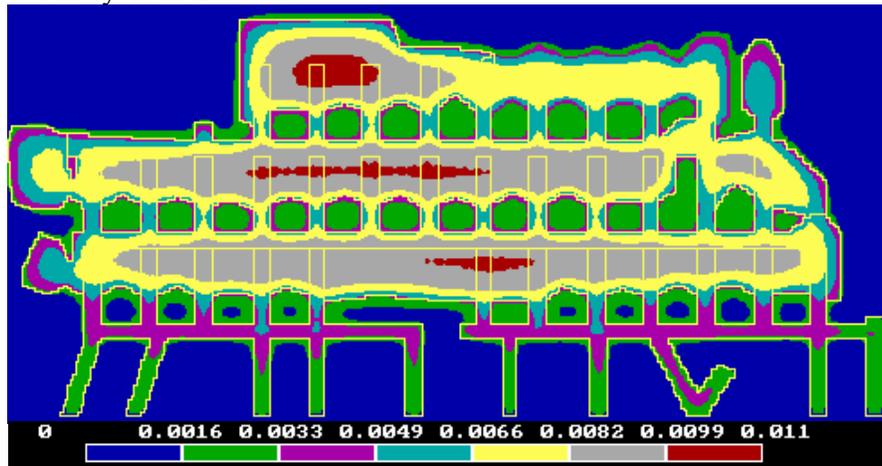
Figure 8.4: Normal Stress [MPa] in Pillars at 60 Meter Cover Depth Obtained from Analysis in Examine^{TAB}.



Normal stresses at the pillar core peaked at 4.25 MPa for mid-interior pillars. The results illustrate the stress-shielding effects of barrier pillars and oversized pillars. No overstressing of any pillars was indicated in the analysis.

Total roof displacement in the mining panel ranged between 6.6 and 11 mm for most of the exposed roof. Roof displacements are influenced by open roof span. It is important to note that these results are irrespective of rock discontinuities, which can strongly influence rock displacements and pillar stresses, depending on the character of the discontinuities. No indication of critical ground movement was indicated in the analysis.

Figure 8.5: Total Displacement in Meters at 60 Meter Cover Depth Obtained from Analysis in Examine^{TAB}.



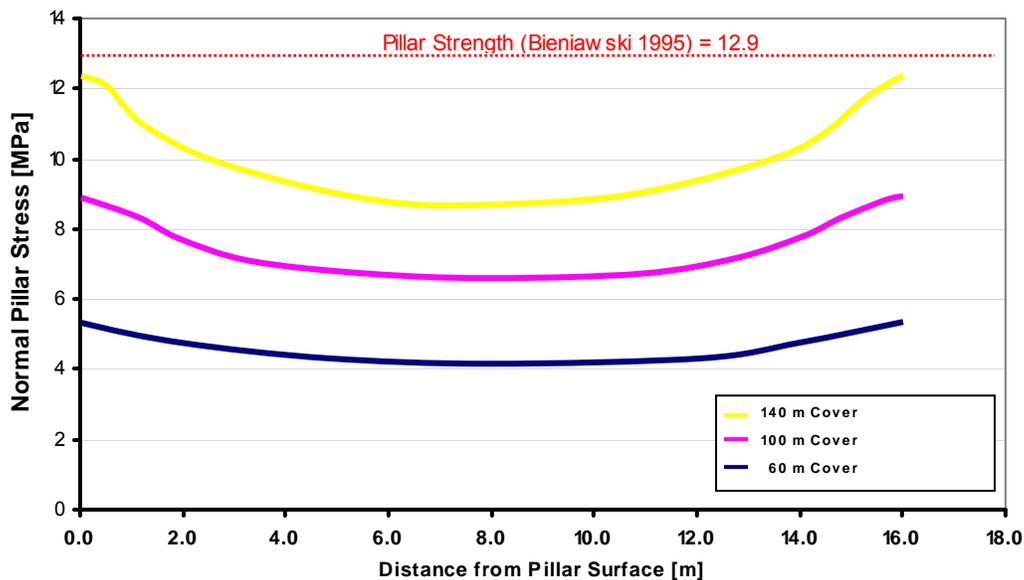
Normal pillar stresses were calculated for Pillar B (see Figure 8.4) to develop stress profiles for overburden depths of 60, 100 and 140 meters. Figure 8.6 shows the results of the analysis.

The results indicate elastic pillar stress levels below the pillar strength of pillar B for all cover depths up to 140 m. The primary range of roof deflection for cover depth 140 m was 19 to 24 mm. The safety factor for pillar B at 140 meters depth was 1.5

Reports from underground mine personnel (Morely et al., 2008) on the efficacy of the in-line pillar mining method were mixed. S. Macdonald commented that caving was unpredictable and recalled an incident where unexpectedly, a large slab of sandstone roof

had completely blocked off the roadway. He added that the complexity of the sandstone roof caving behavior was a motivation to use a non-caving mining method. K. Morely commented in-line pillaring was a successful mining method with little complications. N. Johnson noted that policeman posts, commonly used in depillaring as a tool to indicate roof to floor convergence, was ineffective at 4 South mine.

Figure 8.6: Pillar Stress vs. Distance from Pillar Surface for Pillar B, North ILP Mining Section – 4 South Mine.



The overmining of final pillars in the north ILP mining panel did not result in any serious injuries; however, at the increased depths of cover indicated for future mining panels at 4 South mine, the consequences of overmining panel pillars also increase. This fact coupled with the dangers brought on by roof discontinuities in the sandstone roof and water-based deterioration of sandstone mine roof support the change to a non-caving, partial pillar mining method.

In summary, in-line pillar mining, as practiced in the north mining panel at 4 South, has been shown to pass the stress/strain analysis, yet fail the compliance test. Operationally, unexpected caving events diminished trust in the pillar extraction plan and prompted the transition to a non-caving mining method where greater assurance of roof stability could be provided. One of the main drawbacks of the in-line pillar mining method was the

creation of continuous caving lines across the mining section which were uninterrupted by pillars. The complexity of in-line pillar mining in practice is also not amenable to the current, young and inexperienced workforce at Quinsam Coal. A significant amount of in-field and technical training would be required to revisit in-line pillar mining at 4 South. Moreover, a quality control policy would have to be implemented to ensure minimum pillar sizes were respected and roof displacements were monitored.

8.2 CHECKERBOARD PARTIAL PILLARING (CBPP)

The checkerboard partial pillaring method is one of many partial pillaring methods including pillar splitting, split-and-tee and three-cut. A common characteristic of this mining method is the provision of a substantial amount of coal in the remnant fenders to achieve a minimal risk of roof caving. CBPP was applied to two mining panels north of No.1 Mains: A six-roadway panel spanning cross-cuts 16 to 18 and a five-roadway panel spanning cross-cuts 19 to 21. At 4 South mine, the CBPP layout was designed for a mining height of 4 meters and depth of cover of 100 meters. CBPP involves developing coal pillars 16 x 30 meters in plan with six meter roadways. During pillar extraction, angled cuts are made into panel pillars such that final coal pillars are a minimum of 16 meters diameter and oppose each other diagonally in plan. Pillar extraction is conducted from right to left, row by row in retreat to the Mains with provisions made for slab cuts and fan-outs where safety permits. The theoretical extraction ratio is 66.3%. Figure 8.7 illustrates the CBPP mining method as practiced at 4 South mine.

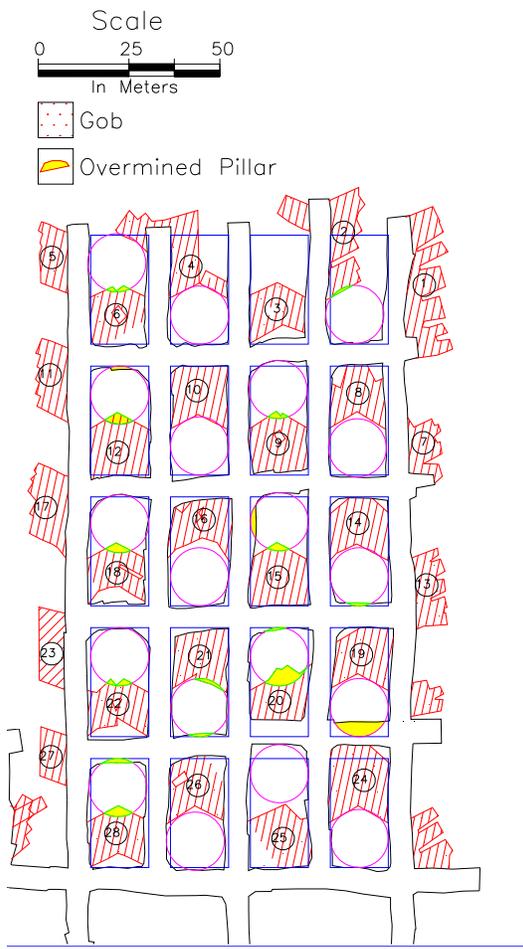
The location and angle of pillar cut in the CBPP method is governed by equation 8.1. Final coal pillars are designed to have a minimum diameter of 16 meters in plan.

The CBPP method counteracts the tendency of the gob to form in-line across the mining section by offsetting final pillar locations in the mining panel and consequently, promotes roof stability and reduces the risk of unexpected caving events. Changes in the location and depth of cuts are allowed when such action augments roof and pillar stability. There are currently no known records of caving events in panels mined using the CBPP method.

8.2.2 Numerical Modeling Analysis

Examine^{TAB} was used to determine the normal pillar stresses and roof displacements in the final coal pillars for the CBPP mining section. Figure 8.9 below illustrates the model setup and the location of pillar ‘A’ which was used to develop a pillar stress profile. Pillar fenders and stumps were not modeled. The model and orientation properties used for this analysis can be referenced in Table 8.3.

Figure 8.8: Checkerboard Partial Pillar Extraction Panel with Pillar Extraction Sequence and Degree of Pillar Overmining Illustrated.



Distribution of Final Pillar Diameter

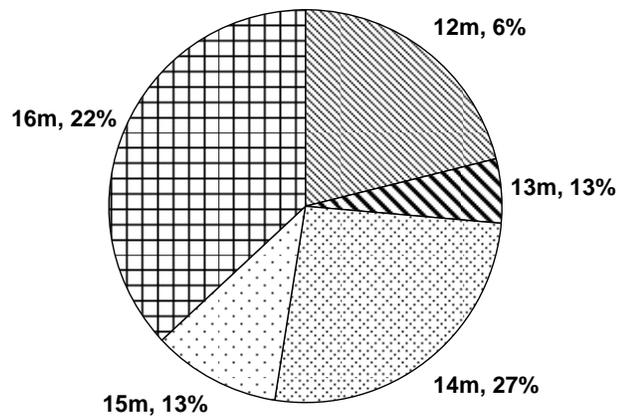


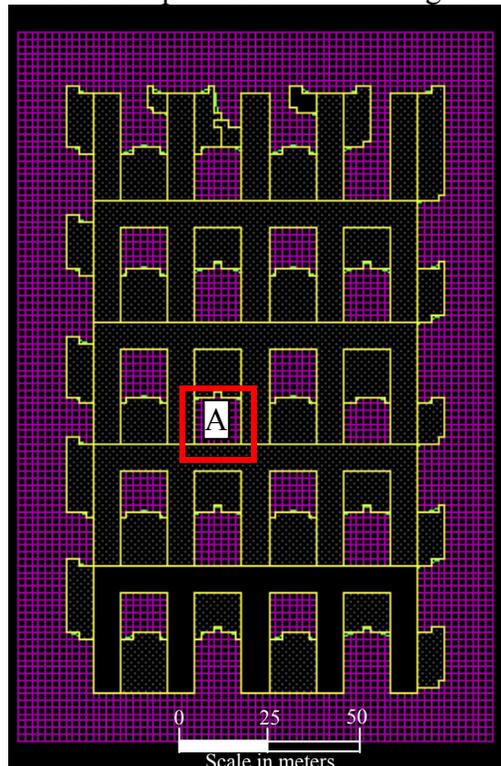
Figure 8.10 shows the calculated normal stress in pillars at 140 meters depth of cover.

Figure 8.11 shows total roof displacements for the CBPP mining panel. Figure 8.12

shows the normal stress boundary as a function of distance from the pillar edge for pillar

‘A’ in the model. Core, rib and pillar edge normal stresses for mid-interior pillars were 9.0, 12.0 and 13.5 MPa respectively. The normal stress profile for CBPP pillars was similar to that determined for the north ILP mining section. Maximum roof displacement ranged from 19 to 24 mm, also comparable to the north ILP analysis results. Roof displacement is within the elastic range of roof bolts and is not expected to result in roof instability.

Figure 8.9: Examine^{TAB} Model Setup – North ILP Mining Panel, 4 South Mine.

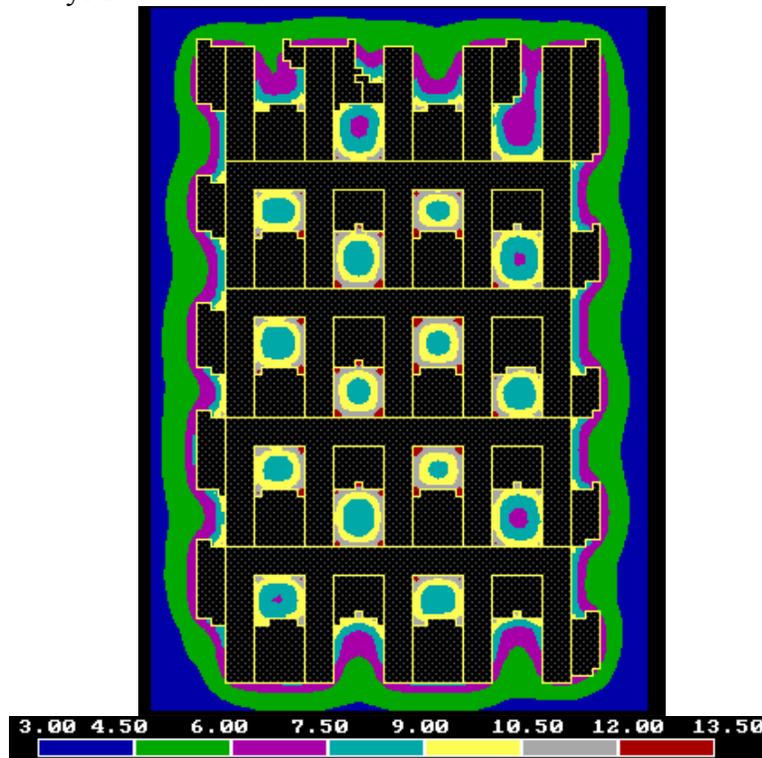


In summary, higher safety factors are achieved in the CBPP mining section due to statistically greater compliance on sizing of final coal pillars. The offset layout of final coal pillars promotes roof stability which provides greater assurance of achieving mine production goals. These factors make CBPP more amenable to the younger and inexperienced workforce at Quinsam Coal. As a non-caving mining method, CBPP is well suited to massive sandstone roof conditions when cavability is poor and difficult to predict. An optimized checkerboard partial pillar extraction method is presented in the next section.

8.3 OPTIMIZED PANEL PILLAR EXTRACTION METHOD

The checkerboard partial pillar extraction method effectively address the complex caving mechanics of massive sandstone roof, while providing opportunities to management environmentally sensitive CCR using underground stowing in the mined-out, free-standing panels. Moreover, its simple design allows for consideration developing additional roadways above the 5-roadway standard to increase operational flexibility and improve coal recovery.

Figure 8.10: Normal Stress [MPa] in Pillars at 140 Meter Cover Depth Obtained from Analysis in Examine^{TAB}.



The optimized CBPP mining method is designed for an extraction height of 4 meters. Pillars are extended two meters to 16 by 32 meters in dimension. Six meter roadways are recommended and a full cutting width (3.5 m versus 3.2 m) of the Continuous miner is planned for during pillar extraction. Final stump fenders are also sized slightly larger for increased intersection stability, yet sufficiently slender to reflect roof convergence of the massive sandstone roof. The safety factor for final coal pillars at 140 m depth of cover is 1.68 and 2.36 at the average depth of cover of 100 meters.

Figure 8.11: Total Displacement in Meters at 140 Meter Cover Depth Obtained from Analysis in Examine^{TAB}.

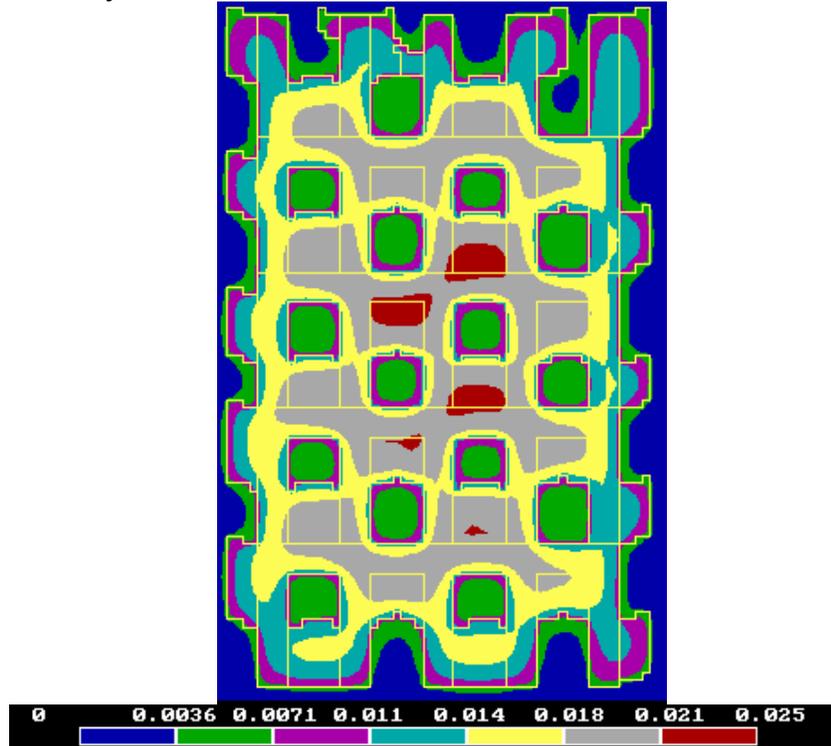
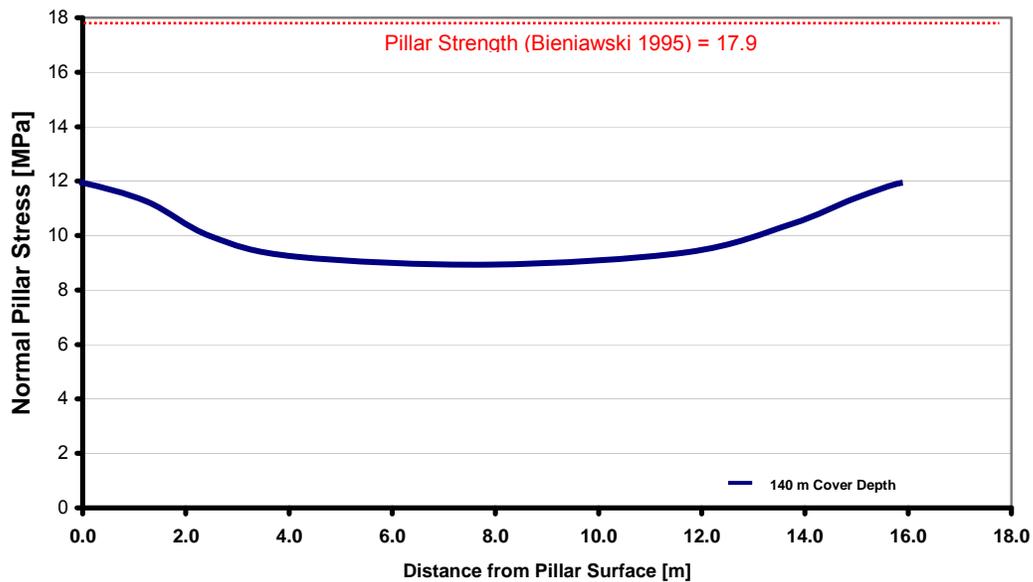


Figure 8.12: Pillar Stress vs. Distance from Pillar Surface for Pillar A, CBPP Section at 140 m Depth of Cover – 4 South Mine.

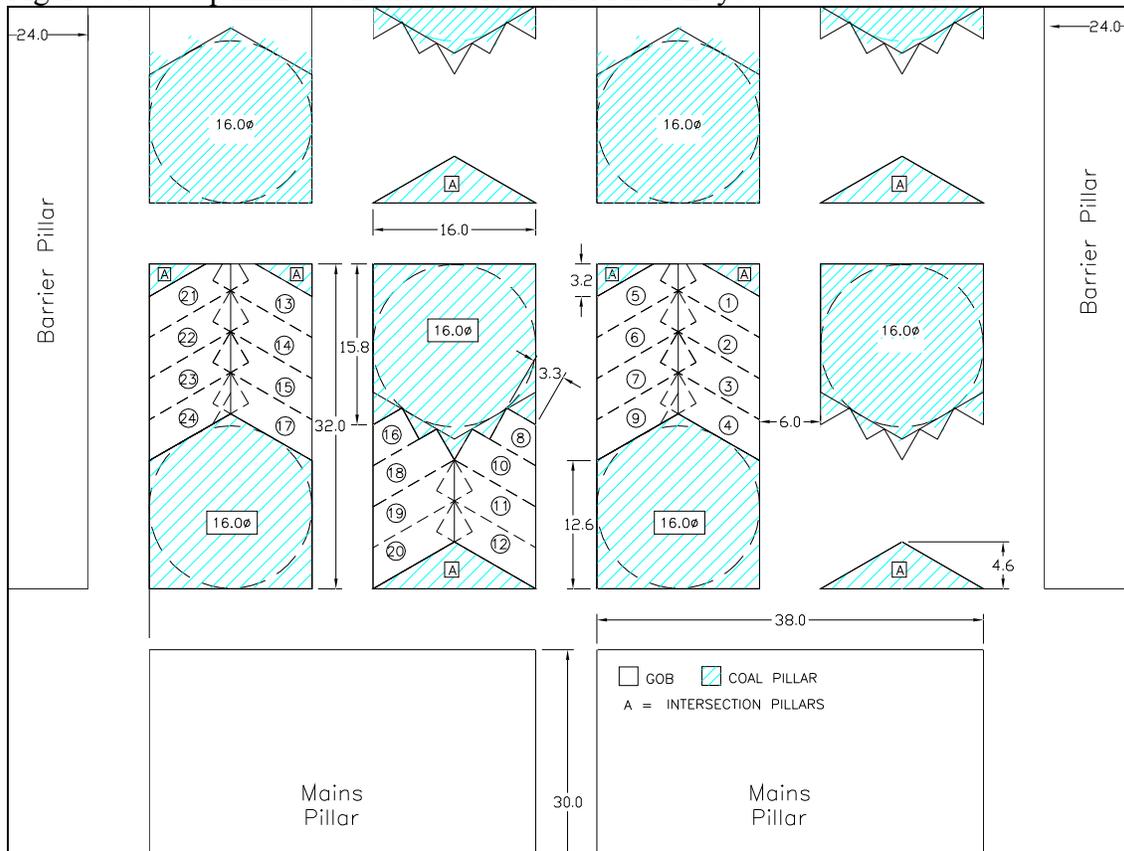


This provides an adequate safety margin against undersized development final pillars in mining sections. The pillar extraction sequence will progress from right to left across each cross-cut in retreat. Barrier pillars are to be a minimum of 24 meters wide. Slab-cuts

into the barrier pillar are be evaluated based on the conditions of the roof, local structure and roadways. Slab-cuts are only be taken adjacent to final coal pillars and be limited in depth such that the barrier pillar width is at least 16 meters or greater. Figure 8.13 illustrates the optimized CBPP layout.

The layout can easily accommodate 7-roadways or more, particularly if Jeffrey ram cars are available for use in future mining endeavors at 4 South mine. The CBPP mining extraction method is well matched to the tolerance for risk and value of the reserve. The simple and practical layout of the CBPP method is amenable to the relatively young an inexperienced workforce that Quinsam Coal currently retains. It follows that training programs would be tailored to the CBPP method and explain the technical reasons for the mine design.

Figure 8.13: Optimized Checkerboard Partial Pillar Layout – 4 South Mine.



8.4 CONCLUSIONS AND RECOMMENDATIONS

In this section, the in-line pillar and checkerboard partial pillar extraction methods have been back-analyzed using a compliance-to-design analysis and numerical stress/strain modeling analysis as measures of safety assurance. As a conclusion to the design methodology illustrated in this thesis, a non-caving mining method has been shown to be highly amenable to massive sandstone roof conditions. In the case of the 4 South mine, a checkerboard partial pillar mining method has been presented, optimized to maximize the operational efficiency of Continuous mining machine and address the geomechanical conditions forecasted for the remaining mining reserve. The chief conclusions of this chapter are:

- A. For the north in-line pillar mining section spanning cross-cuts 6 to 14 off of No.1 Mains:
1. Final coal pillars were 6% compliant to the design size of 16 meters diameter with 42% of pillars being overmined to a diameter of 12 meters or less and the most severely overmined pillars tending to be grouped in-line.
 2. Overmining of pillars reduced the safety factor of mid-interior panel pillars to near unity as determined by Tributary theory.
 3. Average roof spans were 22 to 24 meters, with a 40-meter span being opened 15 meters at the northwest extent of cross-cuts 7A, 8 and 8A.
 4. Normal stress for final coal pillars averaged 9.5 at the pillar core at 140 m depth of cover and 6.2 MPa at 100 m depth of cover with total roof displacements ranging from 19 to 24 mm and 13 to 17 mm at 140 and 100 m depth of cover respectively. Neither pillar overstressing, nor excessive roof deflection was indicated by the modeling analysis.
 5. Unpredictable caving events were noted by underground miners working in the area, while others stated safe and workable mining conditions overall.
 6. Overmining of final coal pillars to the degree documented would impose a significant safety risk at depths of 100 meters and above.
 7. The ILP mining method fails to counteract the tendency of the gob to form in-line as a result of the layout of coal pillars.

8. The greater risk of roof caving imposes greater training and experience demands on the younger, inexperienced component of the miner workforce at Quinsam Coal, and thereby requires more direct supervision.
- B.** For the checkerboard partial pillar mining section spanning cross-cuts 19 to 21 north of No.1 Mains:
1. Final coal pillars were 22% compliant to the design size of 16 meters diameter with 67% of pillars being overmined to a diameter of 14 meters or greater.
 2. Average roof spans were 21 meters in massive sandstone roof.
 3. Normal stress for final coal pillars averaged 9.5 at the pillar core at 140 m depth of cover with total roof displacements ranging from 19 to 24 mm, similar to the results for the north ILP mining panel as determined by Examine^{TAB}. Neither pillar overstressing, nor excessive roof deflection was indicated by the modeling analysis.
 4. Safe and workable mining conditions were noted by underground miners using the CBPP mining method.
- C.** For the optimized checkerboard partial pillar mining method proposed:
1. Pillars should be lengthened by 2 meters to develop pillars 16 by 32 meters in plan.
 2. 3.5 meter cuts should be taken by the Continuous Mining machine during pillar extraction as per the sequence described in Figure 7.13.
 3. Barrier pillars should have a minimum width of 24 meters during panel development. Slab-cuts should be reviewed on a case by case basis, occur only adjacent to final coal pillars, and be limited in depth such that the barrier pillar width is not reduced below 16 meters along its length.
 4. Final coal pillars should have a minimum diameter of 16 meters.
 5. The design factor of safety at 140 meters depth for final coal pillars is 1.68 and 2.36 at the average depth of cover of 100 meters for a 3 meter coal seam thickness.
 6. The mining layout is amenable to the addition of roadways.

7. Simplicity of mining method and design for non-caving is well-suited to the young and inexperienced component of the Quinsam Coal workforce.
8. Training should be conducted to inform all employees working in 4 South of the technical reasons for the mine design.

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS

9.0 INTRODUCTION

This thesis aims to be a logical design methodology for coal mine extraction optimization under competent sandstone mine roof, illustrated by a comprehensive investigation and optimization study of the Quinsam Coal 4 South mine, a shallow underground room and pillar mine with a massive sandstone roof. This research is intended to guide Quinsam Coal and other coal mines globally in efforts to develop or optimize coal extraction and address the geomechanical challenges presented by massive sandstone roof. The key aspects and tools required to facilitate effective site characterization, ground support design, excavation stability, pillar design, environmental risk management and mining method optimization have been presented, as part of the design methodology.

The field work for this thesis was conducted at the Quinsam Coal mine site on Vancouver Island between May 2002 and December 2005 and in July 2003. Rock mechanical properties test work and fabrication of PVC extensometers for this study was conducted at the Geomechanics laboratory at the University of British Columbia.

The objective of this thesis was to advance the design methods used to develop ground support systems, stable excavation spans and coal pillars and optimized coal extraction methods at shallow underground coal mines characterized by a massive sandstone roof. Current design methodologies do not give adequate treatment of sandstone roof geology in the design of safe mining conditions for coal extraction, where significant engineering design input is required to address the complexities of this category of mine roof behavior. The importance of environmental risk management is also not given adequate treatment in design methodologies at a time when neglect of this issue can severely undermine the credibility of a new project proposal. This thesis critiqued available design methods. It defined the geomechanical and environmental considerations important to design and then applied analytical, empirical and numerical methods to support the final design of a ground support system, coal pillars and pillar extraction method for the 4 South mine. It is hoped that this research will be particularly valuable to the Quinsam

mine in its efforts to develop future coal mines with massive sandstone roof conditions. Furthermore, this thesis is expected to benefit coal mine projects globally where similar geotechnical, geological and environmental risks are present, yet not adequately addressed with available tools and references.

This chapter will review the important conclusions and recommendations as they relate to the six study questions outlined in chapter 1. Contributions to the advancement of the state-of-the-art are also presented, followed by recommendations for future work.

9.1 SITE CHARACTERIZATION

Site characterization is the first part of the design methodology illustrated in this thesis. Site characterization is required to address unique geological, geotechnical and environmentally sensitive issues influencing mine design for coal mine extraction under massive sandstone roof. Site characterization data is used to guide the selection of a ground support system, coal pillar design and coal pillar extraction method.

The 4 South mine is intersected by a series of major faults, typically bearing water that offset the No.3 coal seam up to 10 meters and promote local deterioration of the sandstone roof and to a lesser extent, coal pillar ribs. The dominant fault and joint sets mapped in the sandstone roof strike North/South and North-east/ South-west, and dip between 21 and 54°. The sandstone roof is otherwise massive and very competent.

The 4 South mine is moderately wet, due in part to the water bearing joints and faults that intersect the exposed mine roof. A water joint survey conducted in the fall of 2005 at the 4 South mine found an average flow of 1.0 USGPM from water-bearing joints. It is expected flow would be substantially reduced in the dry season. Ground water flow was intercepted during exploration drilling on the 4 South property. Wet conditions are forecasted for the remaining mining reserve.

Water has been shown to reduce the average unconfined compressive strength, Young's modulus, and Poisson's' ratio of saturated specimens relative to dry specimens by 71, 75

and 17% respectively. Where wet roof conditions are encountered, roof stability monitoring is highly recommended, either visual or with the use of instrumentation, as well as an assessment of cablebolt and weldmesh supplementary support installation.

A Geomechanics rockmass rating (1976) survey for the 4 South mine found the sandstone roof and No.3 coal seam averaged 70% and 35% respectively. The data strongly suggest similar conditions will be encountered in remaining mining reserve. These values are recommended for design. Table 3.9 summarizes the design elastic, strength and physical rock mechanical properties for the 4 South sandstone, mudstone/siltstone and No.3 coal seam. Rock mechanical testing should be performed on the basal mudstone/siltstone and No.3 coal seam since indirect estimation of rock mechanical properties was used for these geologic units.

Hydrostatic stress conditions are strongly supported by literature references and underground observations in the 4 South mine. In-situ stress measurements are recommended for the 4 South mine before the mine goes back into production. The data from these tests should be validated and used for design.

Pre-mining horizontal and vertical stresses will range from 1.7 to 3.7 MPa based on an overburden cover range of 66 to 141 meters.

9.2 GROUND SUPPORT STRATEGY

The second part of the design methodology illustrated in this thesis involves developing a ground support strategy to complement massive sandstone roof conditions. Ground support selection directly affects mining safety, productivity, and economics. A work plan to determine an optimized ground support strategy has been presented that consists of determining an appropriate rock bolting support theory rock support design method to guide ground support selection, considering the advantages and disadvantages of the candidate rock bolt designs relative to the mine conditions anticipated or encountered and evaluating the capacity of the ground support systems to address the geomechanical conditions of the prospective mine.

Elastic beam theory is the best rock bolting support theory for the 4 South mine sandstone roof where the continuity of the roof is not interrupted by major geologic structure. In such instances, a more complex review of Voussoir beam theory or numerical modeling is strongly recommended.

The 4 South massive sandstone roof is strongly self-supporting at development spans of 6 meters and intersection spans of 8.5 meters in the absence of major geologic roof structure. A precautionary ground support role is implied by these conditions, where induced stresses and strains do not exceed the capacity of the ground support in active mining areas.

Intersection wedges have been shown to represent the primary roof hazard at the 4 South mine. The Rocscience software *Unwedge* is highly recommended for analyzing intersection wedges where the roof discontinuities defining the wedge are known. AutoCAD pyramid idealizations of intersection wedges have been shown to be best suited to massive sandstone roof in the evaluation of roof bolt anchorage capacity and safety factors against gravity failure. Joints inclined greater than 18° should be reinforced with cable bolts irrespective of location based on a critical wedge analysis.

Analytical support predictions are the most applicable reference to guide support selection. Empirical estimates of support requirement for the 4 South sandstone roof suggest a conservative range of rock bolt lengths and pattern support spacing and a greater support load density and anchorage depth that has successfully been provided by the current installed ground support. The Unal (1983) roof support recommendation for the 4 South mine, consisting of 1.5-meter long #6 – point-anchored rebar or 11-tonne capacity mechanical rock bolts on 1.5 meter centers, is regarded as the most practical ground support recommendation for the 4 South massive sandstone roof. Excellent bond strength capacity is indicated for the massive sandstone roof at 4 South mine. Short-encapsulation pull tests on 1.8-meter torque-tension rebar indicated an average bond strength of 45 tonnes/meter at an average yield load of 15.5 tonnes. A design value of 45 tonnes/meter is recommended for the bond strength for resin rebar bolts.

Twenty-eight standard rock bolt pull tests were completed for this study on #6 forged-head point-anchor and fully grouted bolts of lengths 1.2, 1.5 and 1.8 meters. Five short-encapsulation pull tests were also completed. Twenty rock bolt pull tests completed in 1997 were referenced for the ground support optimization study: three standard pull tests on 1.8-meter long, Grade 60, #6 point-anchor resin rebar, four on 1.8-meter long, C1060 Grade #5 mechanical bolts and 13 on C1060 Grade #5 mechanical bolts with 1" FIF expansion shells.

Corrosion of rock bolts was shown to not be of concern in affecting the capacity of the installed rock bolts. Rock bolts and plates should be inspected for corrosion damage at least every 12 years based on these results, or sooner if accelerated corrosion is indicated.

Two ground support systems are recommended for application in the 4 South mine. The Type 1A minimum standard consists of 1.5 m long Grade 60 #6 fully-grouted forged-head resin rebar on 1.5 m center spacing. Plates will consist of 3/8" Grade 30 flat plates. No weld-mesh screen is prescribed. A yield load of 15 tonnes at 10 mm displacement should be used for design. The Type 1B minimum standard consists of 1.8 m long C1060 grade mechanical bolts with hardened steel washers and #5 RH FIF Frazer & Jones expansion shells or equivalent. Plates will consist of 3/8" Grade 30 flat plates. No weld-mesh screen is prescribed. A yield load of 8.5 tonnes at 100 mm displacement should be used for design. Further pull test and anchorage performance tests are recommended on Type 1B support before adoption in the 4 South mine. A protocol for torque-testing will be required. The recommendations quality assurance of mechanical bolts presented by Mraz (1997) should be followed until a site-specific protocol is developed.

Supplementary support consisting of No.9 gauge screen for surface spalling control, 4 m long passive cablebolts for wedge support and timber cribs and posts for rib and roof support should be reviewed for inclusion on a case by case basis or according to design rules.

9.3 PILLAR DESIGN

The third part of the design methodology is pillar design. A key aspect of pillar design requires determining the in-situ pillar strength through careful consideration of the effects

of pillar discontinuities and temporal changes to pillar stress, size and shape. Pillar loading should be reviewed using Tributary area theory (TAT) particularly when irregular pillar layouts are used. Where available, Examine^{TAB}, or similar three-dimensional, or pseudo-3D software should be used to evaluate pillar normal stresses, understanding that TAT will tend overestimate core pillar loads. The NIOSH software *Analysis of Retreat Mining Pillar Stability* (ARMPS) is recommended where pillar layouts are regular and/or caving extraction is planned.

In the case of the 4 South mine, the Bieniawski (1983) pillar strength formula was used in conjunction with TAT to estimate coal pillar strength. The formula is valid for the range of pillar width:height ratios expected and adequately models the rapid increase in pillar strength with increasing width:height ratios. Estimations of jointing effect on pillar strength correlate well with the design in-situ strength value of 6.2 MPa for the No.3 coal seam adopted for this thesis.

The 4 South mine coal pillars observed in the mine were unaffected by skin deterioration, except where major geologic structure was encountered. In such cases, rib slough depth ranged from 0.3 to 0.6 meters. Empirical estimations of pillar duty life based on temporal rib-slough suggested a pillar life of 4 years for panel pillars in the 4 South mine; however, observations of pre-existing final coal pillars in the mine suggested a pillar duty life of at least 10 years. Further work is recommended in this area to estimate the pillar life of final coal pillars in the 4 South mine.

The following guidelines are recommended as a minimum for pillar sizing in the 4 South mine:

Panel Pillars - width:height ratio: 4, Safety factor: 1.5

Final Coal Pillars - width:height ratio: 4, Safety factor: 1.5

Barrier Pillars - width:height ratio: 6, Safety factor: 2.0

Stump Pillars – minimum edge length of 2.5 meters

Pillar extraction plans should be adjusted to provide support to major roof geologic structure.

9.4 EXCAVATION STABILITY

Excavation stability assessment forms part four of the design methodology. Excavation stability has been examined in this thesis based on the cavability and critical span of massive sandstone roof. The role of roof convergence monitoring has been presented as part of a roof stability monitoring program. A unique, intrinsically safe air pump convergence stick has been developed for use in monitoring massive sandstone roof. In general, massive sandstone roof is difficult to cave and promotes the development of large unsupported excavation spans. Although the interaction of the pillar floor and coal pillar was not addressed in this study; it can greatly influence pillar design. Further work on this design methodology should include case study review at operations with soft floor conditions where such interaction would greatly influence the coal pillar extraction design.

The poor cavability of the sandstone roof at 4 South mine has been shown to limit the use of full-extraction retreat and partial pillar mining caving methods. Experience in the Indian coal fields where massive sandstone roof dominates roof composition by mine, suggests very poor cavability potential at 4 South. Statements by underground miners working in the north in-line pillar mining panel also suggest poor cavability and support partial pillar non-caving mining methods at 4 South mine. Further work is recommended in this area to evaluate full-extraction mining methods under massive sandstone strata at shallow depth. Advanced numerical modeling is recommended for analysis work.

The critical span of the 4 South mine roof was estimated through analysis tools in the technical literature, both empirically and analytically derived. Massive sandstone roof has been shown to bridge spans of up to 40 meters at the 4 South mine. The literature indicates that spans up to 70 meters may be possible in massive sandstone strata. Further work is required to evaluate the critical span of massive sandstone roof at underground coal mines. Advanced numerical modeling is recommended for this work.

Test trials of a roof convergence instrument fabricated for this study for application at 4 South mine are inconclusive. Roof convergence monitoring has important application in roof stability monitoring, particularly during pillaring operations where roof convergence rates and magnitudes are typically greatest. No geotechnical instrumentation has been used at 4 South to date. Electric ground movement monitors are recommended for trials provided an intrinsically safe model can be approved for use in the 4 South coal mine. In-field trials of the air-pump convergence sticks at the 4 South mine are recommended as well.

Empirical relationships to determine the critical and maximum convergence velocity for underground roadways, developed from experience in Indian coalfields, suggest a critical and maximum convergence velocity of 0.23 mm/day respectively for 4 South mine roadways. A critical convergence velocity of 1 mm/day is recommended for the massive sandstone roof at 4 South mine. Geotechnical instrumentation is invaluable for determining safe convergence rates for intersections and roadways. Further work is recommended in this area as well.

9.5 ENVIRONMENTAL RISK MANAGEMENT

Environmental risk management represents part five of the design methodology. Review of mine water discharge and mine waste quality and quantity is required as a minimum with respect to the mining method and coal extraction plan.

At 4 South mine, mining activity contributes to the dramatic increase in sulphate levels in the Long Lake receiving environment, where the British Columbia water quality guideline for sulphate (100 mg/L) have been exceeded at depth. Non-caving mining methods at 4 South mine is strongly suggested to minimize the total amount of water managed annually by limiting fracturing of the sandstone roof and minimizing exposure to fracture flow paths in the porous sandstone. Fracturing of the sandstone roof is expected to increase sulphide oxidation and increase sulphate concentrations in the mine water emanating from fractures in the roof.

Waste management has been shown to be important to the viability of the 4 South mine, particularly when coarse coal reject material is produced from ROM coal washing. Coarse coal reject produced from coal washing is strongly potentially acid generating (PAG) based on acid-base accounting testing of the material. The chemical character of 4 South CCR shows levels of arsenic, cadmium, chromium, cobalt and copper exceeding the British Columbia Contaminated Sites Regulation (CSR) for freshwater, even with a 50/50 mix of non-acid generating coarse coal rejects from the 2 North/ 3 North mine. The test results and data require 4 South coarse coal rejects to be disposed of sub-aqueously in the 3 South pit – a limited capacity repository for PAG coarse coal reject that is highly valued by Quinsam Coal Corporation and remains the only permitted site for PAG coarse coal reject disposal.

Non-caving pillar extraction methods are strongly believed to reduce waste management costs and risks by providing options for underground storage of PAG materials. Further work is required to determine the viability of underground storage of PAG coarse coal rejects at 4 South mine and other mines around the world.

9.6 MINING METHOD OPTIMIZATION

The final part of the design methodology involves developing coal extraction method under massive sandstone roof. Non-caving pillar extraction mining has been shown to minimize environmental risk and promote stability of the massive sandstone roof. In-line pillar mining is not well suited to massive sandstone conditions in the example of the 4 South mine. Checkerboard partial pillar mining is recommended for mining under massive sandstone roof

At 4 South mine, the north in-line pillar mining section north of cross cuts 6 to 14 north off No.1 Mains was 94% non-compliant on final pillar size, design being 16 meters minimum diameter. Forty-two percent of final pillars were 12 meter diameter or less with overmined final pillars tending to be grouped together in the mining section. TAT analysis found overmining reduced final pillar safety factor to near unity in some cases.

Average roof spans of 23 meters were opened with caving of the roof occurring unpredictably. An extraction of 67.8% was achieved.

No overstressing of final coal pillars or excessive roof strains were suggested by an Examine^{TAB} analysis of the north in-line pillar mining section. Normal stress in final coal pillars averaged 9.5 at the pillar core at 140 m depth of cover and 6.2 MPa at 100 m depth of cover with total roof displacements ranging from 19 to 24 mm and 13 to 17 mm at 140 and 100 meters respectively. Extensive overmining as documented in this study would result in significant safety risk at 100 meters depth of cover and yielding pillar conditions. The complex and unpredictable roof response during depillaring puts increased training demands on the younger, inexperienced workforce at Quinsam Coal Corporation and direct supervisors of this workforce. It is not recommended for application at 4 South mine.

Checkerboard partial pillar mining has been successfully applied as a six-roadway mining panel and five-roadway mining panel. Average extraction in both panels was 66.3%, 1.5% less than extraction achieved using in-line pillar extraction methods. Twenty-two percent of final pillars were compliant to design and 67% of all final pillars were 14 meters diameter or greater. The sandstone roof did not cave in checkerboard partial pillar mining sections with average roof spans of 21 meters. Neither pillar overstressing, nor excessive roof deflection was indicated by the Examine^{TAB} analysis.

The optimized partial pillar plan recommended in this thesis contains the following amendments to the current checkerboard partial pillar mining layout:

1. panel pillars should be lengthened to 32 meters,
2. 3.5 meter wide cuts should be taken by the Continuous mining machine
3. barrier pillars should be a minimum of 24 meters wide during panel development with provisions for slab-cuts during depillaring provided the final barrier pillar width is not reduced below 16 meters along its length and cut are only taken adjacent to final coal pillars.

Final coal pillars should be a minimum of 16 meters diameter in plan such that safety factors of 1.68 and 2.36 are provided at a depth of cover of 100 meters and 140 meters respectively. The optimized layout provides a 67% extraction. Developing additional roadways within panels can be implemented without compromising the safety of the mining section.

The checkerboard partial pillar mining method is recommended for all future mining panels at Quinsam Coal Corporation and as a strong consideration at underground coal mines operating under massive sandstone roof around the world. In the case of the 4 South mine, the technical details of the mine design should be incorporated into a training program in the hopeful event the mine goes back into production.

9.7 CONTRIBUTIONS TO THE STATE-OF-THE-ART

This thesis provides a comprehensive methodology for developing a safe coal pillar extraction method under massive sandstone roof, particularly at shallow depth, addressing the important aspects of modern design: site characterization, ground support selection, pillar design, excavation stability assessment, environmental risk management and mining method optimization.

The site characterization work augments the global dataset of strength and elastic rock mechanical properties, fault attitude, for sedimentary rock units on Vancouver Island. Data has also been presented to further the understanding of the adverse effects of water on rock mechanical properties of sandstone. The use of Fiberscope stratascope roof investigations in support of evaluating the condition of the immediate roof has been demonstrated as a safety measure in design work. Engineers and Geoscientists abroad will find this information valuable in their own design endeavors, particularly where little site data is available.

In ground support design, simple and effective computer aided drafting tools have been presented to evaluate the capacity of a roof support design to support intersection wedges, either idealized or as defined by primary joint and fault sets. Extensive in-situ pull test

data has been presented in support of full-column forged head bolts of lengths 1.2, 1.5 and 1.8 meters. Additionally, short encapsulation pull tests were conducted to advance the understanding of the bond strength achieved with resin-assisted rebar. This data will be valuable globally at underground coal mines with massive sandstone roof evaluating full-column forged head rebar as primary roof support.

Practical and simple tools have been presented in the evaluation of mine roof cavability, drawn primary from experience at the Indian coal mines, where massive sandstone roof is commonplace. Furthermore, evaluative tools for critical span are provided based on plate and beam theory and empirical data to assist in the design of caving and non-caving coal extraction methods with massive sandstone roof where large unsupported spans are possible. The monitoring of roof stability in underground coal mines has been advanced with the development of an intrinsically safe and inexpensive convergence rod as developed and tested. This work has worldwide application.

The presentation of the sulphate response to underground mining activity in the water effluent and receiving environment of the water management system advances the environmental considerations in coal mine design, especially in sensitive environments. Moreover, this thesis provides a review of the chemical makeup of coarse coal reject waste that has been shown in the case of the 4 South mine, to have profound implications on mining method and ROM coal processing. This information also advances the environmental aspects of coal mine design, giving in worldwide application and relevance.

Finally, this thesis advances the understanding of the stress-strain response of coal pillars in in-line pillar and checkerboard partial depillar mining. The shortcomings and advantages of both mining methods as presented advance safety and design, all of which has worldwide application.

9.8 RECOMMENDATIONS FOR FURTHER WORK

The research in this thesis represents potential building blocks for developing a comprehensive design guideline for coal extraction under a massive sandstone roof with global application. Empirical studies of operating underground coal mines at shallow depth with massive sandstone roof are recommended to improve the understanding of sandstone roof behavior. Recommendations for future work applicable to operating underground coal mines with massive sandstone roof conditions follows:

Site Characterization: A database of rock mechanical properties and rock mass ratings should be collected. This information should also be related to the average depth of cover the mines are operating under and pre-mining stress conditions. Information on the effects of water in the roof and on rock mechanical property testing should also be collected.

Ground Support: A database on ground support systems, both primary and secondary, should be collected with as much detail as possible, including support pattern, length and type of bolts, drill hole diameter and length, drill bit type and consumption, etc. This information should be augmented with records of ground falls, including location, size and weight, and installed ground support.

Pillar Design: A database of pillar stand-up times relative to dimensions and pillar loading should be collected and reviewed for correlations. Guidelines for pillar design with soft floor conditions should be reviewed and considered for inclusion in this design methodology.

Excavation Stability: Data should be collected to support the development of a critical span curve for a massive sandstone roof. Advanced numerical modeling studies are likely required for this work. Further review of a standard index for cavability is warranted, followed by the development of a cavability guideline for massive sandstone roof conditions. Finally, the use of roof convergence instrumentation should be reviewed and

compiled as a reference database with focus on the advantages and disadvantages of prospective instruments and their limitations.

Environmental Management: A review of underground disposal options of PAG coal waste is strongly recommended. It has been shown in this thesis that surface disposal options may be limited.

Mining Method Design: A more global review of mining methods practiced, particularly when caving is designed for, would be extremely valuable. This database would have to be related to the information obtained in the site characterization part of the design methodology.

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