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Proceedings of the Second International Workshop on Coal Pillar Mechanics and Design

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UNIT OF MEASURE ABBREVIATIONS USED IN THIS REPORT

ft	foot (feet)	kPa/m	kilopascal per meter
GPa	gigapascal	m	meter
ha	hectare	mm	millimeter
hr	hour	MN/m	meganewton per meter
in	inch	MN/m^3	meganewton per cubic meter
in/in	inch per inch	MPa	megapascal
kg/cm^2	kilogram per square centimeter	psi	pound (force) per square inch
kg/m^3	kilogram per cubic meter	psi/ft	pound (force) per square inch per foot
km	kilometer	sec	second
km^2	square kilometer	%	percent
kN/m^3	kilonewton per cubic meter	°	degree

PROCEEDINGS OF THE SECOND INTERNATIONAL WORKSHOP ON COAL PILLAR MECHANICS AND DESIGN

**Edited by Christopher Mark, Ph.D.,[1] Keith A. Heasley, Ph.D.,[1]
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ABSTRACT

Pillar design is the first line of defense against rock falls—the greatest single safety hazard faced by underground coal miners in the United States and abroad. To help advance the state of the art in this fundamental mining science, the National Institute for Occupational Safety and Health organized the Second International Workshop on Coal Pillar Mechanics and Design. The workshop was held in Vail, CO, on June 6, 1999, in association with the 37th U.S. Rock Mechanics Symposium. The proceedings include 15 papers from leading ground control specialists in the United States, Canada, Australia, the United Kingdom, and the Republic of South Africa. The papers address the entire range of issues associated with coal pillars and have a decidedly practical flavor. Topics include numerical modeling, empirical design formulas based on case histories, field measurements, and postfailure mechanics.

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INTRODUCTION

By Christopher Mark, Ph.D.[1]

Pillar design is one of the oldest and most fundamental of the mining sciences. Without pillars to support the great weight of the overburden, underground coal mining would be practically impossible. Coal pillars are employed in a wide variety of mining operations, from shallow room-and-pillar mines to deep longwall mines. Yet despite more than 100 years of research and experience, pillar failures continue to occur, placing miners' lives at risk. Some recent examples are [Mark et al. 1998]:

Massive collapses: In 1992, miners were splitting pillars at a mine in southern West Virginia when the fenders in a 2.3-ha area suddenly collapsed. The miners were knocked to floor by the resulting airblast; 103 ventilation stoppings were destroyed. At least 12 similar events have occurred in recent years in the United States and 15 others in Australia, fortuitously without a fatality.

Pillar squeezes: At a coal mine in Kentucky, pillars were being extracted in the main entries under 270 m of cover. The pillars began to crush in response to the vertical load, resulting in a roof fall that killed two miners. This incident is an extreme example of hazardous conditions that can be associated with slow pillar failure. At least 45 recent instances of pillar squeezes in room-and-pillar mines have been identified.

Longwall tailgate blockages: In 1984, 26 miners at the Wilberg Mine in Utah could not escape a deadly fire because of a tailgate roof fall. Similar blockages were common in the 1980s, and 50 cases have been documented.

Pillar bumps: Extracting the initial lift from a standing pillar at a deep operation in eastern Kentucky resulted in a bump that killed two miners. However, bumps are not confined to pillars; another fatal bump occurred at a longwall face in Utah just days later.

Multiple-seam interactions: Some studies indicate that most remaining coal reserves will experience multiple-seam interactions. At a mine in West Virginia where four seams had been previously extracted, one fatality occurred when the roof collapsed without warning beneath a remnant barrier pillar.

Abandoned mine subsidence: As suburban development expands into historic coal mining areas, unplanned subsidence has become an important issue. In one case, residents above 50-year-old workings were disturbed by seismicity emanating from collapsing pillars. In the Republic of South Africa, collapsing pillars in the Vaal Basin are creating large sinkholes that threaten many homes.

To help reduce the safety hazards of pillar failures, this *Second International Workshop on Coal Pillar Mechanics and Design* was organized. (The first workshop was held in Santa Fe, NM, in 1992.) The proceedings of the second workshop feature 15 invited papers from leading rock mechanics experts in the United States, Australia, the Republic of South Africa, the United Kingdom, and Canada. Mines in these five countries employ increasingly similar methods, including:

- Retreat longwall mining, usually using large chain pillars;
- Room-and-pillar mining with continuous mining machines; and
- Roof bolts for primary roof support.

The similarity of mining methods means that it is easier and more valuable to transfer safety technologies like pillar design from one country to another. Indeed, one of the striking features of these proceedings is the convergence of research results across international borders.

Other trends affecting the mining industries of the five countries are also reflected in these proceedings, some of which have been less positive. In the 7 years since the first workshop, underground production has risen in Australia and the Republic of South Africa, declined in the United Kingdom and Canada, and remained steady in the United States. However, great employment losses have occurred in all five countries because of technological advances and dramatic productivity increases.

One consequence has been a significant decline in institutional support for mining research. Since 1992, the U.S. Bureau of Mines (USBM), the Canada Centre for Mineral and Energy Technology's (CANMET) Coal Research Laboratory, British Coal's Headquarters Technical Division, and the South African Chamber of Mines research department have all closed their doors. Government funding for mining research is now indirect and open for competition everywhere, except in the United States. In the United States, the National Institute for Occupational Safety and Health (NIOSH) has taken up the USBM's traditional mine safety research role, although at a reduced level, and continues to receive direct funding from the U.S. Congress.

University mining departments have also been under pressure due to fluctuating student enrollments, reduced research funding, and a shortage of qualified junior faculty. Lower profit margins and a renewed emphasis on the bottom line has meant that few mining companies now maintain any in-house research capability. As the traditional sources of mining research have faltered, in many cases private consulting firms have taken up the challenge. Often staffed by former government researchers and sometimes supported in part by government contracts, consultants are now often on the cutting edge of research.

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In comparing the proceedings of the second workshop with those of the first [Iannacchione et al. 1992], the most obvious difference is that the current collection of papers is a slimmer volume. There are 15 papers in these proceedings, compared with 23 in 1992. Australia, which in many ways has the healthiest mining research community, is the only country to see its representation increase (see table 1). Although the number of papers from industry, government, and academia all decreased by at least 50%, the number of papers from private consultants more than doubled.

Another consequence of the changed research environment is reflected in the proceedings' pervasive emphasis on practical problem-solving. Although about one-half of the papers at the first workshop addressed issues of a more theoretical nature, nearly every paper in the current collection uses case histories, field measurements, and/or practical experience to develop techniques for solving real-world pillar design problems.

The papers divide almost evenly between those that focus primarily on the application of numerical modeling and those that discuss empirical formulas derived from statistical analysis of case histories (table 1). Of the numerical modelers, two used finite-difference methods (Gale, Cassie et al.), four used boundary elements (Heasley-Chekan, Maleki et al., Zipf, Karabin-Evanto), and one used finite elements (Su-Hasenfus). Field measurements feature prominently in six papers, with Cassie et al., Colwell et al., and Gale monitoring stress and deformation, Heasley-Chekan and Karabin-Evanto mapping underground conditions, and Biswas et al. measuring changes in rock strength.

In general, however, the similarities between the papers are more striking than their dissimilarities despite the variety of countries, author affiliations, and research methods. For example, new empirical formulas are presented for the Republic of South Africa (van der Merwe), the United States (Mark), and Australia (Galvin et al.). Derived independently from different sets of case histories from around the world, the three formulas are within 15% of each other (see figure 1).

Five papers (Su-Hasenfus, Gale, Cassie et al., Mark, and Colwell et al.) explicitly address the design of squat (large width-to-height (w/h) ratio) pillars, primarily for protection of longwall gate entries. All agree that the strength of these pillars can vary widely depending on the roof, floor, and seam parting characteristics. Moreover, the strength of the roof is often just as important to the design process as the strength of the pillar itself. The degree of consensus that has been achieved on this complex topic is an important advance. At the other end of the w/h scale, van der Merwe, Zipf, and Mark address slender pillars and their potential for sudden collapse. Again, all three reach similar conclusions regarding the importance of pillar geometry and postfailure pillar stiffness.

The beginnings of a consensus are also evident in one of the oldest pillar design controversies—the value of compressive strength tests on coal specimens. Only two papers (Karabin-Evanto and Maleki et al.) make use of laboratory tests to evaluate seam strength. On the other hand, van der Merwe, Su-Hasenfus, Cassie et al., Galvin et al., Gale, and Mark all conclude that variations in the uniaxial compressive strength have little effect on the in situ pillar strength.

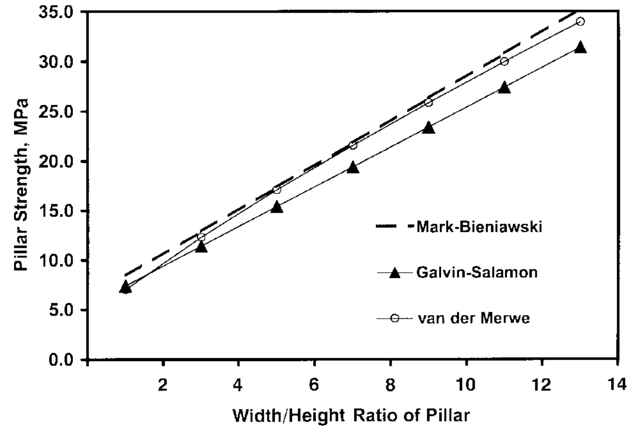

Figure 1.—Empirical pillar strength formulas derived from case histories by Mark (U.S.A.), Galvin (Australia), and van der Merwe (Republic of South Africa).

With the focus on pillar strength, it is important not to overlook the other half of the design equation—the load. Gale and Colwell et al. describe field measurements that shed new light on the loads that occur during longwall mining. Heasley-Chekan and van der Merwe address the effect of overburden behavior on the pillar loading. Kramer et al. have extended their fracture mechanics approach for estimating load distribution to consider the effects of other kinds of supports.

Other special topics that are discussed in these proceedings include the effect of weathering on long-term pillar strength (Biswas et al.), the geologic and geotechnical factors that affect the potential for coal bumps (Maleki et al.), thick-seam room-and-pillar mining (Cain), multiple-seam mine design (Heasley-Chekan), and the strength of rectangular pillars (Galvin et al. and Mark).

One final comparison between the first and second workshops is perhaps in order. The proceedings of the first workshop [Iannacchione et al. 1992] included papers from a number of now retired individuals whose names have been synonymous with pillar design for nearly 3 decades: Salamon, Bieniawski, Wagner, Barron, and Carr. In many ways, their contributions laid the foundation upon which rests much of our current understanding of coal pillars. Their retirement has left a large gap that cannot be filled (although it is hoped that they will continue to contribute to the profession!). To paraphrase Sir Isaac Newton, it is only by standing on the shoulders of such giants that we can hope to achieve further progress.

Table 1.—Summary of papers for the Second International Workshop on Coal Pillar Mechanics and Design

Primary author	Country	Affiliation	Method
Biswas	Australia	University	Empirical.
Cain	Canada	Mining company . .	Empirical.
Cassie	U.K.	Consultant	Numerical.
Colwell	Australia	Consultant	Empirical.
Gale	Australia	Consultant	Numerical.
Galvin	Australia	University	Empirical.
Heasley	U.S.A.	Government	Numerical.
Karabin	U.S.A.	Government	Numerical.
Kramer	U.S.A.	Government	Numerical.
Maleki	U.S.A.	Consultant	Empirical/numerical.
Mark	U.S.A.	Government	Empirical.
Su	U.S.A.	Mining company . .	Numerical.
van der Merwe .	South Africa .	Consultant	Empirical.
Zipf	U.S.A.	University	Numerical.

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A UNIQUE APPROACH TO DETERMINING THE TIME-DEPENDENT IN SITU STRENGTH OF COAL PILLARS

By Kousick Biswas, Ph.D.,[1] Christopher Mark, Ph.D.,[2] and Syd S. Peng, Ph.D.[3]

ABSTRACT

In general, it cannot be assumed that the strength of coal pillars remains constant over long periods of time. Field observations indicate that a coal seam, especially when it contains a parting layer, deteriorates over time, reducing the load-bearing capacity of the pillars. This paper discusses a unique approach to determining the time-dependent strength of coal pillars in the field. Three coal pillars that were developed 5, 15, and 50 years ago were chosen for the study. Holes were drilled in coal and parting layers in each pillar, and the strength profiles were determined for each hole using a borehole penetrometer. The strength data were treated statistically to establish time-dependent strength equations for different layers. The results can be used to help estimate the loss of pillar capacity over time.

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INTRODUCTION

All manmade structures deteriorate over time; pillars in underground coal mines are no exception. There are numerous examples of coal pillars failing many years after they were developed. Scrutiny of existing pillar design theories indicates that few make any attempt to consider the effect of time. Similarly, there is rarely an attempt to consider the inhomogeneous nature of most coal seams. For example, the classic pillar design methodology involves the following three steps:

1. Calculate the vertical stress on the pillar:

$$S_v = \frac{\gamma H(W + W_e)(L + W_e)}{(WL)}, \tag{1}$$

where S_v = vertical stress,

γ = unit weight of the overburden,

H = depth of the seam,

W = pillar width (minimum pillar dimension),

L = pillar length (maximum pillar dimension),

and W_e = entry width.

2. Calculate the pillar strength using Bieniawski's formula [Bieniawski 1992]:

$$S_p = S_1\left(0.64 + \left(0.36\,\frac{W}{h}\right)\right), \tag{2}$$

where S_p = pillar strength,

S_1 = in situ seam strength,

and h = seam height.

3. Calculate the stability factor (SF) as

$$SF = \frac{\text{Pillar strength}}{\text{Pillar stress}} = \frac{S_p}{S_v}. \tag{3}$$

The stability factor that is calculated using equations 1-3 assumes that—

- The coal strength is constant and does not deteriorate over time; and
- Coal seams are homogenous.

Back-analyses of subsidence above abandoned mines using the classic methodology have found that pillar failures have occurred over a broad range of stability factors [Marino and Bauer 1989; Craft and Crandall 1988]. The implication is that over time the standard pillar design methodology loses its ability to accurately predict the strength of coal pillars.

One recent South African study focused on the phenomenon of pillar scaling over time [van der Merwe 1998]. Twenty-seven case histories of pillar failure, occurring as long as 15 years after mining, were included in the database. Three parameters were found to be statistically significant: coal seam, pillar height, and time to failure. The study concluded that the scaling rate decreases exponentially over time and further hypothesized that "the inner portions of the pillar, being protected from the atmosphere, would then weather at a lower rate."

This paper describes a detailed study of the time-dependent structural deterioration of coal pillars and proposes a means to estimate the strength reduction of the coal seam in situ by taking into account the seam heterogeneity.

FIELD OBSERVATIONS

A survey conducted by West Virginia University, Department of Mining Engineering, of room-and-pillar mines in the eastern Appalachian region found that some of the coal seams contain one or more mudstone or claystone layers with variable thicknesses [Tsang et al. 1996]. For example, the Pittsburgh and Twin Freeport Seams contain parting layers in the coal seam. During field visits to several coal mines developed in these seams, the conditions of many pillars in worked-out districts, some as much as 100 years old, were visually inspected. Most of the pillars did not show any apparent sign of instability because of their large size compared to their depth (stability factors ranged from 2 to 12).

A more detailed inspection revealed several kinds of weathering actions on the different layers of the coal seam with varying degrees of severity. The following structural deteriorations were noticed on older pillars:

• Conversion of mudstone/claystone layer to clay due to prolonged exposure to the mine moisture;

• Squeezing of the softer parting layer by the top and bottom portion of the coal;

• Major peeling of the parting layer;

• Separation of the parting from the host coal along the slick interfaces (perhaps the result of differential slippage); and

• Minor peeling of the top and bottom portion of the coal.

Figure 1 illustrates this deterioration in the structure of a pillar.

From the field observations, it was concluded that the structural deteriorations in both coal and parting are dependent on time. From these observations, aided by some laboratory studies and finite-element modeling [Biswas 1997], it was possible to postulate a conceptual model of the time-dependent strength profiles in the coal and parting layers (figures 2 and 3). Its assumptions are that—

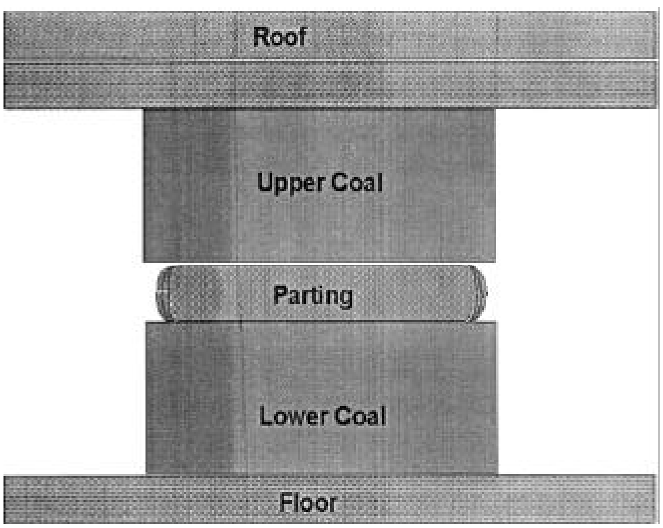

Figure 1.—Peeling of weathered parting in coal seam.

• The pillars are not affected by any mining activity in their vicinity; and

• The majority of the yield zones depicted in figures 2 and 3 are the result of the weathering action on the different layers in the pillar.

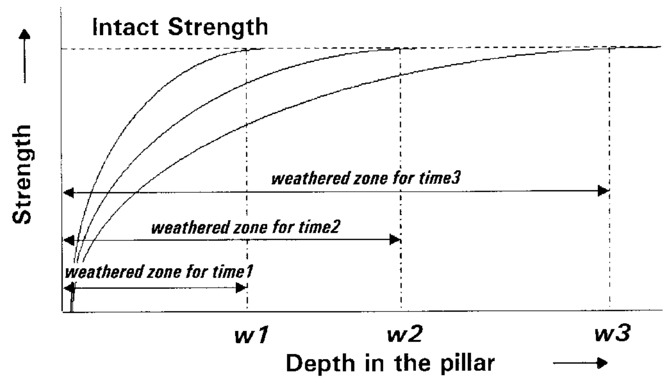

Figure 2.—Conceptualization for strength deterioration for parting. (Note: time1 < time2 < time3.)

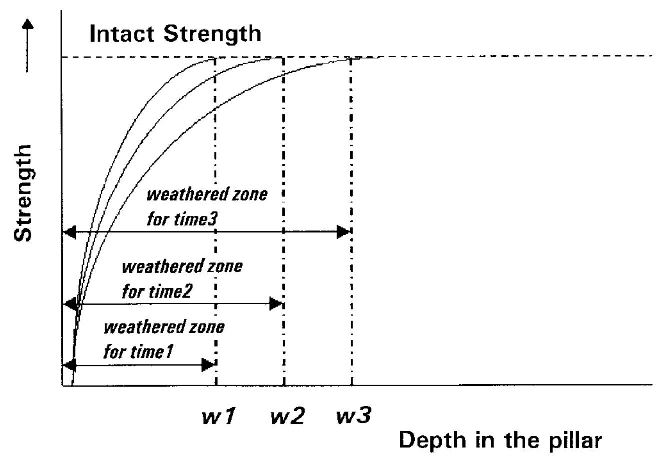

Figure 3.—Conceptualization for strength deterioration for coal. (Note: time1 < time2 < time3.)

IN SITU DETERMINATION OF TIME-DEPENDENT STRENGTH

The goal of this study was to determine one set of time-dependent strength profiles under in situ conditions. A detailed testing program was designed to establish the strength reduction in various layers of a pillar in situ over time.

THE STUDY SITE

The study was conducted at the Safety Research Coal Mine at the National Institute for Occupational Safety and Health's (NIOSH) Pittsburgh Research Laboratory. The Safety Research Coal Mine was selected for the following reasons:

• The overburden depth is very shallow, ranging from 15 to 18 m (50 to 60 ft); thus, any deterioration of the pillars is attributable to the effect of weathering rather than stress.

• The mine is developed in the Pittsburgh Seam, and it contains a parting of varying thickness (from 0.15 to 0.3 m (6 to 12 in)).

• The mine has accessible pillars developed as recently as 1991 and as long ago as the 1940s.

• The mine remains more or less inactive in terms of mining activities.

Three pillars were chosen in the mine based on their current conditions and the thickness of the parting. The three pillars were developed 5, 15, and 50 years ago. Due to other technical difficulties, more faces could not be chosen for this experiment. Figure 4 shows the mine plan and the location of the study sites.

THE APPARATUS

A borehole penetrometer (BPT) was used to measure the strength profiles in the coal and parting layers. The basic principle followed by the BPT is to fracture the borehole wall by means of an indenter and record the pressure that initiates the first fracture [Hladysz 1995]. The recorded failure pressure is then converted by a formula to determine the uniaxial compressive strength (UCS) at that location in the borehole. The BPT's great advantages are that the rock strength is tested in situ, and multiple tests can be conducted within a single borehole [Zhang et al. 1996].

The BPT consists of the following components:

- Head
- Hydraulic pump with oil reservoirs and pressure transducers
- Displacement indicator
- Four-wire electric cable
- High-pressure hydraulic hose with quick couplers
- Set of extension rods

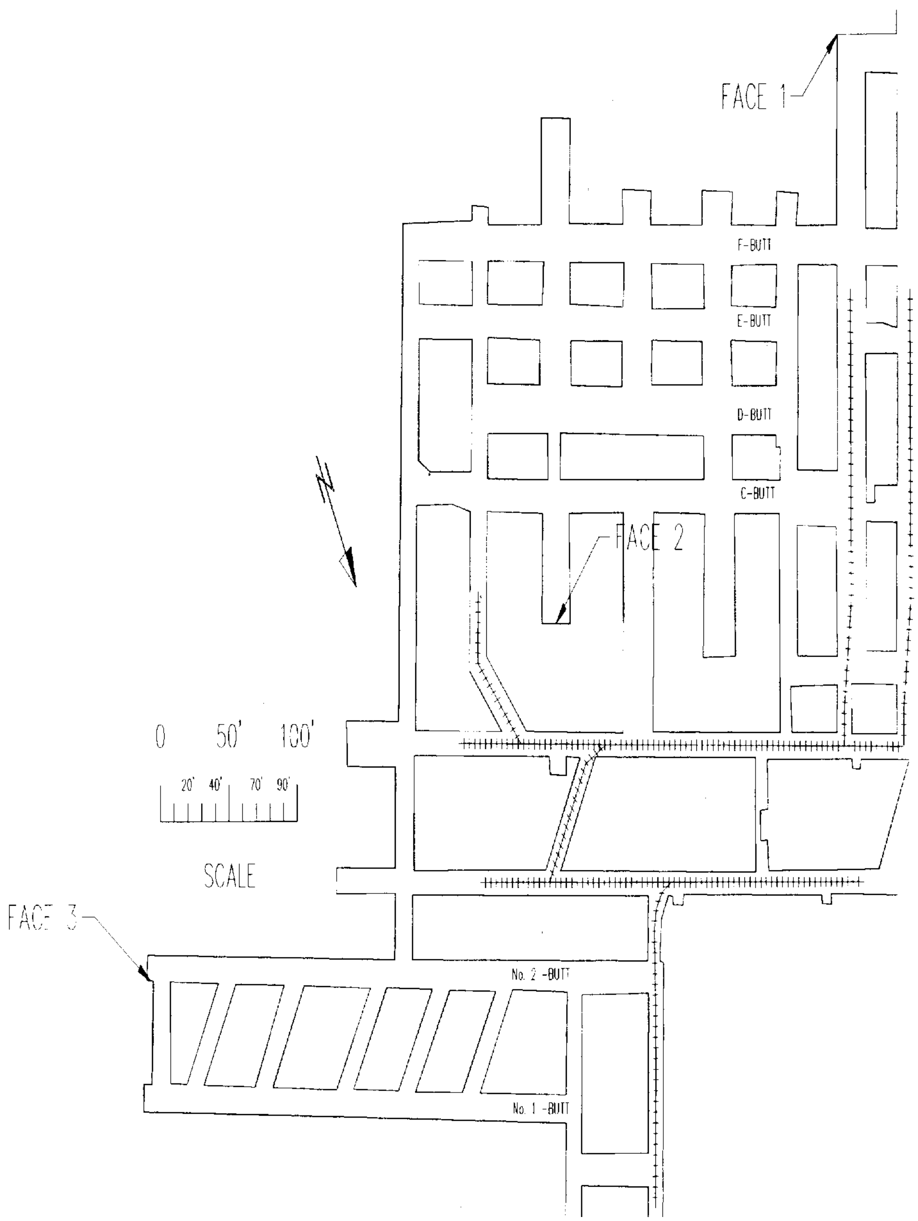

Figure 4.—Mine plan indicating three faces chosen for the BPT tests.

The BPT test setup is illustrated in figure 5. To perform the test, the head of the device is inserted into a standard NX drill hole with the help of a set of extension rods. When the head is positioned at the desired depth, the indenter is forced into the borehole wall using the hydraulic pump. At the critical pressure, the indenter penetrates the rock rapidly, making a small crater around the indenter's tip. This event is indicated by a rapid movement of the needle on the displacement indicator and by a sudden drop in pressure (figure 6). In hard and brittle rock, an audible sound is often associated with rock failure. The critical pressure causing the rock to break is a function of rock separation resistance (or penetration resistance). Penetration resistance is proportional to the material properties of the rock mass and the state of stresses. By repositioning the head and repeating the test procedures along the entire length of the hole, a penetration profile (or strength profile) for the tested section of the rock mass can be determined.

To achieve accuracy, a pressure transducer, a data acquisition module, and a digital readout unit are used. The failure pressure and ram displacement data recorded at a specified time interval are stored during an individual test and later transferred to a computer to determine the failure pressure. A portable battery-operated recorder unit records the collected data. The pressure transducer that is connected to the hydraulic pump generates the pressure signal; the displacement signal comes from a linear variable differential transformer (LVDT) that is linked to the indenter. The recorded data are stored in the data logger unit memory and later played back using a personal computer driven by application software. The data from a typical BPT test include the pressure, displacement of ram or indenter, time and an identification for the hole No., test depth, test date, etc. More details about the instrument, its specifications, principles, and testing procedure can be found elsewhere [Hladysz 1995].

THE EXPERIMENT

For each BPT test, the following steps were conducted:

1. Connect the hydraulic hoses to the head and to the pump.
2. Connect the cable to the head and to the data acquisition displacement input terminals.
3. Connect the cable to the pressure transducer and to the data acquisition pressure input terminals.
4. Set up the recording session parameters in the data logger unit (e.g. date, ID No., etc.).
5. Insert the head into the borehole and position the device at the desired depth.
6. Close the main valve of the pump.
7. Initiate a data recording session.
8. Increase pressure slowly at a constant rate, continuing to pump until failure occurs.
9. Open the valve to allow the indenter to retract fully and stop recording.
10. Reposition the penetrometer head and repeat steps 4 to 9.

Two NX boreholes were drilled in each test pillar, one in coal and one in the parting. The holes were each 3 m (10 ft) long. About 15-20 tests were conducted along each borehole. The testing frequency was higher near the pillar edge; it was postulated that the rib edge would be more disturbed than the intact central portion of the pillar. All of the data for each test were collected in the storage module during the tests and later transferred to a computer for more detailed analysis. The data for each test point were manipulated in a spreadsheet program; finally, a graph was plotted for each test point. The graph consists of time on the X-axis, failure pressure on the primary Y-axis, and the relative displacement rate of the indenter on the secondary Y-axis. Typical graphs for the parting and the coal are shown in figures 6 and 7, respectively. The failure pressure in the hard rock, in general, is characterized by a distinct jump (increase) in the ram displacement.

DATA ANALYSIS

The first step in analyzing the data was to determine the failure pressures at all test points. Then, the following conversion formula was used to convert the failure pressure to the UCS:

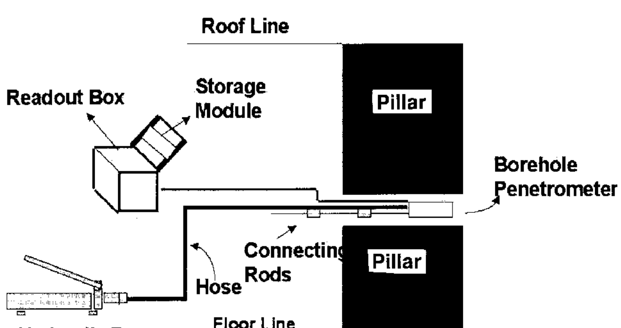

Figure 5.—BPT test setup.

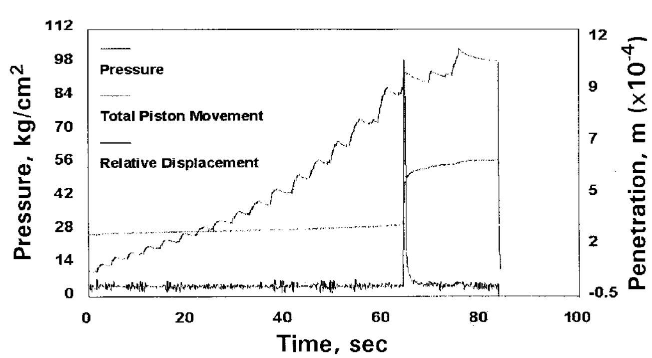

Figure 6.—Typical raw BPT test data analysis for parting.

$$UCS = (F_s)P_f, \qquad (4)$$

where F_s = strength factor,

and P_f = failure pressure from the BPT test.

For coal, the value of the strength factor was 1.25, as suggested by Zhang et al. [1996]. For the parting, a value of 1.00 was used based on laboratory studies of the cores of the parting obtained from the BPT test holes [Biswas 1997].

The scatter plots of the converted strength values were obtained for each hole in each face. Because these scatter plots showed considerable variability in the trend of the strength deterioration, which is a typical characteristic of any experiment conducted in situ, a curve-fitting program called Curve Expert was used to fit the best curve with the highest correlation coefficient. Figures 8-10 illustrate the best-fit curves for the parting, and figures 11-13 illustrate the best-fit curves for the coal for all three faces.

The general form of all of the best-fit equations for both coal and parting is

$$y = a(1.01 - e^{-bx}), \qquad (5)$$

where a and b are the coefficients,

y is the failure pressure or the strength,

and x is the depth (in this case, the range is from 0.06 to 3 m (0.2 to 10 ft).

The negative exponential and its negative power give the best-fit curves their asymptotic form. The correlation coefficients for the best-fit equations for the parting and coal for each age group are 0.84, 0.85, 0.89 and 0.96, 0.88, 0.94, respectively.

For the parting, the gradient in the weathered zone for the younger face is initially steeper, but the slope flattens as the age increases. This change in strength gradient before it reaches the intact or stabilized strength is considerable. The weathered zone apparently expands from 1 to 3 m (3.2 to 10 ft) over the 50 years. For coal, the strength gradient for all of the age groups is steeper than that of the parting, and the expansion of

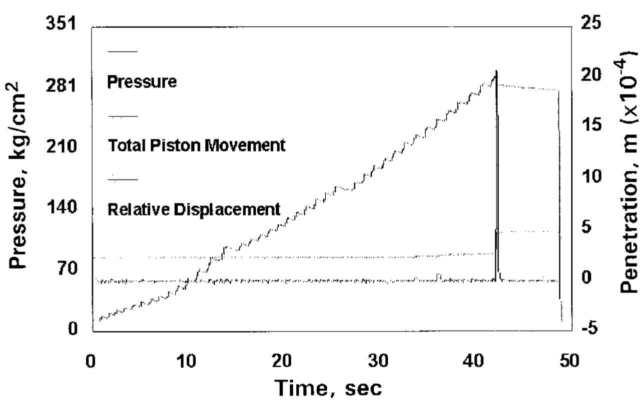

Figure 7.—Typical raw BPT test data analysis for coal.

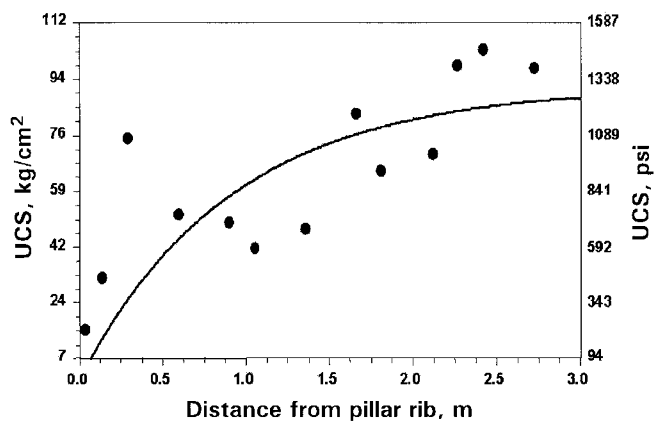

Figure 9.—Best-fit curve for 15-year-old parting.

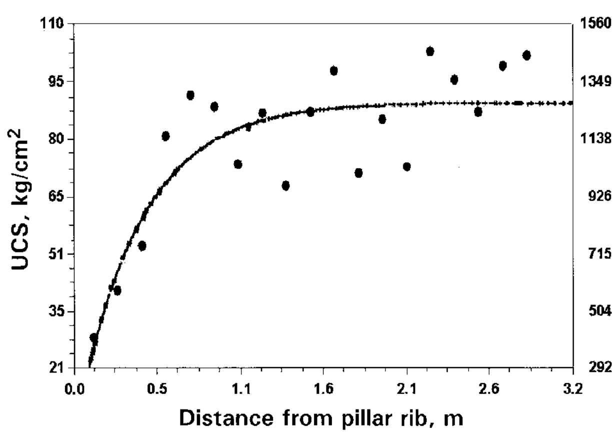

Figure 8.—Best-fit curve for 5-year-old parting.

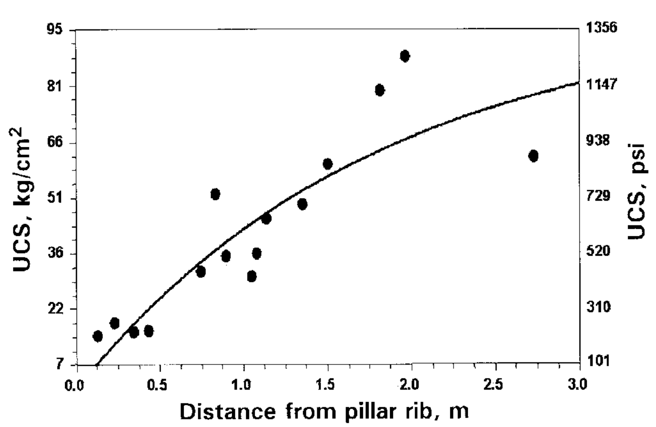

Figure 10.—Best-fit curve for 50-year-old parting.

the weathered zone is much less (from 0.2 to 1 m (0.7 to 3.2 ft)). These findings fit the conceptual model of the strength degradation for parting and coal over time described earlier.

Figures 8-13 also indicate that there is some borehole-to-borehole variability in the intact strength measured in the interior of the pillars for both the coal and the parting. This variability may be attributed to natural variability between the three different faces. In order to generalize the results, the data from each borehole were normalized to the measured intact strength. The normalized strength curves are shown in figures 14 and 15.

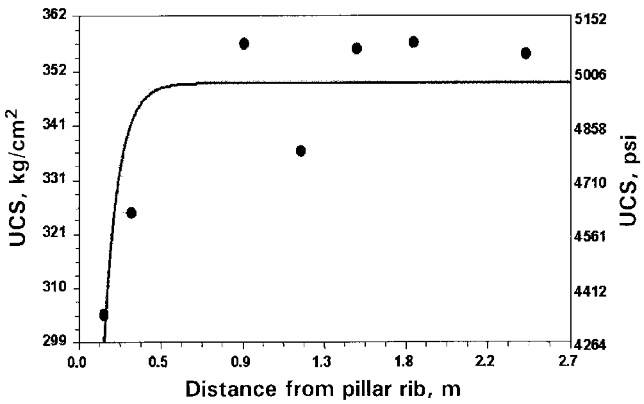

Figure 11.—Best-fit curve for 5-year-old coal.

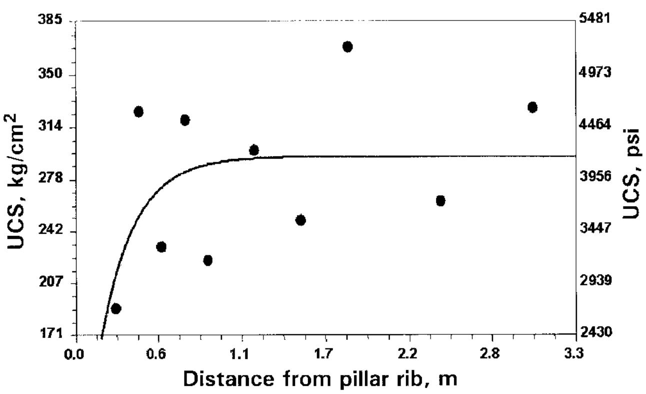

Figure 12.—Best-fit curve for 15-year-old coal.

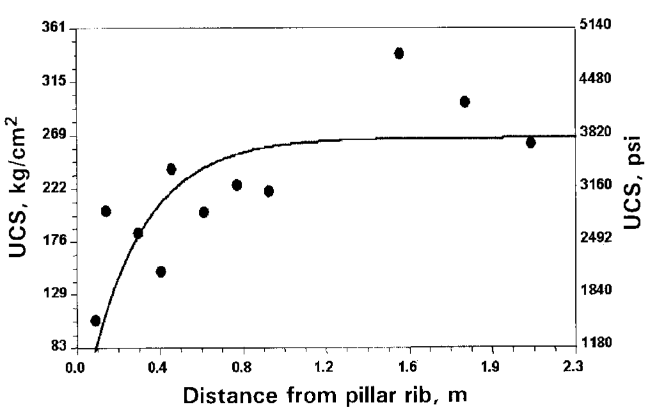

Figure 13.—Best-fit curve for 50-year-old coal.

FORMULATION OF TIME-DEPENDENT STRENGTH DETERIORATION

The BPT data can be used to derive a time-dependent strength formula for the pillars in the study. Using the best-fit equations shown in figures 14-15, data sets were generated for each material for all three ages. The data sets were generated for the depth ranges from 0.06 to 3 m (0.2 to 10 ft). No data could be generated right at the ribline because no BPT tests were conducted there. A nonlinear regression analysis was conducted on these data sets separately for the coal and for the parting with two independent variables (time and depth) and one dependent variable (strength). A freeware software called NLREG34 was used to perform the nonlinear regression. Equation 6 is the stress gradient for the parting, and equation 7 is the final equation for coal:

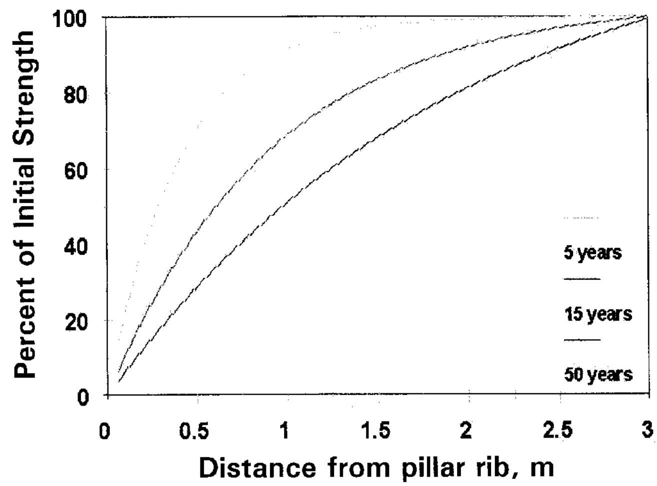

Figure 14.—Time-dependent strength deterioration for parting.

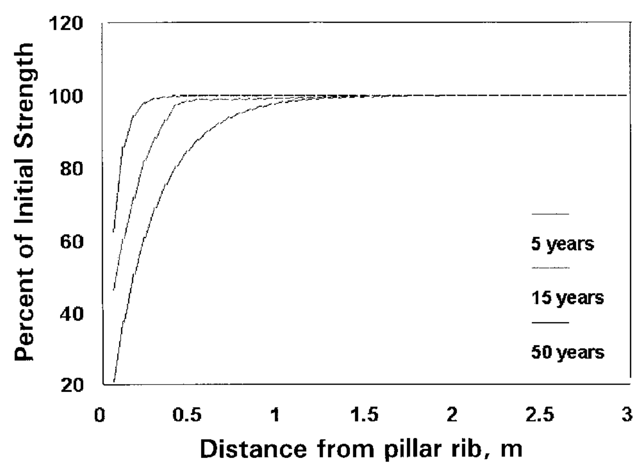

Figure 15.—Time-dependent strength deterioration for coal.

$$\%\ \text{parting strength} = 100\,(1.01 - e^{-0.5\,D}) - 0.45t \qquad (6)$$

$$\%\ \text{coal strength} = 100\,(1.01 - e^{-3.5\,D}) - 0.13t \qquad (7)$$

where D = depth into the rib, ft,

and t = time after mining, years.

In these equations, the strength is defined as a percent of the original intact compressive strength that is assumed to be constant in the core of the pillar. Near the rib, the strength is a function of the distance from the rib (depth) and the time after mining. The relationship between the strength and the depth is a negative exponential, but that between strength and time is linear.

Unfortunately, applying these time-dependent strength equations to predict the strength of full-scale pillars is not simple. Three issues are foremost:

1. *Effect of parting thickness:* If the parting is the pillar's weakest layer, as in this study, then a thicker parting would be expected to result in a weaker pillar.
2. *Effect of parting on confining stress within the pillar:* Most of the load-bearing capacity of a coal pillar is due to the development of confining stress within the pillar's core. Studies have shown that many pillars contain weak layers of clay or friable coal, but their effect on overall pillar strength is ambiguous [Mark and Barton 1996].
3. *Nonlinear effect of time:* In reality, the rate of strength degradation probably decreases with time, as suggested by van der Merwe [1998]. Because this study included only three pillars, it was difficult to quantify the nonlinear relationship between time and strength.

Nevertheless, if the limitations of the necessary assumptions are kept in mind, it is possible to use the strength gradient equations to shed light on the possible effects of time on coal pillar stability. The following example illustrates one possible approach. The key assumption is that *at any particular time, the distance from the actual pillar rib and the depth at which the strength is 60% of the intact strength will be considered as the width of the portion of the weathered zone that is not capable of carrying any load and thus transfers the load on the intact portion of the pillar*. The effect of this assumption is that the pillar's strength is decreased over time as the width-to-height ratio diminishes, whereas the applied stress increases as the pillar's load-bearing area is reduced.

To calculate the time-dependent stability factor, the following steps are followed:

1. Calculate the original stability factor using equations 1-3.
2. Determine the strength profile at a specified time using equation 3 or 4, and determine the depth of weathering (where the strength is 60% of the intact).
3. Calculate the resultant pillar width by subtracting the depth of weathering from the original pillar width.
4. Recalculate the applied stress using equation 1 and the new pillar dimensions.
5. Use equation 2 to determine the new pillar strength and equation 3 to calculate the reduced stability factor at the specified time.
6. Repeat this process to determine the approximate lifespan of the pillar.

For example, assume the following parameters:

- The overburden depth is 244 m (800 ft).
- The pillar is a square pillar with a 15.2-m (50-ft) dimension.
- The seam height is 1.8 m (6 ft).
- The entry width is 6.1 m (20 ft).
- The in situ seam strength is 6.2 MPa (900 psi).

Because the parting is the weakest layer of the seam in this case, to be on the conservative side, equation 6 (for the parting) is used to determine the strength profile and also the width of yielded zone due to the weathering process. From a statistical point of view, it is recommended that equations 6 and 7 be used within the same time range as the original field data used in their development, i.e., 5 to 50 years [Myers 1990].

Figure 16 illustrates the changes in strength and applied stress over time. Where the two curves meet, at time = 35 years, the stability factor is 1.0, which means that the pillar has a 50% chance of failing before that time.

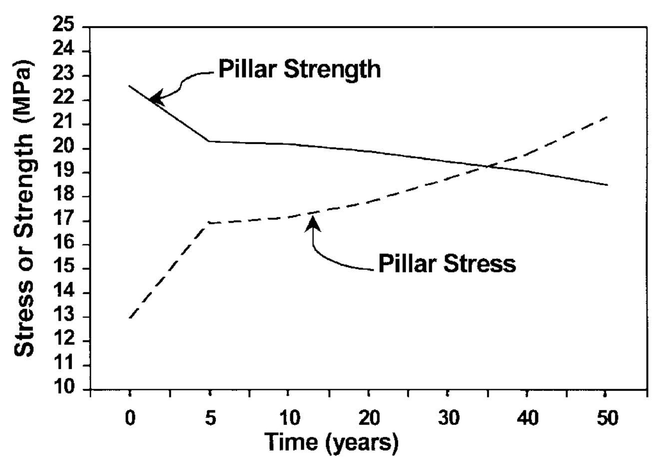

Figure 16.—Safety factor reduction over time.

CONCLUSIONS

The use of the BPT to measure the in situ time-dependent strength is the unique feature of this study. It generated a set of in situ strength data in a relatively simple field-testing program. The in situ data were used to develop time-dependent strength equations for coal and parting layers. An example case was used to demonstrate the use of these equations in predicting the change of stability factor over the years.

The parting material weathered much more rapidly than the coal. This implies that much of the observed between-seam variability in long-term pillar strength may be due to the presence or absence of partings in the coal. However, this study only addressed a single type of parting material within a single coal seam. Much work remains before the effect of time on coal pillar strength is fully understood.

ACKNOWLEDGMENTS

The authors are grateful for the opportunity to use the Safety Research Coal Mine at NIOSH's Pittsburgh Research Laboratory (PRL) for the in-mine studies. The expertise and assistance of PRL employees David C. Oyler, mechanical engineer, and Craig S. Compton, engineering technician, were also invaluable.

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DEVELOPMENTS IN COAL PILLAR DESIGN AT SMOKY RIVER COAL LTD., ALBERTA, CANADA

By Peter Cain, Ph.D., P.Eng.[1]

ABSTRACT

Smoky River Coal Ltd. mines low-volatile metallurgical coal by surface and underground methods in the foothills of the Rocky Mountains of Alberta, Canada. Current underground operations are confined to the 5B-4 Mine. Development of 5B-4 began in January 1998; production from depillaring sections commenced in July 1998.

This paper describes the history of underground mining on the Smoky River property in terms of extraction methods and pillar design. The development of the present pillar design guidelines is discussed in this context. Recent work to prepare a number of case histories for back-analysis using the Analysis of Retreat Mining Pillar Stability (ARMPS) method is described, along with the modifications developed for calculating the ARMPS stability factor for retreat extraction of thick seams. The design criteria are described, as well as the geotechnical program implemented in order to verify its applicability.

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INTRODUCTION

The Smoky River Coalfield is located in west-central Alberta, Canada, within the inner foothills of the Rocky Mountains. The mine is approximately 20 km north of Grande Cache and 360 km west of Edmonton (figure 1). Most of the property is contained in a block approximately 29 km long by 19 km wide. The coal leases cover about 30,000 ha. The general mine layout is shown in figure 2. Underground mining is currently located in the 5 Mine area.

The coal seams and surrounding strata are within the Gates Formation (of the Lower Cretaceous Luscar Group) and outcrop near the mine. The Gates Formation is divided into three members: Torrens, Grande Cache, and Mountain Park (figure 3). The Torrens is a distinct marine sandstone and siltstone sequence about 30 m thick. It is overlain by the Grande Cache Member, which consists of approximately 158 m of nonmarine siltstones, sandstones, mudstones, and all of the significant coal seams in the area. The Grande Cache Member is overlain by the Mountain Park Member, which consists of 155 to 192 m of nonmarine sandstones, mudstones, siltstones, and minor coal seams.

The predominant structure of the coalfield strikes northwest to southeast and comprises thrust sheets containing folded layers of competent sandstone and siltstone units, incompetent mudstone, and coal. Dips vary considerably, from horizontal to overturned. Underground mining by room-and-pillar methods is restricted to areas where the strata dip less than 16°, which is the practical limit of continuous miner and shuttle car operation. The orientation of the underground mine workings in figure 2 gives a clear indication of the structural environment; the workings are either faulted or steeply folded off on the northeast and southwest limits of mining.

The significant coal seams present are numbered from the lower (older) to the upper (younger) and comprise the 4, 8, 10, and 11 Seams. 4 Seam has been mined extensively (figure 2) using conventional room-and-pillar mining techniques. 8 and 11 Seams are not considered economical to mine because of thickness and low quality. Mining in 10 Seam has been attempted, including two longwall panels above 9G-4 Mine; however, a weak immediate roof comprising two 0.6-m coal seams in the first 2 m of strata has always presented stability problems.

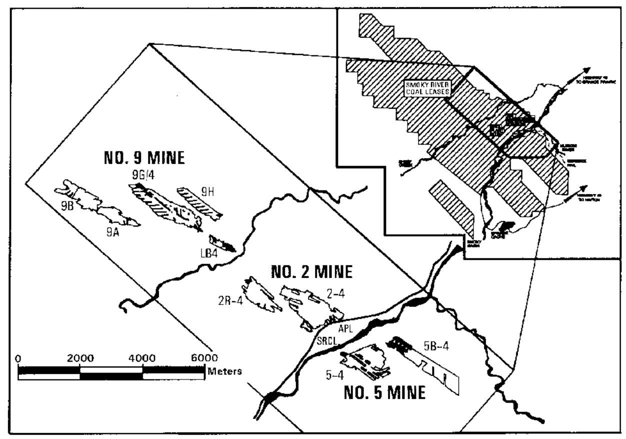

Figure 2.—Site layout.

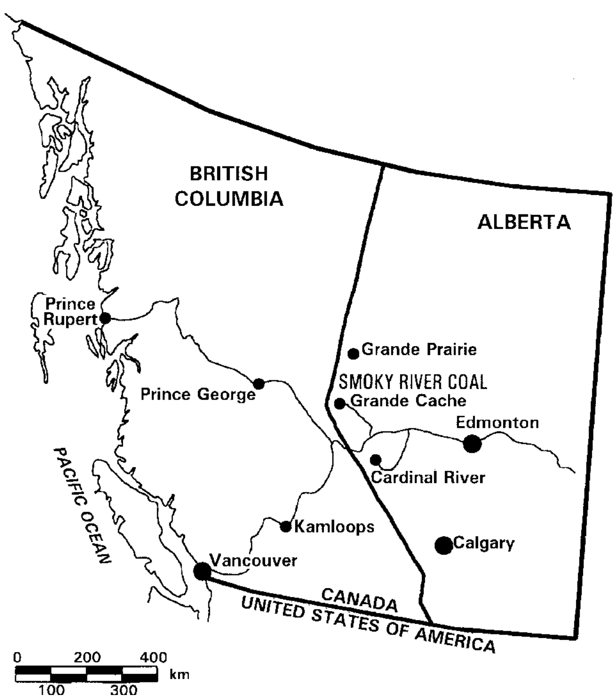

Figure 1.—Location of Smoky River Coal Ltd.

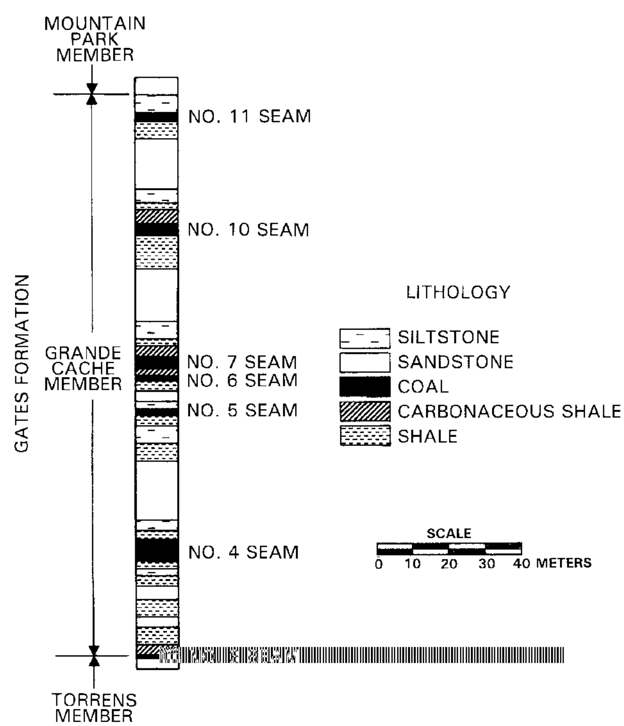

Figure 3.—Generalized stratigraphic column, Smoky River Coalfield.

HISTORICAL MINING METHODS AND PILLAR DESIGN

Underground mining at Smoky River Coal Ltd. (SRCL) commenced in 1969 in 5-4 and 2-4 Mines. The initial intent was to develop for longwall extraction; however, two early attempts at longwall mining failed and retreat room-and-pillar extraction became standard.

The original mining method was to develop three 6-m-wide entries on 30-m centers from the portal to the limit of mining, generally along strike, with crosscuts at 30-m centers. Parallel sets of entries were driven separated by 50-m barrier pillars (figure 4). On reaching the limit of mining, the road and barrier pillars were split along strike to form blocks approximately 12 m wide and mined using an open-ended "Christmas tree" method, taking 6-m passes each side with a conventional continuous miner. This method, described in more detail by Wright [1973], worked well in 2-4 Mine, but was unsuccessful in 5-4 Mine due to the weaker roof and pervasive thrust faulting in and above the coal seam.

In the early 1970s, a major geotechnical investigation program was launched to assist mine staff in planning pillar dimensions and support. Extensive load and deformation monitoring was conducted [Bielenstein et al. 1977]; concurrent testing by air injection investigated the development of yield and elastic zones within coal pillars [Barron et al. 1982].

In the early 1980s, the many disadvantages of the three-entry system were overcome by adopting a five-entry system (figure 4*B*) with short-life panels [Robson 1984]. Panels comprising five parallel entries were developed off of main development sections. This mining method depended for its success on the stability of pillars separating the panels and pillars that protected the main entries from the depillared areas. In fact, five types of pillars were recognized:

- Barrier pillars between mining panels;
- Entry pillars protecting the main entries;
- Panel pillars formed during the development of mining panels;
- Split pillars formed by splitting panel pillars prior to depillaring; and
- Remnant pillars, the diminishing remnants of split pillars formed during depillaring operations.

Tolerable probabilities of failure were estimated for each pillar type, and an empirical design criterion was developed that took into account this probability of failure [Barron et al. 1982]. Favorable trials of the five- entry system in A Mine (figure 2) resulted in its adoption in 9H and 9G Mines. Further refinement of pillar design methods, relying heavily on practical experience and a comprehensive review of pillar design methods from around the world, resulted in a design nomogram [Kulach 1989]. The method was based on the tributary area method of load calculation (considered to represent the best and safest estimate of the loads developed on pillars) and Bieniawski's [1983] method of determining pillar strength.

Mining continued in the late 1980s and 1990s in 9H and 9G Mines using this method of pillar design. The small resource block exploited by the LB-4 Mine necessitated a change in method, with entries developed to the farthest extent and retreated back, but all three mines were successful from a pillar stability standpoint.

In 1997, plans were developed to exploit a previously untouched parcel of coal to the north of the old 5-4 Mine. The shape of the resource block, 370 m wide by 2,500 m long, bounded by steeply dipping thrusted zones to the northeast and southwest, largely dictated the mining layout, which is shown in figure 5.

During the planning stages of the mine, it was soon realized that conditions would be very different from the more recent underground operations, which were carried out at shallow to moderate depths under a competent sandstone roof. The proposed 5B-4 Mine would operate at depths of up to 550 m and beneath a roof affected by pervasive thrust faulting. Both pillar design and roof support requirements necessitated reevaluation for the operation to be successful.

Although the SRCL pillar design criterion had been used successfully in a number of mines, it had some obvious disadvantages with respect to its application in 5B-4 Mine:

- The nomogram is restricted to 12-m-wide by 3.6-m-high pillars and 6-m-wide roadways.
- The method is based on a strength calculation for square pillars and severely underestimates the strength of rectangular pillars.
- The design criterion is based on U.S. methods that have undergone substantial modification in the past 10 years.

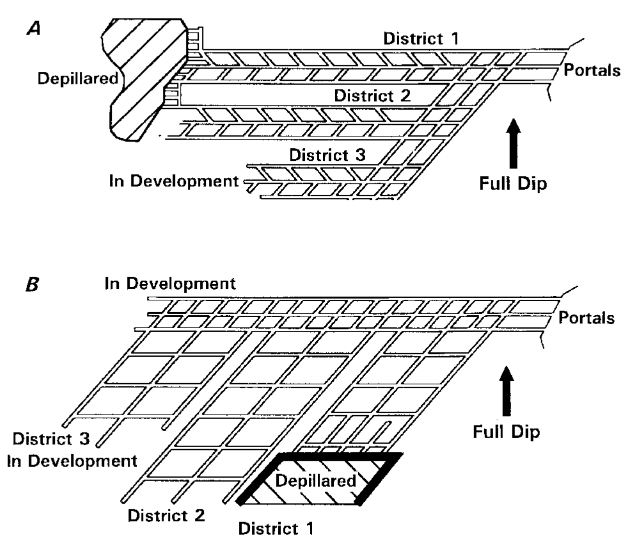

Figure 4.—Development of mining methods. *A*, three-entry system, long-life panels; *B*, five-entry system, short-life panels.

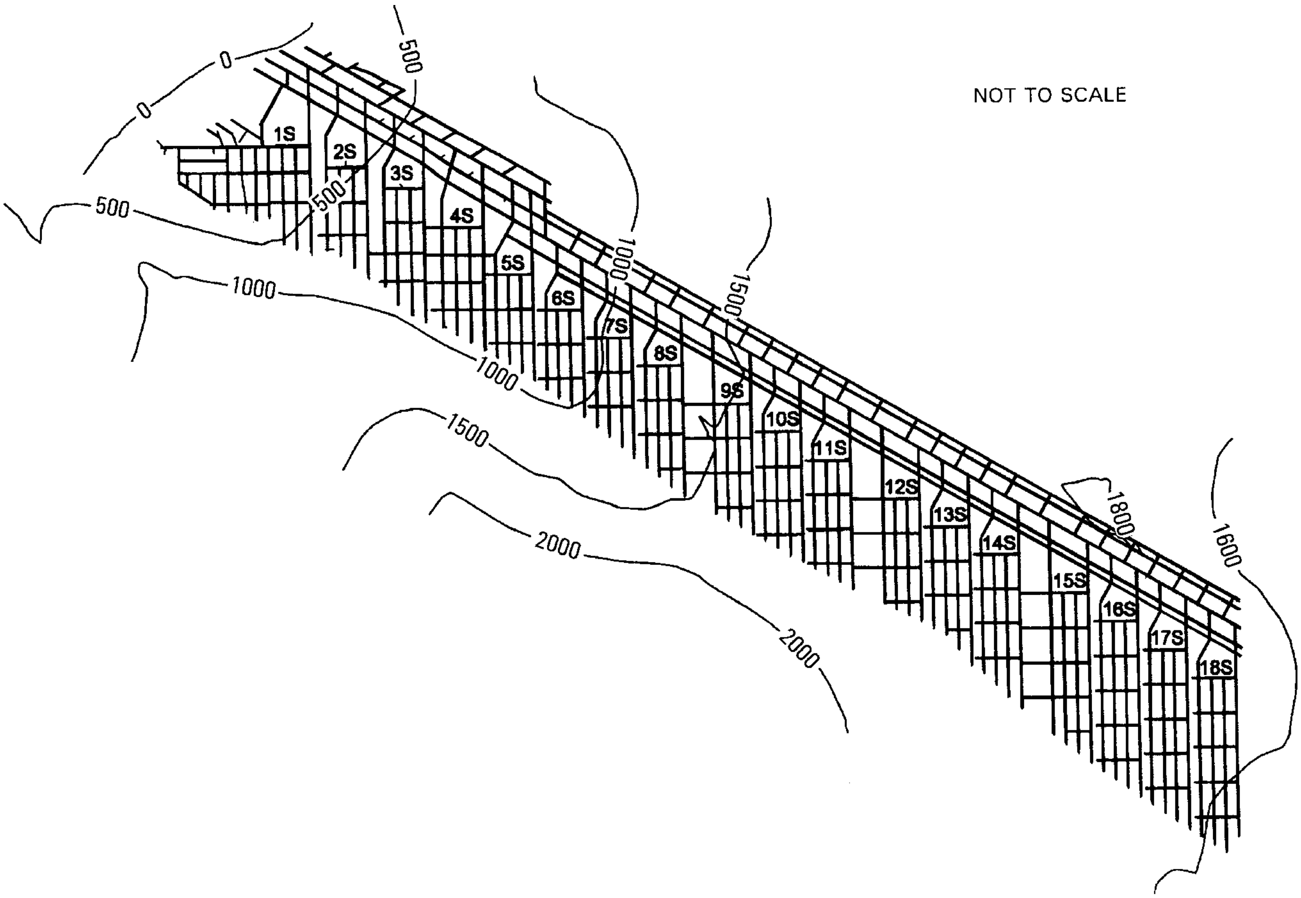

Figure 5.—Layout of 5B-4 Mine. ***(Elevation in feet.)***

Mining plans for 5B-4 included rectangular pillars ranging from 15 m to 36 m wide and 3.6 m high, standing between 4.9-m-wide roadways, which lay outside the empirical basis of the design nomogram. Although a nomogram for 5B-4 parameters could have been developed, the availability of more recently developed design methods that specifically address the strength of rectangular pillars warranted consideration of a change in design approach.

ANALYSIS OF RETREAT MINING PILLAR STABILITY (ARMPS)

The most recent development in pillar design in the United States is the Analysis of Retreat Mining Pillar Stability (ARMPS). ARMPS was developed by the former U.S. Bureau of Mines [Mark and Chase 1997] based on extensive case history data. ARMPS is available as a Windows 95™ software package and has the following advantages over previous methods used by SRCL:

- The increased load-bearing capacity of rectangular pillars over that of square pillars of the same width is taken into consideration.

- The load-bearing capacity of diamond- or parallelogram-shaped pillars is taken into consideration.

- ARMPS allows for an analysis of the stability of pillars in the active mining zone (AMZ) during development, during retreat, and with gobs on one or both sides.
- The effect of depth on abutment loading, based on angles of caving, is considered.
- The effect of slabbing the interpanel pillar on pillars in the AMZ is considered.

ARMPS is a very flexible method of analysis. The software allows the user to input all of the major parameters relating to layout, mining, and pillar dimensions and location of any worked-out, caved areas. It also allows analysis of changes in pillar stability as a result of mining progress, from development to the extraction of coal pillars alongside a gob or between two gobs. Mark and Chase [1997] present a full description of the methods used to calculate pillar loading and pillar strength in the ARMPS program. The principal output of the program is the stability factor (SF), which is the product of the estimated load-bearing capacity of pillars in the AMZ divided by the estimated load on those pillars.

The concept of the AMZ follows from a hypothesis by Mark and Chase [1997] that pillars close to the retreat extraction line behave together as a system, i.e., if an individual pillar is overloaded, load is transferred to adjacent pillars. If these are of adequate size, the system remains stable, otherwise the pillars fail in turn, resulting in a domino-type transfer of load and pillar failure.

The size of the AMZ is a function of depth, H, based on measurements of abutment zone widths conducted by Mark [1990], which showed that 90% of abutment loads fall within a distance 2.8 / H from the gob edge.

U.S. case history data indicate that where the ARMPS SF is <0.75, nearly all of the designs were unsatisfactory; where the SF is >1.5, nearly all of the designs were satisfactory. For the deeper case histories, there was some evidence that stability factors can be lower and still ensure overall pillar stability. In addition, case histories with less competent roof rock were more stable than those with stronger roof strata, as this promoted pillar squeeze or burst activity.

Despite its utility and comprehensive analytical method, ARMPS has several drawbacks when applied to SRCL conditions:

- Case histories were confined to U.S. mines. As with any empirically based design method, this presents problems in application outside the case history environment.
- The case history database extends only to depths of about 1,100 ft, and only a few case histories were obtained at this depth of cover.
- None of the case histories matched the seam thicknesses mined at SRCL (up to 6 m).

After discussions with the developers of ARMPS [Mark 1998], it was decided that in order to confirm the applicability of ARMPS to SRCL operations, a series of calibration analyses based on depillaring operations in the coalfield was required.

BACK-ANALYSIS OF CASE HISTORIES

Mine plans from 9G, 9H, and LB-4 Mines (figure 2) were reviewed, and relevant mining data were extracted to develop a series of case histories. Each case history was then analyzed using the ARMPS method, and safety factors were recorded and compared to the existing U.S. case history database.

In order to consider the extraction of thick seams as practiced at SRCL, the calculation of the SF was modified. ARMPS allows input of a single working thickness; in most SRCL depillaring operations, however, there are two mining heights. During development, the mining height is 3.7 m; during depillaring, the mining height is 6.1 m. This variation in mining height has a marked effect on pillar stability through the height/width ratio of the pillars. Rationally, load shed to the AMZ from the 6.1-m-high pillars in the mined-out area is more effectively controlled by the pillars of 3.7-m height in the AMZ.

In order to take into account this variation in mining height, ARMPS stability factors and details of pillar loading were calculated for extraction heights of both 3.7 m and 6.1 m. The SRCL stability factor was derived as follows:

(a) The pillar load transferred to pillars in the AMZ for a mining height of 6.1 m was determined using ARMPS.

(b) The load-bearing capacity of pillars in the AMZ for a mining height of 3.7 m was determined using ARMPS.

A stability factor was calculated as: (b) divided by (a).

Table 1 presents details of the mining parameters for each of the case histories considered, as well as the stability factors obtained. Figure 6 compares the SRCL stability factors with those obtained from the published U.S. database [Mark and Chase 1997] and indicates that SRCL stability factors representing satisfactory conditions range from 0.47 to 1.74, with the majority (66%) in the range of 0.5 to 1.0.

Local mining conditions provided some assurance that the low SF values were valid. Firstly, the lowest values occurred at the greatest depth; it has been recognized that acceptable stability factors appear to be lower at depth, perhaps due to the influence of horizontal stresses in reducing the pillar loading. Secondly, the SRCL case histories are characterized by a strong, competent roof; under such conditions in the United States, acceptable pillar stability was obtained at lower values of the calculated SF.

Table 1.—Summary of SRCL case histories analyzed using the ARMPS method

Mine	District	Depth, ft	ARMPS SF (6.1 m)	Load shed to AMZ, tons	ARMPS SF (3.7 m)	Capacity of AMZ, tons	SRCL SF	Load condition
LB-4	Mine	580	1.35	5.83E+6	1.99	1.16E+7	1.56	2
9H-4	SW2	390	1.23	1.18E+6	1.80	2.05E+6	1.74	3
9H-4	SW3	485	1.35	1.69E+6	0.92	1.63E+6	0.96	3
9H-4	SW4	575	0.73	2.44E+6	1.12	2.49E+6	1.02	3
9H-4	SW5	660	0.56	3.43E+6	0.89	2.69E+6	0.78	3
9H-4	SW6	715	0.49	4.05E+6	0.77	2.77E+6	0.68	3
9H-4	SW7	755	0.61	4.71E+6	1.04	4.14E+6	0.87	3
9H-4	SW8	832	0.50	6.11E+6	0.79	4.35E+6	0.71	3
9H-4	SW9	932	0.35	4.60E+6	0.53	2.30E+6	0.50	3
9G-4	SW2	560	0.85	2.05E+6	1.27	2.46E+6	1.20	3
9G-4	SW3	650	0.58	3.26E+6	0.94	2.65E+6	0.81	3
9G-4	SW4	730	0.49	4.10E+6	0.80	2.83E+6	0.69	3
9G-4	SW5	745	0.51	3.98E+6	0.85	2.83E+6	0.71	3
9G-4	SW6	780	0.51	4.01E+6	0.88	2.90E+6	0.72	3
9G-4	SW7	840	0.41	5.21E+6	0.69	2.97E+6	0.57	3
9G-4	SW8	885	0.37	5.84E+6	0.62	3.05E+6	0.52	3
9G-4	SW9	920	0.34	6.56E+6	0.51	3.11E+6	0.47	3
9G-4	SW10	915	0.34	6.49E+6	0.53	3.10E+6	0.47	3

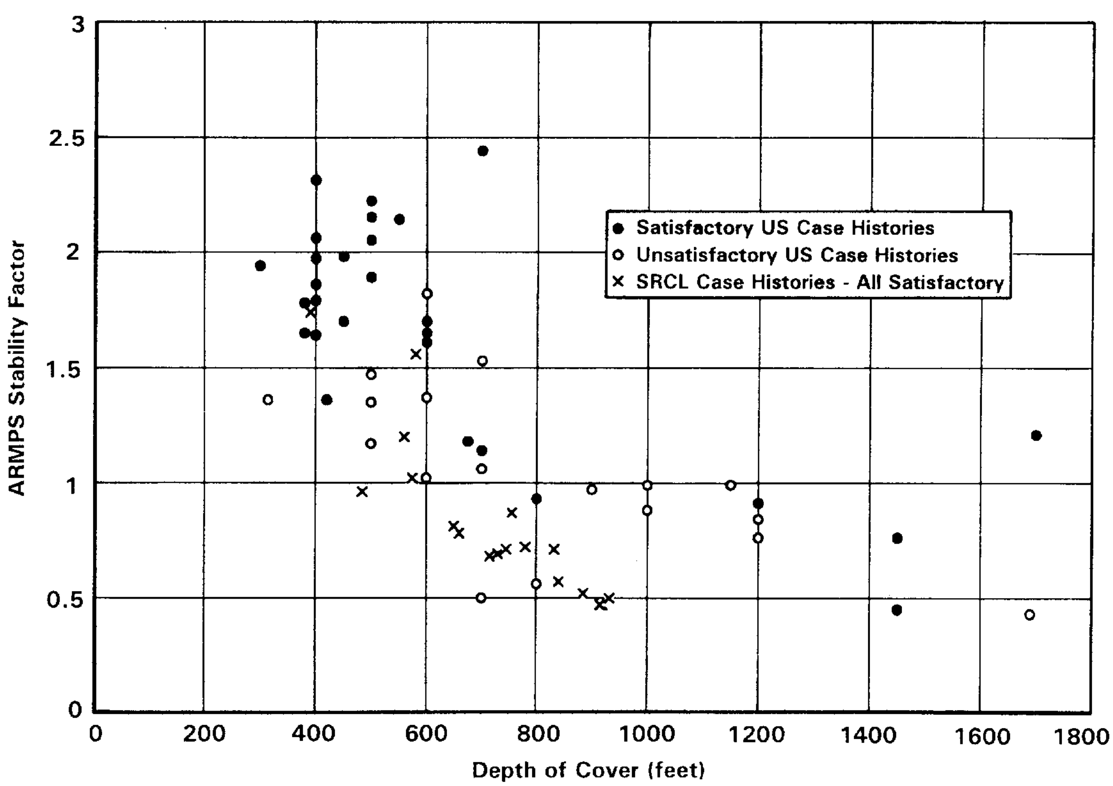

Figure 6.—Comparison of U.S. and SRCL stability factors.

DEVELOPMENT OF A DESIGN CRITERION

After considering the results of the case history analysis, it was decided to use the ARMPS method to assist in pillar design at 5B-4 Mine. Appropriate engineering practice in such cases is to design to the minimum SF that resulted in stable conditions. Evidence suggests that a pillar design resulting in an ARMPS SF of $0.5 would be stable in Smoky River Coalfield conditions. A more conservative SF of 0.7 was established.

A further limitation was imposed after an analysis of the pillar stresses on the gob corner pillar. This pillar, located adjacent to both the active retreat section gob and the barrier pillar between the active panel and the old gob, is subjected to the highest stresses and is therefore more prone to failure. The primary concern in this case is the threat of coal bumps or pillar burst, resulting in the transference of loads to adjacent pillars in the AMZ and possibly massive failure.

ARMPS analyses of SRCL case histories revealed that the maximum stress experienced on any gob corner pillar was about 41 MPa. At this stress level, the pillar proved to be stable.

A third criterion was adopted based on the size of pillars analyzed from the case histories. The minimum pillar size analyzed was 12 m wide between 6-m roadways. Maintaining this extraction ratio for the 4.9-m-wide roadways employed at 5B-4 Mine precluded the use of ARMPS for pillars <9.7 m wide.

Based on the ARMPS output from the case history data compiled from previous pillar retreat mining in the Smoky River Coalfield, the following design criterion for pillars is suggested:

- The ARMPS SF should be maintained above 0.7.
- The maximum stress on the corner pillar should not exceed 41 MPa (6,000 psi).
- Pillar widths must not be <9.7 m.

It was realized that the ARMPS-derived design criterion was also limited in application, specifically to the depths encountered in the case history analysis. With depths of cover projected to exceed those of the case histories by 50%, there was an element of uncertainty with respect to the applicability of the design criterion. This is currently being addressed by a geotechnical program that includes pillar stress monitoring, numerical modeling, and continuing assessment of the design criterion.

Vibrating wire stress cells, electronic convergence meters, and an I. S. Campbell data logger have already been deployed at three monitoring sites to collect data on the effects of mining on pillar stability. Two of the sites monitored stress changes while the site was being "mined by" during the development phase. It is hoped that these two sites will provide valuable information on the strength of the coal pillars monitored.

Results are still being evaluated; however, indications are that the design criterion is applicable. Further sites will be established as mining progresses, and the results will be incorporated into the design criterion.

SUMMARY

Development of pillar design methods at SRCL's underground operation has proceeded with developments in the mining method. The extension of mine workings to previously unencountered depths at the new 5B-4 Mine has resulted in a requirement to develop pillar design methods to match the new mining environment.

Pillar designs are currently being based on the results of a back-analysis of case histories using the recently developed ARMPS method. As with any empirical method of design, prudent engineering practice dictates the collection and analysis of pillar behavior information for design verification. Monitoring results already obtained are being analyzed to improve the design criteria. Future sites will collect data from greater depth and adjacent to more extensive workings.

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COAL PILLAR DESIGN FOR LONGWALL GATE ENTRIES

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ABSTRACT

This paper describes measured data on strata behavior obtained in recent years from sites in the United Kingdom and the implications for pillar design. The data include results from overcoring stress measurements adjacent to coal mine roadways and deformation monitoring related to longwall extraction. The stresses adjacent to mine roadways or entries have been measured at a number of coal mine sites in the United Kingdom. The results are analyzed with regard to the information they provide on pillar behavior and strength estimates.

A reduction in stress consistent with yielding of the strata adjacent to the roadways is evident. This is consistent with the confined core model for pillar behavior. The pillar strength is dependent on the rate at which vertical stress can increase with distance from the pillar edge and hence the confinement provided to the yielded material.

The measured data indicate a wide range in pillar strengths. Two groups of results are identified that show significantly different behavior corresponding to differing effective pillar strengths. Estimates of pillar strengths derived from the measured data for these two groups are compared with established equations used for pillar design.

The differing behaviors and strengths are attributed to variations in the amount of yielding and deformation in roof and floor strata and hence in the amount of confinement they provide to the coal seam. Numerical modeling is used to provide a comparison with the measured data and to indicate that this provides a feasible mechanism to account for the measured data.

As the depth of mining increases, pillars tend to become increasingly wide and squat. In such cases, it is possible for the surrounding roadways to become badly deformed and damaged while the pillars remain stable. The criteria of comparing pillar strengths and loads to establish pillar stability become less applicable in these circumstances; rather, considerations of roadway stability may be the limiting factor in determining suitable pillar dimensions.

This is the case for pillar dimensions typically employed around longwall panels in the United Kingdom. Depending on the properties of the site and what are deemed to be satisfactory roadway conditions, this can lead to wide variations in required pillar dimensions. Measured data for deformations in roadways influenced by adjoining longwall workings are presented. These show that in some circumstances the influence of longwall extraction can be transmitted over large distances and confirm the variability in required pillar sizes depending on site properties.

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INTRODUCTION

There are many equations and methods for designing coal pillars; these include back-analyses of failed and successful case histories, extrapolation from strength tests on small-scale coal samples to full-size pillars, and analytical consideration of the limiting stress distribution across the pillar. The latter approach would nowadays normally involve the use of numerical modeling. In many instances, a combination of these approaches is adopted.

The range of methods developed can be accounted for by the wide range of geological conditions encountered underground and the different functions that coal pillars must fulfill in different mining methods. It would be remarkable if a single design equation were applicable to the entire range of coal pillar types and conditions. The design approach employed should be relevant to both the geological conditions at the site and the function of the coal pillar being considered.

Stress measurements provide a tool that can assist in the study of pillars. Comparison of the results from different sites shows a wide range of potential strata conditions and resulting pillar characteristics. For pillars of moderate widths sufficient to allow the development of confinement within the coal, the stress measurements can be used to obtain estimates of the available pillar strengths or load-bearing capacities.

For wider pillars employed in deeper mines and with longwall layouts, characterizing pillars simply by their strength is less applicable. Such pillars are unlikely to fail in the sense of collapsing. However, the size of pillar employed can have a major influence on conditions in the surrounding entries. In this case, the distribution of stress within the pillars becomes more relevant, and the performance of pillars can be assessed by its impact on deformations and support requirements in the surrounding entries.

STRESS MEASUREMENT DATA

Measurement of stresses provides another tool for studying pillar behavior. During recent years, the stresses adjacent to mine roadways or entries have been measured at a number of coal mine sites in the United Kingdom. The results have been analyzed, and estimates of pillar strengths derived from them were compared with established pillar design equations [Cassie et al., in press]. The data and main points of the analysis are discussed here.

The general form of the results obtained was consistent with the confined core concept—the stresses are reduced immediately adjacent to the ribside and increase deeper into the strata. They provide a measure of the rate of increase of vertical stress actually obtained underground and can be studied with regard to their implications for the potential strength and behavior of pillars at sites where the confined core concept is considered valid.

Twenty sites have been included in this analysis where there were sufficient reliable results to allow the stresses to be characterized. At these sites, 63 stress measurements were available; they were carried out by overcoring hollow inclusion stress cells. Relevant data on the 20 sites are presented in table 1; individual test results are listed in table 2. Although only the vertical stress component has been used in this analysis and listed in the table, the measurement technique employed provides all six stress components. Knowledge of these can be invaluable in assessing the reliability of individual tests and interpreting overall behavior at a site.

The results were collated from several field investigations that have been previously reported and analyzed on a site-by-site basis [Hendon et al. 1995; ECSC 1997a, 1997b, 1998]. In several instances, the primary objective of the measurements was to investigate mine entry, rather than pillar behavior. The extraction geometries varied widely, including individual entries unaffected by other mine openings, twin-entry developments, room-and-pillar panels, and yield pillars. Working depths at the sites ranged from <200 m to >1,000 m. Site T was located at Jim Walter Resources, Inc.'s No. 7 Mine in Alabama; all other sites were in the United Kingdom.

Table 1.—Measurement sites

Site	Depth, m	Seam height, m	Roadway height, m	Mining geometry	Deformation level
A	620	7.5	3.5	Single-entry gate road	High.
B	500	3.0	2.9	20-m pillar	High.
C	500	3.0	2.9	30-m pillar	High.
D	480	2.5	2.7	30-m pillar	High.
E	950	2.2	2.8	20-m pillar	High.
F	950	2.2	2.8	Single-entry gate road	High.
G	900	2.2	3.0	Single-entry gate road	High.
H	800	1.5	3.0	Irregular pillar	High.
I	950	2.4	3.0	60-m pillar	High.
J	840	2.2	2.8	Single-entry gate road	Low.
K	840	2.2	2.8	Yield pillar trial	Low.
L	320	2.8	2.9	Single-entry gate road	Low.
M	400	3.0	3.7	Trunk roadway	Low.
N	480	2.7	2.6	Single-entry gate road	Low.
O	560	2.5	2.9	Single-entry gate road	Low.
P	700	2.0	4.0	Trunk roadway	Low.
R	1,060	2.6	3.0	Trunk roadway	Low.
S	1,085	2.6	4.1	40-m pillar	Low.
T	560	2.5	2.5	Multientry gate road	Low.
U	180	1.2	1.2	11-m pillar	Low.

Table 2.—Measurement data

Site	Height above roof, m	Distance into ribside, m	Vertical stress, MPa	Site	Height above roof, m	Distance into ribside, m	Vertical stress, MPa
A . . .	3.2	4.0	5.9	L	1.8	1.7	6.3
A . . .	4.5	5.7	8.2	L	1.6	3.4	[1]7.6
A . . .	5.0	9.4	14.1	L	2.1	6.4	[1]7.8
B . . .	4.6	3.9	7.4	L	2.0	10.0	[1]8.0
B . . .	4.6	6.2	10.5	M	3.1	1.1	10.0
B . . .	4.6	6.4	15.2	M	3.2	2.6	14.8
B . . .	4.6	8.1	17.5	M	3.0	4.3	[1]15.5
C . . .	4.6	4.2	9.0	M	6.6	10.7	[1]13.8
C . . .	4.6	6.9	8.7	N	3.5	1.5	9.0
C . . .	4.6	8.6	15.0	N	3.5	3.0	16.9
C . . .	4.6	11.7	[1]15.7	N	3.6	7.0	[1]11.4
D . . .	1.4	2.5	6.0	N	3.6	7.5	[1]10.8
D . . .	1.2	4.1	10.3	O	4.8	2.9	13.3
E . . .	4.8	4.6	8.8	O	5.0	5.4	[1]19.8
E . . .	5.2	7.2	10.6	O	5.0	7.4	[1]15.6
E . . .	3.9	9.6	20.0	P	3.8	1.9	10.0
F . . .	1.5	2.2	4.6	P	3.6	3.0	14.7
F . . .	2.9	4.2	11.3	P	3.3	4.8	19.5
F . . .	4.0	5.9	13.7	P	6.5	8.1	[1]18.5
G . . .	5.3	2.8	5.0	R	0.6	0.8	2.6
G . . .	4.2	3.7	9.5	R	1.7	2.4	12.0
G . . .	6.3	6.1	15.2	R	1.8	3.2	17.1
G . . .	6.8	10.9	24.5	R	3.5	4.7	21.6
H . . .	3.0	3.0	5.5	S	1.7	1.1	15.4
H . . .	5.9	5.2	8.9	S	1.5	3.0	26.7
H . . .	4.2	7.3	14.1	S	1.5	6.1	30.0
I	1.0	1.5	1.1	T	1.0	2.5	16.5
I	2.2	3.0	8.5	T	1.0	5.0	19.4
I	3.5	3.9	18.2	T	1.0	10.0	[1]21.0
J . . .	2.2	5.6	26.0	U	1.6	1.0	8.4
K . . .	2.6	4.1	11.7	U	1.8	3.3	22.3
				U	1.7	5.2	[1]23.5

[1]Postpeak.

ANALYSES OF DATA

For consistency and ease of interpretation, it would have been preferable to conduct the tests in the coal seam. However, because of the need for sufficiently competent strata in which to conduct the overcore tests, they were conducted above, rather than within, the coal seam, with the height above the roof dependent on the strength and condition of the roof at the site. At each site, several tests were conducted at varying distances from the mine entry (figure 1). Those tests deeper into the strata and judged to be beyond the sector of increasing stress (i.e., postpeak) were omitted from the analyses (figure 2). A tendency for the data to form two groups with different rates of stress increase was evident (figure 3). It was also observed that the sites where the rate of stress increase was lower were characterized by large and deep-seated strata deformations. These sites were all at depths >480 m. The stress gradients measured were lower than for similar data from sites in the United States [Mark and Iannacchione 1992].

The lower rate of stress increase observed at sites where the strata deformations around roadways were large was not unexpected. The rate at which the vertical stress can increase will be related to the degree of confinement that the roof and floor provide to the coal seam. If the roof and floor provide a high degree of confinement to the coal in the ribside, the stress it can sustain will increase rapidly with distance from the ribside. The frictional properties of the coal and its bounding strata will influence this. The amount of failed or yielding ground surrounding a roadway will also have a large influence. If the roof and/or floor are themselves deforming, the confinement that they can provide to the coal ribside will reduce, as will the rate at which the vertical stress can increase. This is consistent with the correspondence observed between the measured stresses and entry deformations.

The nonzero stresses at the ribside indicated by the results in figure 3 are worth noting here. They may be a consequence of the stresses being measured above, rather than within, the seam. Very low stresses in the immediate yielded coal ribside, which increase rapidly with distance into the ribside, would be expected to result in nonzero stresses in the roof immediately above the coal rib. Measuring the stresses in the roof may therefore average out the stress variations in the seam.

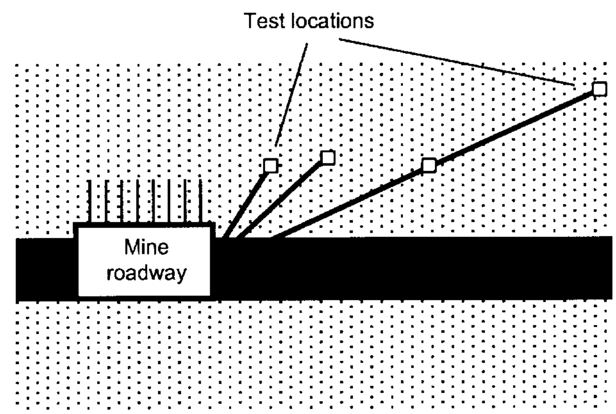

Figure 1.—Typical measurement site.

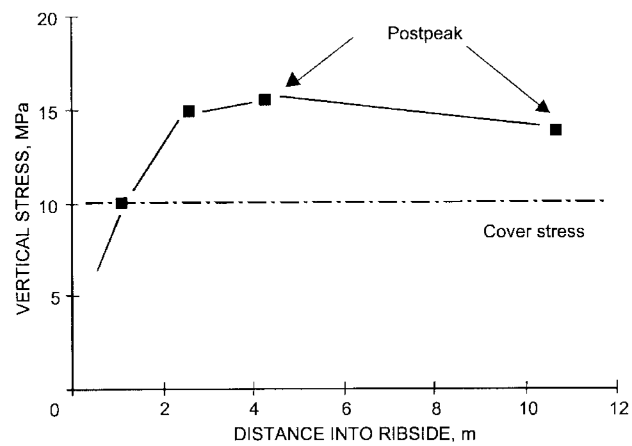

Figure 2.—Interpretation of test results.

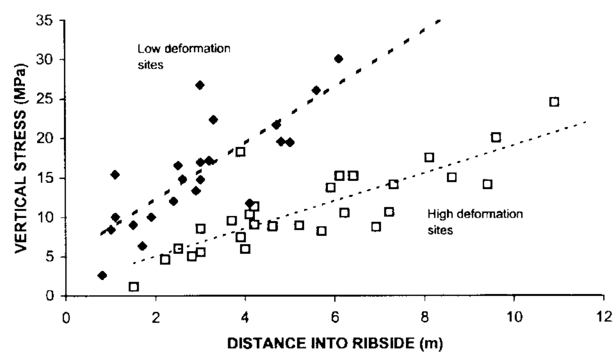

Figure 3.—Measured data from high and low deformation sites.

ESTIMATES OF PILLAR STRENGTHS

Pillar load-bearing capacities were estimated from the measured stress data with the assumption that the stress is related linearly to distance from the ribside normalized with respect to roadway height. Utilizing the measured stress data in this manner could underestimate pillar strengths. They provide an estimate of stresses that can be sustained in the ribside, but not necessarily of the maximum stresses. Given that the stress distribution in the ribside may be expected to be nonlinear (with the gradient increasing deeper into the pillar), assuming a linear distribution will also tend to underestimate pillar strengths when extrapolated to greater pillar widths. The linear estimates of pillar strength have been obtained not because it is proposed that they be adopted as a design equation, but rather to enable a comparison with the values given by recognized equations.

The formulas used as a basis for comparison were those presented by Bieniawski [1984], Wilson [1983], and the Salamon squat pillar equation with the parameters described by Wagner [1992]. An in situ coal compressive strength of 6 MPa was used in the Bieniawski formula.

Using results from sites typified by low deformations, the strengths were similar to those obtained using the Bieniawski equation and the Salamon squat pillar formulas (figure 4). This

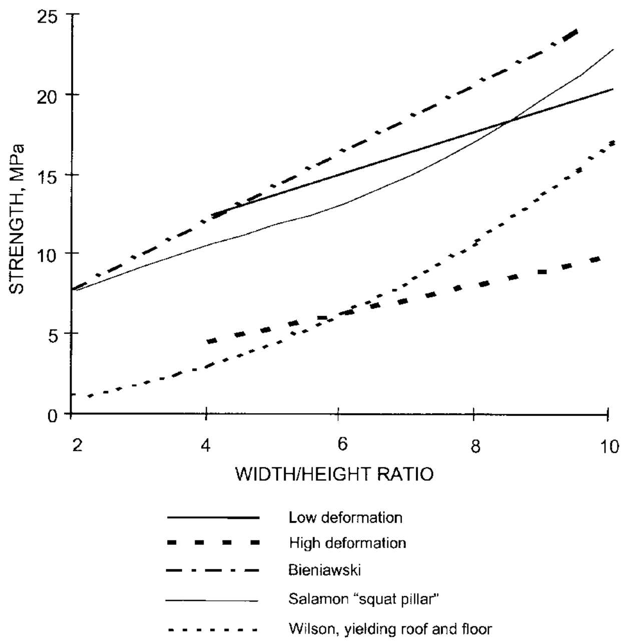

Figure 4.—Comparison of pillar strength estimates.

was making use of the average or regressed stress distribution. Estimates obtained for single sites within this group would imply strengths significantly in excess of or below these values. The Bieniawski and Salamon formulas were derived from back-analysis of failed and unfailed pillars or from testing of rock and coal specimens with different sizes and shapes; they have been widely recognized and applied to room-and-pillar layouts. In the case of the formulas, the strength at low width-to-height (w/h) ratios is associated with the in situ coal strength. For the estimates derived from the stress measurements, it is associated with the nonzero intercept obtained from linear regressions of the data. Despite this conceptual difference, the correspondence with the strength estimates for the low deformation sites is striking.

The pillar strengths implied using results from sites typified by high deformations were considerably lower. They indicate that, in these cases, strengths obtained using the same formulas and parameters could represent an overestimate. Significantly lower in situ coal strengths would be required to obtain a match with the measured data. Given that these equations are rooted in experience and the degree of acceptance that they have gained, in the mining environments where they are applied the strata conditions giving rise to the lower pillar strengths cannot be widely encountered. This could largely be accounted for by the observation that all of the stress measurement sites categorized as high deformation were at depths of 480 m or more; room-and-pillar mining operations are mostly at depths less than this. Not all of the deeper sites fell into the category of high deformation with weaker pillars. At one of the deepest sites (>1,000 m), analysis of the measured results and experience indicated pillar strengths significantly greater than the estimates provided by the equations used in figure 4. The weaker pillar strengths are in closer agreement with those estimated using Wilson's equations.

The measured stress data imply a wide range of possible pillar strengths depending on whether a site falls into the high or low deformation categories used here. Using a set of case histories that includes some of the sites listed here, two types of behavior were similarly identified by Gale [1996]. He noted that the identification of two groups is somewhat arbitrary and it may be expected that the full range of behaviors between these extremes could be encountered.

It is possible that part of the apparent variation in pillar strength inferred from the measured stresses was associated with variations in the in situ uniaxial compressive strength (UCS) of the coal. However, the form of behavior assumed in interpreting the measured stress data implies that the coal in the ribside had already yielded (with a reduction in cohesion) and that its strength was due to its frictional properties and confinement rather than cohesion. This would suggest that variations in the coal's UCS were unlikely to have a major influence. A study by Mark and Barton [1996] suggested that variations in laboratory test values for coal UCS were poorly correlated with pillar strengths determined by back-analyses of failed and unfailed cases.

It appears that for the sites considered here the degree of confinement provided to the coal seam was a major factor in determining the pillar strength. If the roof and/or floor are themselves yielding and deforming, the confinement that they can provide to the coal ribside will reduce, as will the rate at which the vertical stress can increase, thus leading to a weaker pillar. This is consistent with the marked correlation between the measured stresses and roadway deformations and is largely equivalent to the distinction between the cases of rigid or yielding roof and floor made by Wilson.

COMPARISON WITH NUMERICAL MODELING

Computer modeling has been used to investigate pillar or entry behavior at the various sites in conjunction with the field measurements. The model parameters used and results presented here were not intended to represent any individual site; rather, they illustrate the strata behavior and properties that may explain the measured data, in particular, the influence of the strata bounding the coal pillar.

The main parameters are summarized in table 3. Plane strain was assumed with two-dimensional cross sections of pillars being represented and boundary conditions set to define vertical axis of symmetry through the center of both the pillar and adjoining roadway. Initial stresses were applied and the roadway excavated to form the pillar. The loading on the pillar was then increased in several stages by displacing the upper and lower boundaries of the model grid. Results obtained for two cases are included. In the first, a uniformly strong host rock has been used; in the second, 3.0 m of weaker strata have been included above and below the seam. In other respects, the properties were identical. A cohesion equivalent to an in situ UCS of 6 MPa was used for the coal.

Table 3.—Modeling parameters

Modeling code	FLAC (version 3.3).
Initial stresses, MPa	5 (sxx, syy, and szz).
Dimensions:	
Seam height, m	2.4
Roadway height. m	2.4
Roadway width, m	4.8
Pillar width, m	20.0
Strata sequence:	
Case 1	Host rock and seam only.
Case 2	3.0 m of weak strata in roof and floor.

Material properties	Coal	Host rock	Weak strata
Density, kg/m^3	1,500	2,500	2,500
Bulk modulus, GPa	1.5	12.0	6.0
Shear modulus, GPa	1.0	7.0	3.5
Cohesion, MPa	1.6	12.0	4.0
Friction angle, °	35	40	30
Tensile strength, MPa	0.8	6.0	2.0
Residual cohesion, MPa	0.1	0.1	0.1
Residual friction angle, °	35	40	30
Dilation angle, °	0	0	0

In the case of the stronger strata, yielding was effectively confined within the coal seam. The vertical stresses in the ribside increased progressively, and large stresses developed as loading proceeded (figure 5). Examining the stresses at a horizon 3 m above the seam, the results were compared with the measured data that were also obtained from above the seam, although not at a constant horizon. The model results show the rate of stress buildup increasing as the pillar was loaded. For average stresses across the pillar corresponding to the range likely to be encountered in practice, they lay through the measured data from low deformation sites. Given sufficiently strong roof and floor strata, very high pillar strengths can be developed.

With weaker strata introduced in the immediate roof and floor, the behavior was similar for the initial load stages (figure 6). As the loading was increased, the roof and floor started to yield and the rate of stress buildup in the ribside reduced. For the final load stages, yielding of the roof and floor had fully developed, spread across the width of the pillar being modeled, and the stresses settled to an approximately constant residual distribution. For these latter stages, the stress distribution was irregular due to the development of bands of strata that were actively shearing with the stresses at yield; between these bands, the stresses are below yield. The trend of model results matched those of the measured data at high deformation sites.

For the strata properties and loading path used in this example, the weaker strata model exhibits a postpeak reduction in strength to a residual value (figure 7). The loss of pillar strength was associated with the reducing confinement as the strata bounding the coal seam yielded, rather than a reduction in coal strength. Should the initial stresses be sufficient to cause the roof and floor to yield and deform as the entries and pillar were formed, there would be no apparent loss in pillar strength by this mechanism and the postpeak strength would be applicable from the outset. In this way, the initial stresses, in addition to the strata properties, may influence pillar behavior.

Numerical modeling allows an improved interpretation of measured data. The influence of more factors can be taken into account, and it provides a better means of extrapolating to

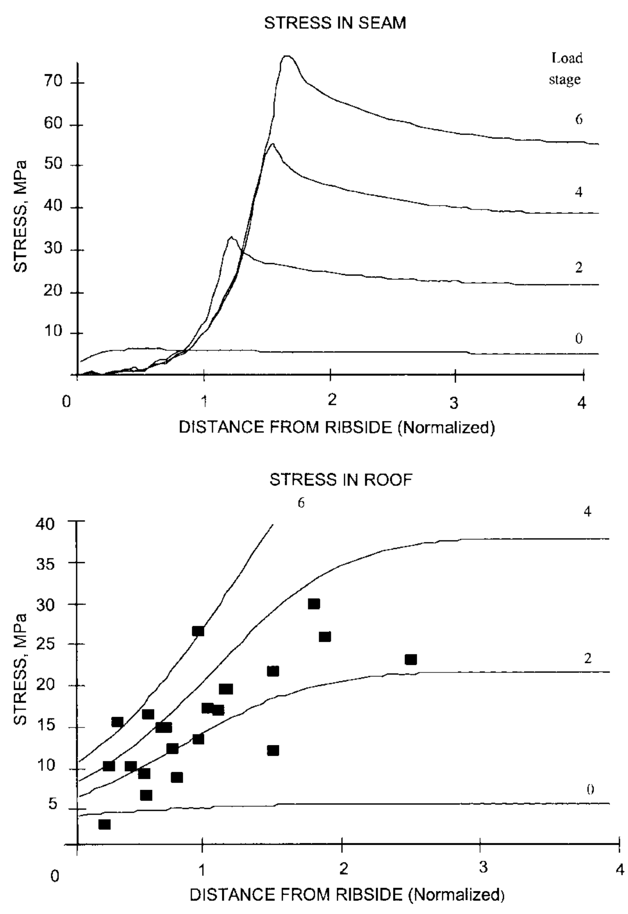

Figure 5.—Strong roof and floor strata.

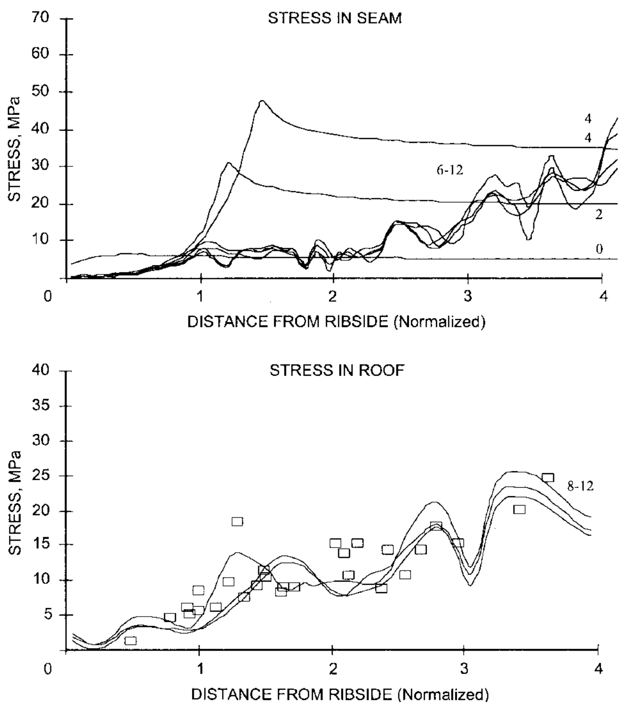

Figure 6.—Weak roof and floor strata.

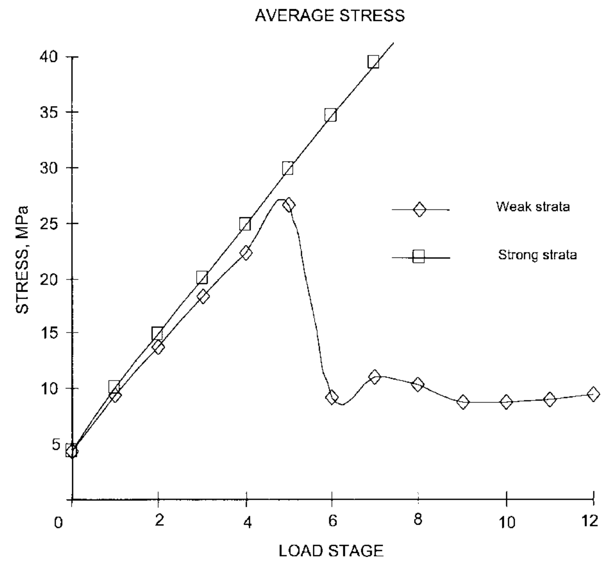

Figure 7.—Modeled pillar loads.

different geometries or loading. In addition, the interaction between pillars and the surrounding entries can be assessed and taken into account. In many circumstances, particularly with wider pillars, considerations of entry rather than pillar stability may be the limiting factor.

WIDE PILLARS

With large w/h ratios, it is widely accepted that the probability of pillar failure and loss of strength decreases. Nevertheless, excessive loading of the pillars may result in damage to the surrounding mine entries. For deeper mines and those using longwall mining methods, pillar w/h ratios frequently exceed those for which the most widely known strength equations were derived. In these circumstances, it is likely that pillar dimensions will be limited by considerations of the stability of the surrounding mine entries, rather than that of the pillars.

Design of pillars or pillar systems to maintain acceptable conditions in the surrounding entries is likely to lead to consideration of the nonuniform stress distribution across pillars, rather than simply the average stress or total load acting through a pillar. Although a simplification, one possible approach is to limit the maximum stress or the stress at a particular location expected within a pillar. This approach was adopted by Wilson with his "entry stability" as opposed to "ultimate stability" criteria for pillar strength [Carr and Wilson 1982].

The choice of a suitable limiting value for the stress is fundamental to this approach. Wilson related the maximum allowable stress to the triaxial strength of the strata and the in situ vertical stress. Other estimates are possible, although it is likely to depend in some degree on the surrounding strata strength. In some regards, the choice of this value is analogous to the problem of determining the appropriate value for the in situ coal strength for use in pillar strength equations such as Bieniawski's.

The wide range of entry conditions encountered at sites subject to similar stress levels, but with different strata properties, suggests that appropriate values for the maximum stress to allow in a pillar may vary widely from site to site. The variation may be greater than that apparent in effective in situ coal strengths.

An advantage for using numerical modeling in investigating pillar behavior is that it enables consideration of the interaction between pillars and the surrounding entries. Mine entry conditions are, of course, influenced by factors other than surrounding pillars. This should be taken into account if adopting an approach of using favorable mine entry conditions as an objective of pillar design.

PROTECTION PILLARS BETWEEN LONGWALL PANELS

The pillars left between longwall panels are a particular case of wide pillars as described above. The method of longwall retreat typically employed in U.K. coal mines uses a single gate at each side of the panel, with adjacent panels separated by wide protection pillars (figure 8). The tailgate for the next in a sequence of longwall panels is driven during or subsequent to retreat of the previous panel. As a result, the tailgate may be driven in a stress regime that is subsequently altered by extraction of the previous panel, one that has already been altered, or a combination of these.

Pillar widths that have been adopted for recent layouts of this type in the United Kingdom are shown in figure 9. They clearly come into the category of wider pillars (the w/h ratios range up to 40:1). Coal pillars of these dimensions do not fail in the normally accepted sense. Despite this, the use of inadequate pillars may result in difficult mining conditions.

The choice of pillar dimensions may influence—

1. The stress change due to extraction of the previous panel and hence conditions in the tailgate while or after it is driven;
2. The concentration of stress and hence conditions at the tailgate-faceline junction during retreat; and
3. The surface subsidence profile across the sequence of panels.

The first and second of the above will almost certainly be considered in determining the pillar size. The third may be considered if the surface is subject to subsidence limitations.

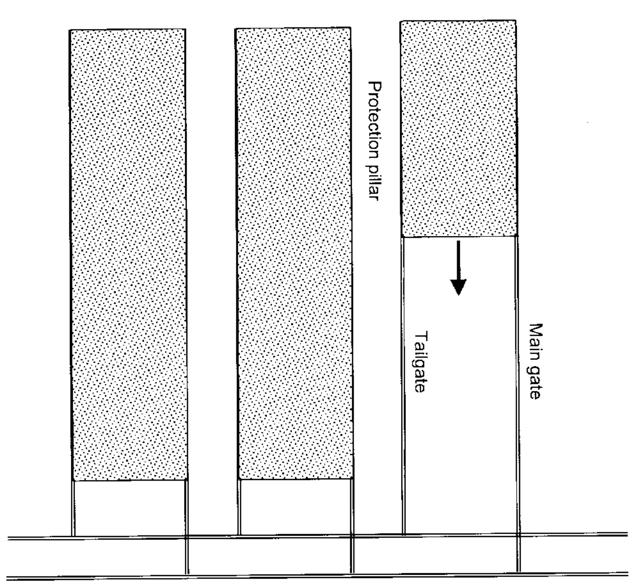

Figure 8.—Typical longwall retreat layout in U.K. coal mines.

Wilson's pillar equations were originally developed as a method for determining dimensions for this kind of pillar. The method estimates the distribution of stresses transferred onto the pillar due to extraction of the panels. It effectively limits the stress at the location of the tailgate with the first panel extracted and the maximum stress across the pillar with both panels extracted. Numerical modeling can now be used to provide a more sophisticated estimate of how the stresses will be distributed across the pillar. It will, however, be strongly dependent on the caving behavior of the longwall and the reconsolidation of the waste that remains subject to considerable uncertainty. Suitable limits to place on the stress levels must also be determined for the site, as described earlier.

Roof displacements showing the influence on gate conditions of stresses distributed over substantial pillars such as these are shown in figures 10-12. The data are from telltale devices used to measure roof deformations [Altounyan and Hurt 1998]. Their purpose is to provide a routine assessment of roof condition, rather than acting as field measurement stations for research purposes. However, the data obtained can be used to enable a comparison between different entries and sites.

In figure 10, a histogram compares data from the tailgate and main gate for a panel at an average depth of 590 m with a 50-m pillar. At this depth, the pillar width is at the lower range in figure 9. For the main gate, none of the instruments showed

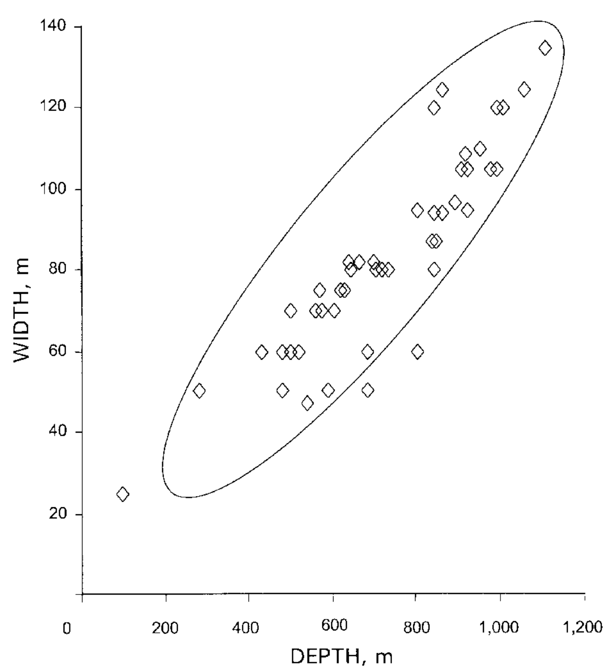

Figure 9.—Pillar widths between retreat longwall panels.

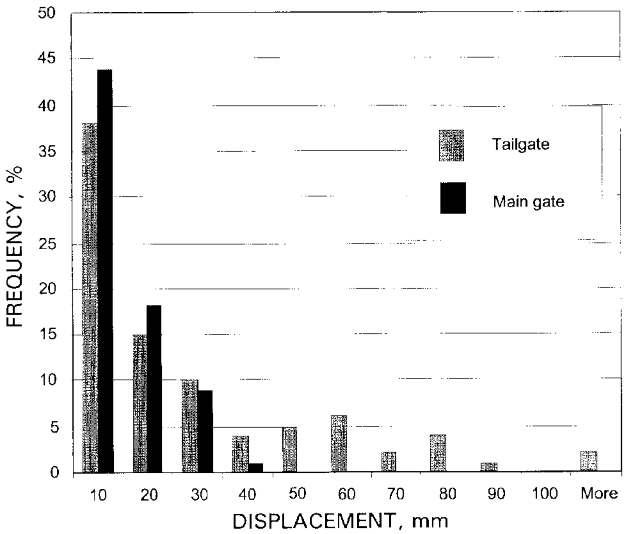

Figure 10.—Comparison of roof displacements in main gate and tailgate during development.

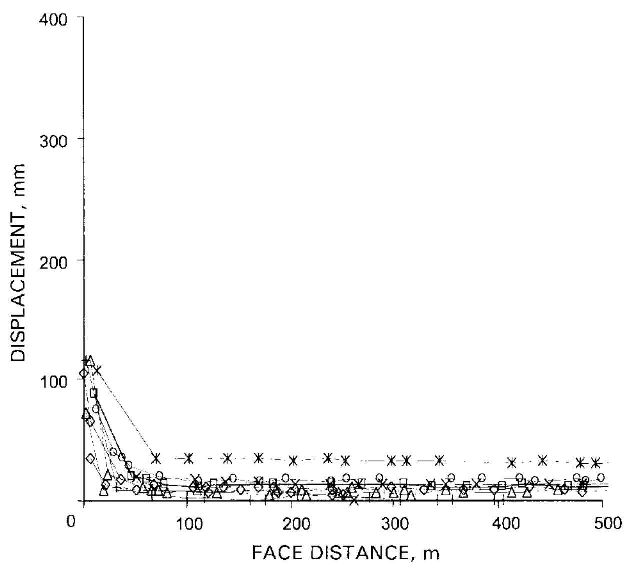

Figure 11.—Roof displacements in main gate during retreat.

displacements in excess of 40 mm; in the tailgate, 20% exceeded this value. There was considerable spread in the roof deformations along the length of each gate; this can be expected due to geological variations. The form of the distributions suggests that in zones of weaker geology the increased stress levels experienced by the tailgate resulted in increased roof displacements. The displacements plotted were those recorded up to 50 days after drivage of the gate; the difference between the gates increased with time and during retreat of the panel.

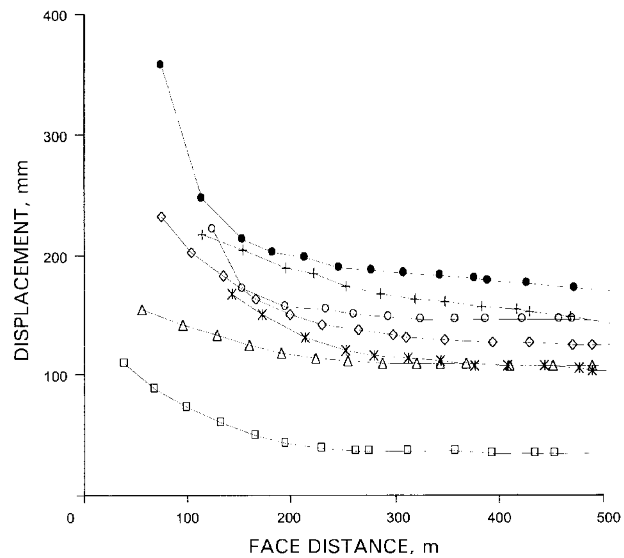

Figure 12.—Roof displacements in tailgate during retreat.

Increasing roof displacements as the retreating panel approaches are plotted in figures 11 and 12. For the main gate (figure 11), its influence only becomes apparent within the final 50 m. The displacements in the tailgate (figure 12) are larger and start to accelerate at an earlier stage than for the main gate. In fact, tailgate conditions for this panel were poor with large amounts of convergence and roof softening. A considerable amount of extra support had to be installed in the tailgate to maintain stability up to the junction with the faceline. The different amount of support employed in the gates needs to be taken into account in comparing figures 11 and 12.

Variability in conditions such as that evident in figures 10-12 may provide a guide in determining suitable pillar dimensions. If the difference between main gate and tailgate attributable to increased stress is small compared to the spread due to geological variability along the length of each gate, there is little point in increasing pillar widths in order to improve conditions in subsequent tailgates.

Although pillar dimensions are usually described with regard to consideration of vertical stresses and their effects, many other factors can also affect longwall gate conditions and influence the choice of suitable pillar dimensions. These include—

- Horizontal stresses and their orientation relative to the panel;
- Timing of gate drivage relative to the previous panel; and
- Interaction with workings in other seams.

If significant interaction is expected, this may be the dominant consideration in determining the position of the tailgate and thus the pillar size. These are technical factors and are not the sole determinants of pillar size. The choice of pillar size will also be strongly influenced by the priorities of the mine management or operator. If the priority is to maximize extraction, smaller pillars are likely to be adopted, with adverse conditions in the tailgate giving rise to increased repair and support costs being accepted. If the priority is to minimize production costs, larger pillars are likely to be adopted.

SUMMARY

Comparison of stress measurement results from different sites, mostly in U.K. mines, shows a wide range of potential strata conditions and resulting pillar characteristics. The range can be accounted for by variations in the degree of confinement provided to the coal by the roof and floor strata. The lower pillar strengths inferred from measured stress data were encountered at deeper sites with weak roof or floor strata and characterized by large deformations. Such sites are likely to employ mining methods other than room- and-pillar and use wide pillars. Although the wider pillars employed between longwall panels may not fail in the usual sense, their dimensions can have a critical impact on conditions in the surrounding entries or gates.

For wide pillars, it is likely that pillar dimensions will be limited by considerations of the stability of the surrounding mine entries rather than of the pillars. This requires that factors other than pillar strengths and load be taken into account. A possible general approach is to establish stress levels that are acceptable for a site and dimension pillars so that these stress levels are not exceeded and to consider the pillar in context with the stability of the entries.

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ANALYSIS OF LONGWALL TAILGATE SERVICEABILITY (ALTS): A CHAIN PILLAR DESIGN METHODOLOGY FOR AUSTRALIAN CONDITIONS

By Mark Colwell,[1] Russel Frith, Ph.D.,[2] and Christopher Mark, Ph.D.[3]

ABSTRACT

This paper summarizes the results of a research project whose goal was to provide the Australian coal industry with a chain pillar design methodology readily usable by colliery staff. The project was primarily funded by the Australian Coal Association Research Program and further supported by several Australian longwall operations.

The starting point or basis of the project was the Analysis of Longwall Pillar Stability (ALPS) methodology. ALPS was chosen because of its operational focus; it uses tailgate performance as the determining chain pillar design criterion rather than simply pillar stability. Furthermore, ALPS recognizes that several geotechnical and design factors, including (but not limited to) chain pillar stability, affect that performance.

There are some geotechnical and mine layout differences between United States and Australian coalfields that required investigation and, therefore, calibration before the full benefits offered by the ALPS methodology could be realized in Australia.

Ultimately, case history data were collected from 19 longwall mines representing approximately 60% of all Australian longwall operations. In addition, six monitoring sites incorporated an array of hydraulic stress cells to measure the change in vertical stress throughout the various phases of the longwall extraction cycle. The sites also incorporated extensometers to monitor roof and rib performance in response to the retreating longwall face.

The study found strong relationships between the tailgate stability factor, the Coal Mine Roof Rating, and the installed level of primary support. The final outcome of the project is a chain pillar design methodology called Analysis of Longwall Tailgate Serviceability (ALTS). Guidelines for using ALTS are provided.

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INTRODUCTION

In many cases, chain pillars in Australia have been designed solely with regard to pillar stability using a process similar to that used for pillars within bord-and-pillar operations. The bord-and-pillar approach is based on analysis of collapsed pillar cases from Australia and the Republic of South Africa [Salamon et al. 1996] and applies a factor of safety in relation to pillar collapse. This approach is inappropriate for a number of reasons when designing chain pillars.

Australian chain pillars typically have minimum width-to-height (w/h) ratios >8, which is approximately 4.5 standard deviations away from the mean of the pillar collapse case histories. In addition, the chain pillar loading cycle and active life are significantly different from those experienced by pillars within a bord-and-pillar operation. Finally, the goal of maintaining gate road stability is very different from that of avoiding a pillar collapse.

The need for a design method uniquely developed for Australian longwall chain pillars was clear. The original submission for funding by the Australian Coal Association Research Program (ACARP) stated that the calibration (to Australian conditions) of a proven chain pillar design methodology offered the least risk for a successful and timely outcome. It was assessed that the most comprehensive chain pillar design tool then available was the Analysis of Longwall Pillar Stability (ALPS) [Mark 1990; Mark et al. 1994]. The primary consideration in selecting ALPS is that it uses gate road (i.e., tailgate) performance as the determining chain pillar design criterion. Secondly, ALPS is an empirical design tool based on a U.S. coal mine database; thus, it provided a ready framework for calibration to Australian conditions.

The aim of the project was to provide the Australian coal industry with a chain pillar design methodology and computer-based design tool readily usable by colliery staff. A further objective was to ensure that the methodology developed by the project had the widest possible application to all Australian coalfields by identifying where local adjustments and limitations may apply.

In formulating the design methodology, the primary goal was to optimize pillar size (specifically pillar width) so as to—

- Maintain serviceable gate roads such that both safety and longwall productivity are unaffected;
- Minimize roadway drivage requirements so as to have a positive impact on continuity between successive longwall panel extraction; and
- Maximize coal recovery.

In designing chain pillars, specifically with regard to satisfactory gate road performance, the following design criteria were proposed:

- The chain pillar must provide adequate separation between the main gate travel road and belt road, such that the travel road (tailgate of the subsequent longwall panel) will be satisfactorily protected from the reorientation and intensification of the stress field caused by the extraction of the first longwall panel.
- The tailgate (with a focus on the tailgate intersection with the longwall face) will be sufficiently serviceable for ventilation and any other requirements (setting of secondary support, second egress, etc).

BACKGROUND

ALPS was originally developed by Mark and Bieniawski [1986] at The Pennsylvania State University. It was further refined [Mark 1990, 1992; Mark et al. 1994] under the auspices of the former U.S. Bureau of Mines (USBM).[4] The initial ALPS research involved field measurements of longwall abutment loads at 16 longwall panels at 5 mines. These measurements were used to calibrate a simple conceptualization of the side abutment, similar to models proposed by Wilson [1981] and Whittaker and Frith [1987]. The side abutment (A) equates to the wedge of overburden defined by the *abutment angle* (β) (see figure 1). The tailgate loading condition is considered to be some percentage of the side abutment, called the *tailgate abutment factor* (F_t). The U.S. field measurements found a range of abutment angles, from $\beta = 10.7°$ to $\beta = 25.2°$. A value of $\beta = 21°$ and $F_t = 1.7$ was selected for use in design.

[4] The safety and health research functions of the former U.S. Bureau of Mines were transferred to the National Institute for Occupational Safety and Health in October 1996.

Because of the encouraging results obtained from the initial study, the USBM commissioned further research directed toward quantifying the relative importance of roof and floor quality and artificial support on gate road performance. The approach was to analyze actual longwall mining experience. Case histories from 44 U.S. longwall mines were characterized using 5 descriptive parameters. Pillar design was described by the ALPS stability factor (ALPS SF = pillar strength ÷ pillar load); roof quality was described by the Coal Mine Roof Rating (CMRR) [Molinda and Mark 1994; Mark and Molinda 1996]. Other rating scales were developed for primary support, secondary support, and entry width.

Mark et al. [1994] reported that statistical analyses indicated that in 84% of the case histories the tailgate performance (satisfactory or unsatisfactory) could be predicted correctly using only the ALPS SF and the CMRR. It was further stated that most of the misclassified cases fell within a very narrow borderline region. The analyses also confirmed that primary

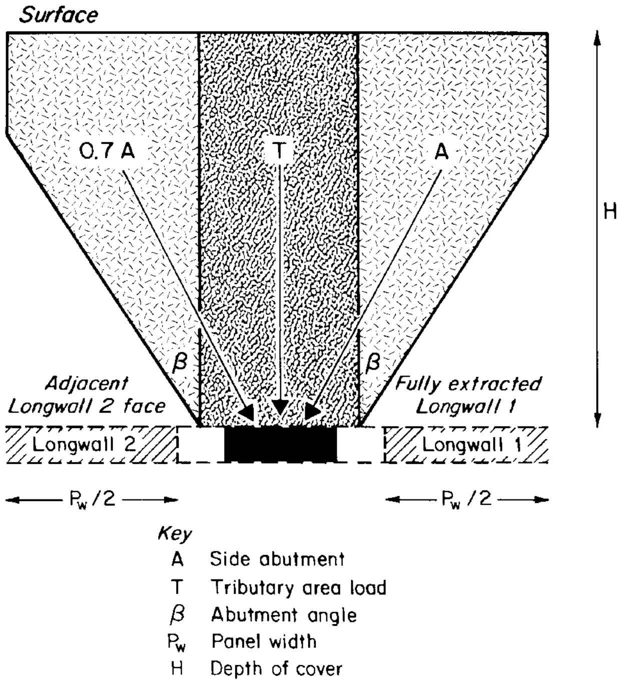


Figure 1.—Conceptual model of the side abutment load.

roof support and gate entry width are essential elements in successful gate entry design. The relative importance of the floor and of secondary support installed during extraction could not be determined from the data.

The following equation (relating the ALPS SF and CMRR) was presented to assist in chain pillar and gate entry design:

$$\text{ALPS SF}_R = 1.76 - 0.014\ \text{CMRR}, \tag{1}$$

where the ALPS SF_R is the ALPS SF suggested for design.

The Primary Support Rating (PSUP) used in ALPS was developed as an estimate of roof bolt density and is calculated as follows:

$$\text{PSUP} = \frac{L_b \times N_b \times D_b}{S_b \times w_e \times 84}, \tag{2}$$

where L_b = length of bolt, m,

N_b = number of bolts per row,

D_b = diameter of the bolts, mm,

S_b = spacing between rows of bolts, m,

and w_e = entry (or roadway) width, m.

PSUP treats all bolts equally and does not account for load transfer properties, pretensioning effects, etc.

NEED FOR CALIBRATION

Conventional longwall mines in the United States generally use a three-heading gate road system; Australian longwall panel design typically employs a two-heading gate road system with rectangular chain pillars separating these gate roads. A typical Australian longwall panel layout is presented in figure 2. Figure 2 also details the stages of the chain pillar loading cycle:

1. Development loading (calculated using tributary area concepts);
2. Front abutment loading, which occurs when the first longwall face is parallel with the pillar;
3. Main gate (side) abutment loading, when the load has stabilized after the passage of the first face;
4. Tailgate loading, when the second face is parallel with the pillar; and
5. Double goafing, when the pillar is isolated between two gobs.

It is during tailgate loading that the chain pillar (or cross section thereof adjacent to the tailgate intersection) experiences the greatest vertical loading during its "active life," i.e., the period where the chain pillar is playing its role in helping to maintain satisfactory gate road conditions. This project focused on tailgate performance (at the T-junction) as the design condition. The pillar stability factor in relation to the tailgate loading condition is designated as the "tailgate stability factor" (TG SF).

The project found that Australian chain pillars have an average length-to-width ratio of 3.2; crosscut centers on average are spaced at 100 m. The pronounced rectangular shape of Australian chain pillars may add strength to the pillar compared to a square pillar of the same minimum width. Mark et al. [1998b] reanalyzed the U.S. database using the Mark-Bieniawski rectangular pillar strength formula and found a slightly better correlation (in relation to the predictive success rate) than using the Bieniawski equation. In addition to the Bieniawski equation, this project assessed both the Mark-Bieniawski rectangular pillar formula [Mark and Chase 1997] and the squat pillar formula [Madden 1988] in relation to the correlation between the pillar stability factor and the CMRR.

In Australia, the significant impact of horizontal stress on coal mine roof stability is well documented [Frith and Thomas 1995; Gale and Matthews 1992]. The in situ horizontal stresses should not have a significant direct influence on tailgate roof stability due to the presence of an adjacent goaf. However, there is an indirect influence in terms of the degree of damage done to the roof during the initial roadway development and

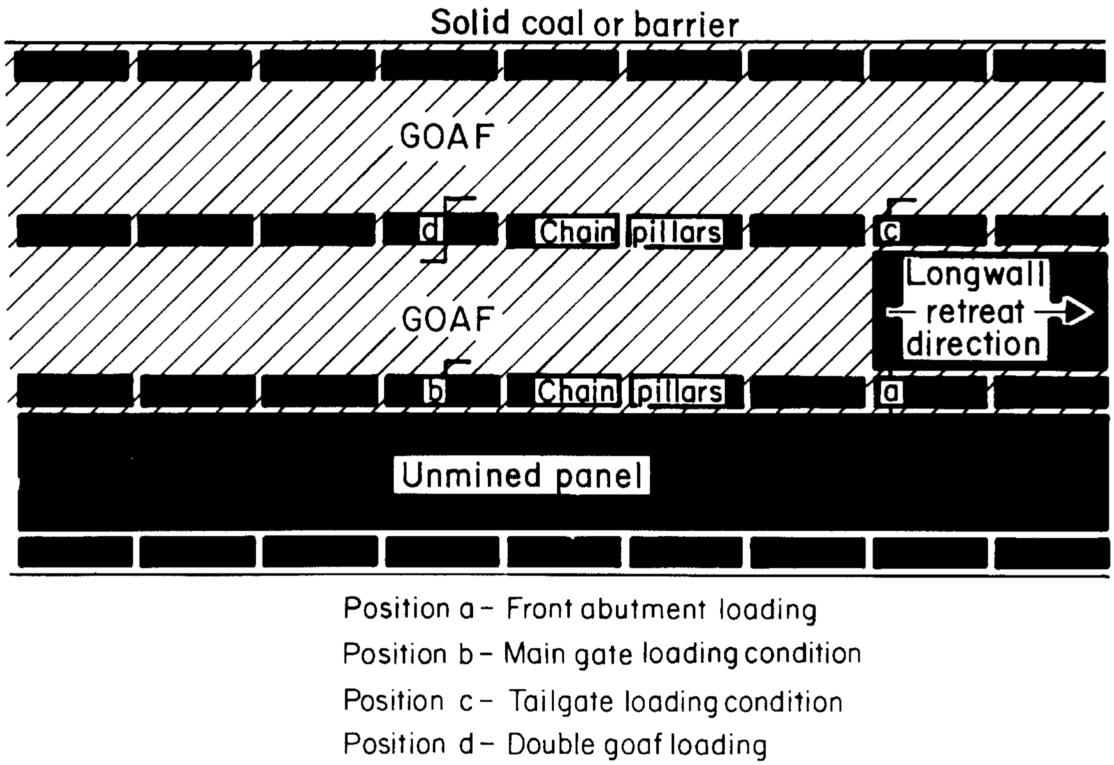


Figure 2.—Stages in the dynamic loading cycle of longwall chain pillars.

then to the main gate travel road and cut-throughs during longwall retreat. The effect of the in situ horizontal stress field on gate road serviceability (particularly on roof stability) is not taken into account directly by the ALPS methodology and was considered in more detail by the ACARP project.

Finally, the project aimed to verify the applicability of the ALPS loading parameters to Australian conditions. The ALPS methodology uses an abutment angle of 21° in all cases, and it assumes that the tailgate load is 1.7 times the side abutment load.

MEASUREMENTS OF AUSTRALIAN ABUTMENT LOADS

The project measured changes in vertical stress across (and within) chain pillars at six collieries to determine whether the ALPS approximations should be refined. Three sites were located in the Bowen Basin Coalfield in Queensland (Central, Crinum, and Kenmare Collieries), two were in the Newcastle Coalfield (Newstan and West Wallsend Collieries), and one was at West Cliff Colliery in the Southern Coalfield. Each monitoring site included an array of hydraulic stress cells (HSCs) generally located at midseam height to measure the changes in vertical stress. Most sites also included extensometers to monitor roof and rib performance. A general instrumentation layout is shown in figure 3.

The HSC used in this project is a modification of the borehole-platened flatjack developed by the former USBM. The HSC was developed, calibrated, and tested by Mincad Systems Pty. Ltd. [1997]. The HSC consists of a stainless steel bladder into which hydraulic fluid is pumped via tubing extending along the borehole. The bladder is encased between two steel platens that are forced against the borehole wall as the bladder is pumped up.

As with every stress measurement instrument, proper calibration is important. Mincad Systems provided two calibration formulas based on its research with the HSC. The formula used in this project employs a calibration factor K ' 1.0 for a stress increase of #5 MPa and K ' 1.3 for that portion of an increase above 5 MPa. Because ALPS is a comparative chain pillar design tool, it is not critical which calibration method is used as long as the method is consistent from site to site.

The six sites add considerably to the ALPS abutment load database. They include a much wider range of cover depths and width-to-depth ratios than the original U.S. data. There is also much more variety in the geologic environments. In addition, because the stress readings could be made remotely, monitoring was possible subsequent to the passing of the second longwall

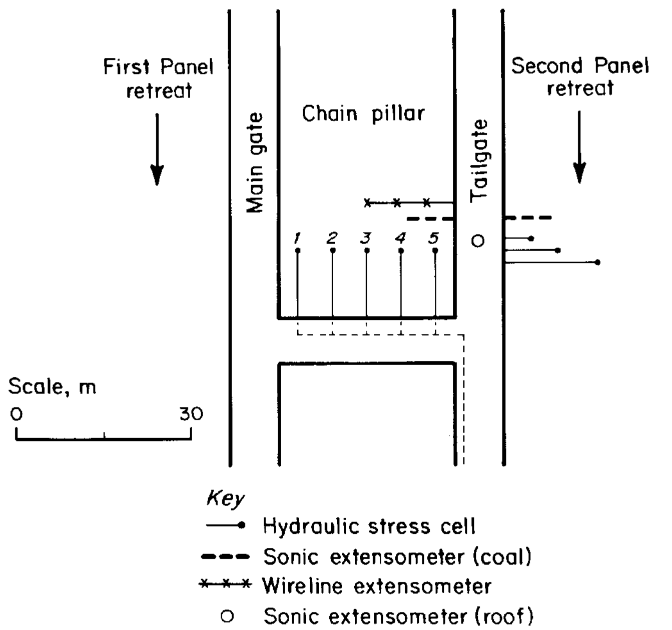


Figure 3.—Instrumentation layout at a typical stress measurement site.

face. Of the 16 original U.S. panels, there were sufficient data to characterize the side abutment load in only 6, and only one panel provided data on the tailgate abutment factor. In contrast, data on both the side and tailgate loads were obtained from all six Australian monitoring sites.

At the Australian sites, entry width and height ranged from 4.8 to 5.2 m and 2.5 to 3.6 m, respectively. Pillar width and length (rib to rib) ranged from 26 to 40 m and 95 to 125 m, respectively; cover depths varied from 130 to 475 m. Due to the relatively high length-to-width ratio of Australian chain pillars (i.e., extracted crosscut coal <5%), a plane strain or two-dimensional loading analysis is common in Australia and was considered appropriate by the Australian researchers. Furthermore, the Australian researchers recognized that the location of the stress cells within the pillar would in all probability affect the measured vertical stress changes. In placing the cells near a cut-through rather than across the longitudinal center of the chain pillar, the monitoring exercises were viewed as recording the loading behavior of a thin, two-dimensional slice of the pillar near a critical location during its "active life."

The ALPS loading parameters account for the extracted coal within the cut-throughs. Therefore, the abutment angles reported by the ACARP project [Colwell 1998] would be slightly different if the load had been addressed in the same manner as the U.S. field measurements in back-calculating the abutment angles. However, the end effect on the design chain pillar width is negligible.

Measurements of the main gate side abutment loading are used to calculate the abutment angle; measurements of the tailgate abutment (when longwall 2 is parallel with the instruments) are used to calculate the tailgate abutment factors. Examples of the data obtained from two of the sites are shown in figure 4. The results from all six monitoring sites are summarized in table 1 and figure 5 (along with the U.S. data).

The measurements of the abutment angle from the three Queensland mines and from Newstan Colliery clearly fall within the range of the U.S. data. However, the abutment angles calculated for the two deepest mines, West Wallsend and West Cliff, are the smallest of any in the database. The overburden at these two mines (and at Newstan Colliery) also contains the massive sandstone and sandstone/conglomerate strata commonly associated with the Newcastle and Southern Coalfields. The low width-to-depth ratio, along with the strong overburden, may be affecting the caving characteristics of the gob.

Table 1 also shows two sets of tailgate abutment factors. The first set was obtained by dividing the measured tailgate loading by the measured main gate (side abutment) loading. The second set, which is the one used in the U.S. version of ALPS, is obtained by dividing the measured tailgate load (adjacent to the T-junction) by the *calculated* side abutment load using an abutment angle of $ ' 21°. The one U.S. measurement found this second tailgate abutment factor to be 1.7. The Australian data in table 1 show a high variability, with the mean at 1.3 in relation to an ALPS-style analysis.

Figure 6 plots the development of the change in load during tailgate loading (as a multiple of the side abutment) against face position. It clearly indicates that the nature of the loading behavior at Central, Crinum, and Kenmare Collieries closely approximates that proposed by ALPS. However, the tailgate loading behavior at Newstan Colliery and particularly at West Wallsend Colliery reveals that the *double goaf load* is significantly greater than twice the measured main gate side abutment load. It is likely that West Cliff would have behaved in a manner similar to Newstan if the cabling and/or cells had not become inoperable with the second longwall face only 5 m past the instrumentation site.

The field data associated with Newstan, West Wallsend, and West Cliff Collieries clearly suggest that a *much* greater portion of the main gate abutment load is distributed onto the adjacent unmined longwall panel than calculated on theoretical grounds (see figure 2).

Although the double goaf loading condition could not be measured at West Wallsend Colliery, it would seem that the bulk of the tailgate load manifests itself within that distance 100 m outby of the face. There are distinct increases in the rate of loading at approximately 70 m and again at 20 m outby of the face. This correlates well with the observed tailgate condition and strata behavior.

In contrast to West Wallsend Colliery, the bulk of the tailgate load at Newstan Colliery manifests itself after the passage of the longwall face. Both Newstan and West Wallsend Collieries have experienced greater difficulties with regard to both gate road and face control issues when massive sandstone/conglomerate channels are within 0 to 30 m of the mining horizon. Face width optimization has played a critical role in alleviating the face control difficulties.

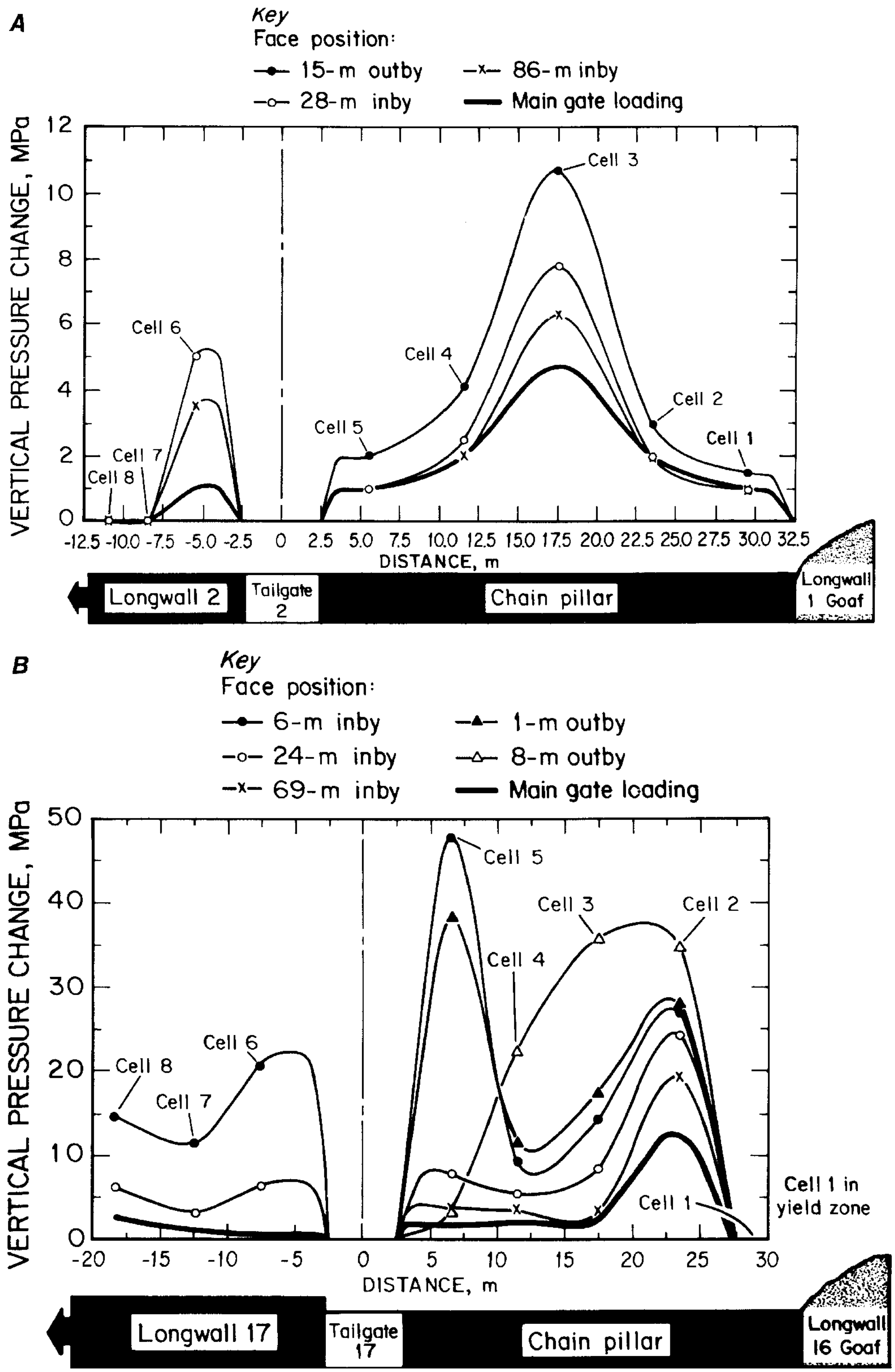


Figure 4.—*A*, Abutment load profiles at different locations of the longwall face (Crinum Colliery) with highly cleated coal. *B*, Abutment load profiles at different locations of the longwall face (West Wallsend Colliery), where the tailgate load is extremely aggressive.

Table 1.—Results of stress measurements

Monitoring site	H, m	w, m	w_p, m	P_w, m	$, °	F_t (Meas)	F_t (Calc)
Central	265	39.9	5.1	230	24.7	1.77	2.05
Crinum	125	30.2	4.8	275	19.1	1.52	1.35
Kenmare	130	24.8	5.2	200	19.2	1.49	1.22
Newstan	180	26.0	5.0	130	15.3	1.48	1.04
West Cliff	475	37.2	4.8	200	5.9	1.81	0.60
West Wallsend . .	240	30.1	4.9	145	8.5	3.79	1.52

NOTE.—$ and F_t (Meas) are based on two-dimensional analyses ((' 0.25 MN/m^3; Kenmare (' 0.23 MN/m^3). F_t (Meas) is based on ALPS loading parameters ($ ' 21° and (' 0.255 MN/m^3).

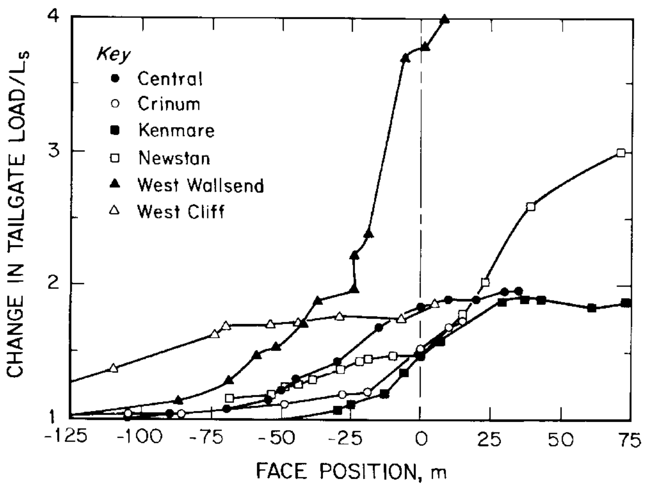


Figure 5.—Development of abutment load at the six monitoring sites.

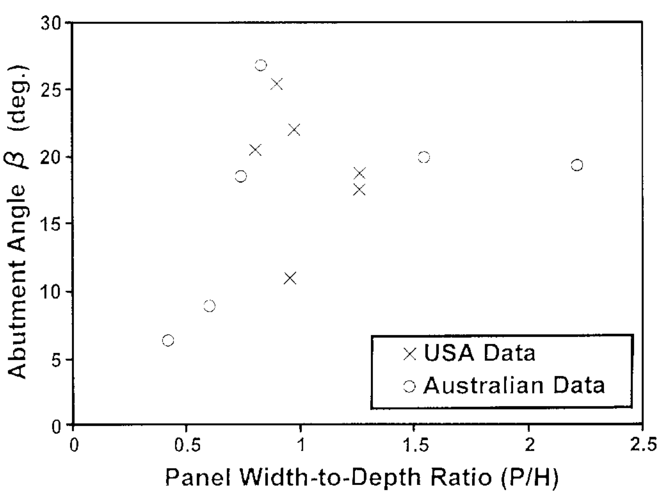


Figure 6.—Abutment angles determined from stress measurements.

A possible explanation for the variation in the manifestation of the tailgate load (in relation to face position) is that while a near-seam conglomerate channel exists in relation to the monitoring site at West Wallsend Colliery, it is absent at the Newstan Colliery site. The anecdotal evidence suggesting the near-seam channel as a possible cause of this variation in load manifestation is strong (i.e., secondary support requirements, seismic monitoring [Frith and Creech 1997]; however, the mechanics are not yet fully understood.

The stress measurements collected by the project were supplemented by data from similar investigations previously conducted by other collieries, which were gratefully made available to the project. The supplementary field data were obtained using nearly all of the different types of stress cells that have been used in Australia (CSIRO HI, IRAD, Geokon, and HSC). The variety of instruments hinders comparison between studies, yet some trends emerge.

In general, the supplementary field data support the observations made from the project data. In Bowen Basin collieries, the loading behavior closely approximates that proposed within ALPS. In contrast, there are some significant departures in New South Wales for collieries that have strong, spanning overburden and a low width-to-depth ratio. Table 2 indicates that at Angus Place, South Bulli, West Cliff, West Wallsend, and Wyee the measured side abutment angles are significantly less than 21°.

In summary, it seems that an abutment factor of 1.5, in conjunction with an abutment angle of $ ' 21°, is a reasonable and generally conservative approximation of the actual tailgate load for most Australian mines. The exceptions are two collieries and one locality (containing three collieries) within the Australian database, where there is sufficient evidence to suggest that site-specific loading parameters are more applicable. These are the Central and West Wallsend Collieries, and the deepest collieries within the Southern Coalfield (South Bulli, Tower, and West Cliff Collieries). For Central Colliery, the appropriate loading parameters seem to be $ ' 26° and F_t ' 1.6. With regard to the three Southern Coalfield collieries, the recommended loading parameters are $ ' 10° and F_t ' 1.5, which also apply to areas associated with West Wallsend Colliery that are unaffected by the *near-seam* sandstone/conglomerate channels. In areas where thickening of the channel occurs, it is assessed that the abutment angle of $ ' 10° should be maintained, while F_t should be increased to 3.5.

Two other variables can influence the calculation of pillar stability factors: *in situ coal strength* (S_1) and the *overburden density* ((). A comprehensive study in the United States recently concluded that uniaxial compressive strength tests on small coal samples do not correlate with in situ pillar strength [Mark and Barton 1996]. That study and others in Australia and the Republic of South Africa [Salamon et al. 1996] found that using a constant seam strength works well for empirical pillar design methods. Accordingly, the in situ coal strength is taken to be 6.2 MPa, as used in ALPS.

In some Australian mines, there is so much coal in the overburden that the overburden density is significantly reduced below the (' 0.25 MN/m^3 that is typical for sedimentary rock. Dartbrook and Kenmare Collieries have undertaken satisfactory analyses of their overburden and have determined that (' 0.22 MN/m^3 and 0.23 MN/m^3, respectively.

Table 2.—Supplemental stress measurements from other Australian mines

Site details	Reference	Cell type	Cell position	Remarks	N, °	F_t (Meas)
Angus Place longwall 12	Clough [1989]	CSIRO HI	In roof	Author indicates vertical stress increase small; may be affected by clay bands within roof strata.	5.5	—
Central longwalls 301-302	Wardle and Klenowski [1988]	IRAD	In seam	Satisfactory results from which to interpret main gate and tailgate loading.	26.8	1.48
Cook longwalls 5-6	Gale and Matthews [1992]	CSIRO HI	In roof	Satisfactory results from which to interpret main gate and tailgate loading.	38.0	1.31
Ellalong longwall 1	Wold and Pala [1986]	IRAD	In seam	Satisfactory results from which to interpret main gate loading for barrier and adjacent development pillars.	17.2	—
Ellalong longwall 1	Wold and Pala [1986]	IRAD	In seam	Satisfactory results so as to interpret main gate loading for chain pillar.	9.8	—
Kenmare longwall 1B[1]	Gordon [1998]	CSIRO HI	In roof	Satisfactory results from which to interpret main gate loading condition.	54.2	—
North Goonyella longwalls 3-4	Nemcik and Fabjanczyk [1997]	CSIRO HI	In roof	Only 2 of 4 cells functioned reliably such that a subjective assessment of the stress profiles was required.	31.5	1.2
South Bulli longwalls 504-505	Mincad Systems Pty. Ltd. [1997]	IRAD and hydraulic.	In seam	Satisfactory results from which to interpret main gate and tailgate loading.	8.8	1.47
Ulan longwalls A and B	Mills [1993]	CSIRO HI	In roof	Satisfactory results from which to interpret main gate and tailgate loading.	35.3	1.09
West Cliff longwall 1	Skybey [1984]	IRAD	In seam	3-heading with large/small pillar combination; subjective assessment of main gate stress profile was required.	4.9	—
West Cliff longwalls 12-13	Gale and Matthews [1992]	CSIRO HI	In roof	3-heading with large/small pillar combination, interpretation of main gate and tailgate loading.	0.9	1.52
West Wallsend longwall 12	Stewart [1996]	Hydraulic	In seam	Satisfactory results from which to interpret main gate loading condition.	5.2	
Wyee longwall 5	Seedsman and Gordon [1991]	Geokon	In seam	Satisfactory results from which to interpret main gate loading condition.	6.2-8.8	

[1]SCT operations stress monitoring exercise with HI Cells located in roof above this project's hydraulic stress cell site.

INDUSTRY REVIEW

The aim of the industry review was to construct a historical database of gate road and chain pillar performance. During the course of the project, 19 longwall mines (a cross section from the 5 major Australian coalfields) were visited. Underground inspections were conducted at each that incorporated a subjective assessment of gate road performance while documenting the relevant details in relation to panel and pillar geometry, roof and floor geology, artificial support, and in situ stress regime. Brief summary reports were then forwarded to each mine to confirm the accuracy of the recorded data. Table 3 summarizes the Australian case histories.

The U.S. database included the Secondary Support Rating (SSUP), which is described as a rough measure of the volume of wood installed per unit length of the tailgate [Mark et al. 1994]. It should be noted that 59 of the 62 cases (i.e., 95%) within the U.S. database used standing secondary support (predominantly in the form of timber cribbing) along the tailgate. In the Australian database, less than 50% (9 out of 19) mines routinely installed standing secondary support along the tailgate. In the context of this study, standing secondary support refers to timber cribbing, the Tin Can system, Big Bags, etc., and does not include tendon support (cable bolts or Flexibolts) installed within the roof. Because of the variety of secondary supports used, no Australian SSUP was attempted. Instead, a yes/no outcome is provided in table 3.

An additional geotechnical parameter included within the Australian database, but not considered during the development of ALPS in the United States, is the presence of adverse horizontal stress conditions (HORST) (see table 3). Horizontal stress can damage roadways when they are first driven, and stress concentrations associated with longwall retreat can cause further roof deterioration. The following criteria were used to categorize the operations visited on a yes/no basis:

- 30° < " < 135° (see figure 7); and
- The magnitude of the major horizontal stress (F_H) is >10 MPa.

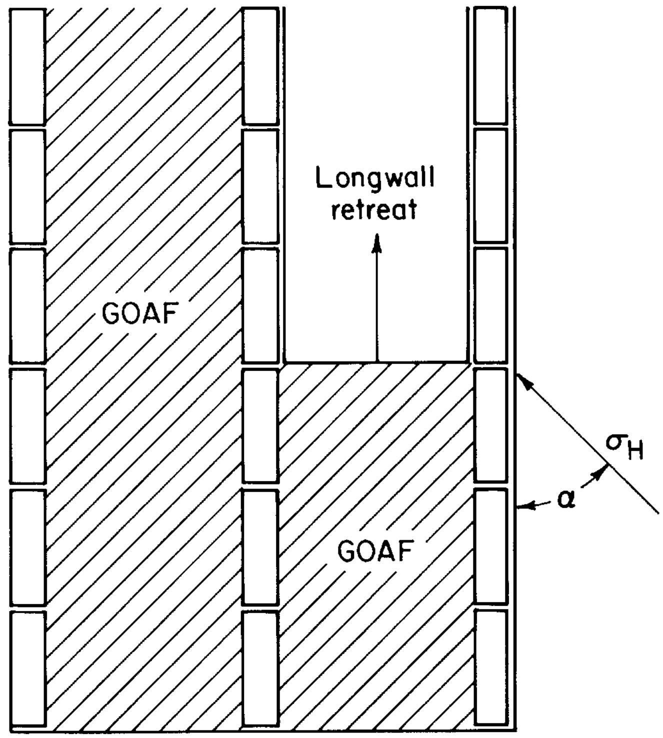


Figure 7.—The angle " used to determine the value of HORST.

Actual stress measurements were available from all except three mines in the database. The major horizontal stress is characteristically twice the vertical stress within Queensland and New South Wales coalfields. Therefore, at a depth of cover equal to 200 m, F_H is approximately 10 MPa.

It is recognized that geological structure can result in an adverse reorientation and/or magnification of the general in situ stress regime. However, there are insufficient data, within the context of this study, to include such an assessment within HORST.

STATISTICAL ANALYSES

The same statistical technique used with the U.S. ALPS database, that of discriminant analysis, was used with the Australian data. Discriminant analysis is a regression technique that classifies observations into two (or more) populations. In the case of the ALPS data, the classified populations are tailgates with satisfactory and unsatisfactory tailgate conditions.

An initial change that was made with the Australian data was to include "borderline" tailgates with the unsatisfactory cases. This modification is consistent with the Australian underground coal industry's desire to have in place strata management plans that design against both borderline and unsatisfactory gate road conditions. It also adds to the otherwise small pool of unsatisfactory cases available for analysis.

In their analysis, Mark et al. [1994] were not able to quantify the effect of standing secondary support on tailgate conditions. However, because nearly every U.S. case used some standing support, SSUP is basically *intrinsic* to the design equation (see equation 1). Because less than 50% of Australian mines use secondary support, it seems reasonable to assume that tailgates that presently incorporate standing secondary support would become unsatisfactory if it were removed. A major modification was to include all collieries utilizing standing secondary support in the modified-unsatisfactory category of tailgate conditions.

Table 3.—Australian tailgate performance case history database

Mine	Location	Pillar width, m	Pillar length, m	Seam height, m	Depth, m	Panel width, m	CMRR	TG SF	PSUP	SSUP, Yes/No	HORST, Yes/No	Tailgate condition
Angus Place	Tailgate 21	40	95.5	3.0	340	256	35	0.84	0.84	Yes	No	S
Angus Place	Tailgate 18	40	94.5	3.0	280	206	35	1.11	0.84	Yes	No	B
Angus Place	Tailgate 22	40	95.5	3.0	360	256	35	0.76	0.84	Yes	Yes	U
Central (200)	Tailgate 202	25	94.9	2.5	165	200	55	1.33	0.27	No	No	S
Central (200)	Tailgate 203	25	94.9	2.5	190	206	55	1.05	0.27	No	No	S
Central (200)	Tailgate 204	30	94.9	2.5	210	206	55	1.26	0.27	No	No	S
Central (200)	Tailgate 205	35	94.9	2.5	225	206	55	1.50	0.27	No	No	S
Central (200)	Tailgate 206	45	94.9	2.5	240	206	55	2.14	0.27	No	Yes	S
Central (200)	Tailgate 207	45	94.9	2.5	265	206	55	1.87	0.27	No	Yes	S
Central (200)	Significant jointing		94.9	2.5			48	1.05	0.50	No	No	S
Central (300)	Tailgate 302	30	94.9	2.8	140	200	50	2.00	0.27	No	No	S
Central (300)	Tailgate 303	30	94.9	2.8	170	206	50	1.63	0.27	No	No	S
Central (300)	Tailgate 304	35	94.9	2.8	190	206	50	1.80	0.27	No	No	S
Central (300)	Tailgate 305	40	94.9	2.8	210	206	50	1.95	0.27	No	No	S
Central (300)	Tailgate 306	45	94.9	2.8	230	206	50	2.07	0.27	No	No	S
Central (300)	Tailgate 307 - 18 cut-through	45	94.9	2.8	285	206	31	1.45	0.50	No	No	U
Clarence	Tailgate 2	45	54.5	4.1	260	178	59	1.20	0.23	No	No	S
Clarence	Tailgate 3	43	54.5	4.1	260	200	59	1.10	0.23	No	No	S
Clarence	Tailgate 5	45	54.5	4.1	260	158	59	1.21	0.23	No	No	S
Clarence	Tailgate 6	45	39.5	4.1	260	200	59	1.22	0.23	No	No	S
Crinum	Tailgate 2	35	125.2	3.6	135	275	40	2.57	0.69	Yes	No	S
Dartbrook	Tailgate 2	35	94.8	3.9	250	200	51	0.86	0.42	No	No	S
Elouera	Tailgate 2 - 4 lower stress	45	12.5	3.3	350	155	40	1.02	0.85	Yes	No	S
Elouera	Tailgate 4 - 19.5 cut-through	45	125.0	3.3	350	155	40	1.00	0.85	Yes	Yes	B
Gordonstone	Tailgate 102	40	94.8	3.2	230	200	30	1.49	0.79	Yes	No	B
Gordonstone	Tailgate 202	40	94.8	3.2	230	255	35	1.49	0.79	Yes	No	S
Kenmare	Tailgate 2 - 13 cut-through	30	119.8	3.1	172	200	65	1.46	0.53	No	No	S
Kenmare	Tailgate 3 - stronger roof	25	119.8	3.1	160	200	65	1.17	0.28	No	No	S
Kenmare	Tailgate 3 - weaker roof	25	119.8	3.1	130	200	46	1.65	0.42	No	No	S
Newstan	Tailgate 10	31	97.0	3.3	180	130	39	1.39	0.66	Yes	Yes	B
North Goonyella	Tailgate 4	30	94.8	3.4	180	255	38	1.26	0.77	No	No	S
Oaky Creek	Tailgate 7 - normal roof	30	94.8	3.2	180	200	57	1.32	0.40	No	No	S
Oaky Creek	Tailgate 7 - weaker roof	30	94.8	3.2		200	48	1.32	0.57	No	No	S
South Bulli (200)	Tailgate 203	24	84.0	2.7	465	138	57	0.23	0.44	Yes	Yes	U
South Bulli (200)	Tailgate 204	31	94.0	2.7	470	183	57	0.36	0.44	Yes	Yes	U
South Bulli (200)	Tailgates 205-208, 210	40	96.0	2.7	460	183	57	0.66	0.44	Yes	Yes	B
South Bulli (200)	Tailgates 209, 211-212	38	97.0	2.7	460	183	57	0.59	0.44	Yes	Yes	B
South Bulli (300)	Tailgate 303	40	96.0	2.7	450	138	65	0.68	0.44	Yes	No	S
South Bulli (300)	Tailgates 304-305	55	74.0	2.7	450	183	65	1.15	0.44	Yes	No	S

See explanatory notes at end of table.

Table 3.—Australian tailgate performance case history database—Continued

Mine	Location	Pillar width, m	Pillar length, m	Seam height, m	Depth, m	Panel width, m	CMRR	TG SF	PSUP	SSUP, Yes/No	HORST, Yes/No	Tailgate condi-tion
South Bulli (300)	Tailgates 308-309	38	97.0	2.7	410	185	65	0.68	0.44	Yes	No	S
Southern (600)	Tailgate 606	30	94.8	2.8	170	200	60	1.62	0.26	No	No	S
Southern (600)	Tailgates 607-608	35	94.8	2.8	190	200	60	1.79	0.26	No	No	S
Southern (700)	Tailgate 702	30	94.8	2.8	160	250	60	1.79	0.26	No	No	S
Springvale	Tailgate 402	45	95.2	2.7	325	250	35	1.22	0.63	Yes	Yes	B
Tower	Tailgate 14	45	66.0	3.2	500	203	40	0.59	1.26	No	No	S
Ulan	Tailgate 11	30	94.8	3.1	145	255	50	1.65	0.28	No	No	S
West Cliff	Tailgate 22	42	97.2	2.5	480	200	48	0.69	0.49	Yes	No	S
West Wallsend	Tailgate 13 - 4.5 cut-through	35	97.1	2.9	240	145	40	1.24	0.75	Yes	Yes	U
West Wallsend	Tailgate 13 - 7 cut-through	35	97.1	2.9	255	233	40	1.11	0.75	No	Yes	S
West Wallsend	Tailgates 14-16	32	110.1	2.9	250	145	40	0.99	0.75	Yes	Yes	U
West Wallsend	Tailgate 17 - 6 cut-through	35	110.1	3.2	250	145	40	1.08	0.75	Yes	Yes	B
Wyee	Tailgate 13	35	102.0	2.8	220	163	45	1.43	0.52	No	Yes	B
Mean		31.2	94.5	3.0	266	200	49.52	1.27	0.49			
Standard deviation		7.2	16.9	0.4	106	33	10.04	0.47	0.24			

S Satisfactory. B Borderline. U Unsatisfactory.

Two cases posed additional complications. Tower Colliery does not incorporate standing secondary support, yet its PSUP (1.26) is 3.2 standard deviations above the Australian mean. Therefore, Tower Colliery was also included within the modified-unsatisfactory tailgate category. Crinum uses standing secondary support, but it is a relatively new operation, and it seems that there has been an understandable, but nonetheless highly conservative approach to its geotechnical design. To include Crinum within the modified-unsatisfactory group would have been overly conservative, so it was excluded from the database entirely.

Therefore, the final database includes 50 case histories with 29 modified satisfactory and 21 modified-unsatisfactory cases. Numerous analyses were conducted to determine the best design equation. Ultimately, the most successful design equation relates the required TG SF to the CMRR, as shown in figure 8:

$$\text{TG SF} = 2.67 - 0.029\ \text{CMRR} \tag{3}$$

Equation 3 correctly predicted the outcome of all except seven case histories, for a success rate of 86%. Comparing equation 3 to the U.S. design equation (equation 1), it may be seen that the TG SF is generally more conservative than the ALPS SF for weaker roof, but the TG SF decreases more rapidly than the ALPS SF as the roof becomes stronger.

Another strong relationship that was evident in the case histories was between the primary support and the roof quality. Figure 9 plots the PSUP against the CMRR, and the best-fit regression is of the following form:

$$\text{PSUP} = 1.35 - 0.0175\ \text{CMRR} \tag{4a}$$

It seems that Australian mine operators have intrinsically adapted their primary support patterns to the roof conditions and operational requirements. Mark et al. [1994] reached a similar conclusion for the United States.

Upper- and lower-boundary equations (4b and 4c, respectively) relating CMRR to PSUP have also been proposed and are illustrated in figure 8:

$$\text{PSUP}_U = 1.45 - 0.0175\ \text{CMRR} \tag{4b}$$

$$\text{PSUP}_L = 1.24 - 0.0175\ \text{CMRR} \tag{4c}$$

Equation 4c may be applicable, for example, when the mining layout is not subject to adverse horizontal stress conditions and/or standing secondary support is planned as part of the colliery's strata management plan.

Mark et al. [1994] also found a strong correlation between the CMRR and the entry width. No such correlation was seen here.

It is interesting to note some similarities and differences between the U.S. and Australian databases. For example, overall roof quality seems to be reasonably similar in the two countries. The mean CMRR in the United States is 53.7 with a standard deviation (SD) of 13.9; this compares with an Australian mean of 49.5 and SD = 10.0. However, the mean Australian PSUP is 0.49 (SD = 0.23), which is approximately twice that of the U.S. database.

Studies by Mark [1998] and Mark et al. [1998a] suggest that the horizontal stress levels in the two countries are comparable. It seems that philosophical differences are more likely responsible for the different levels of primary support. Most

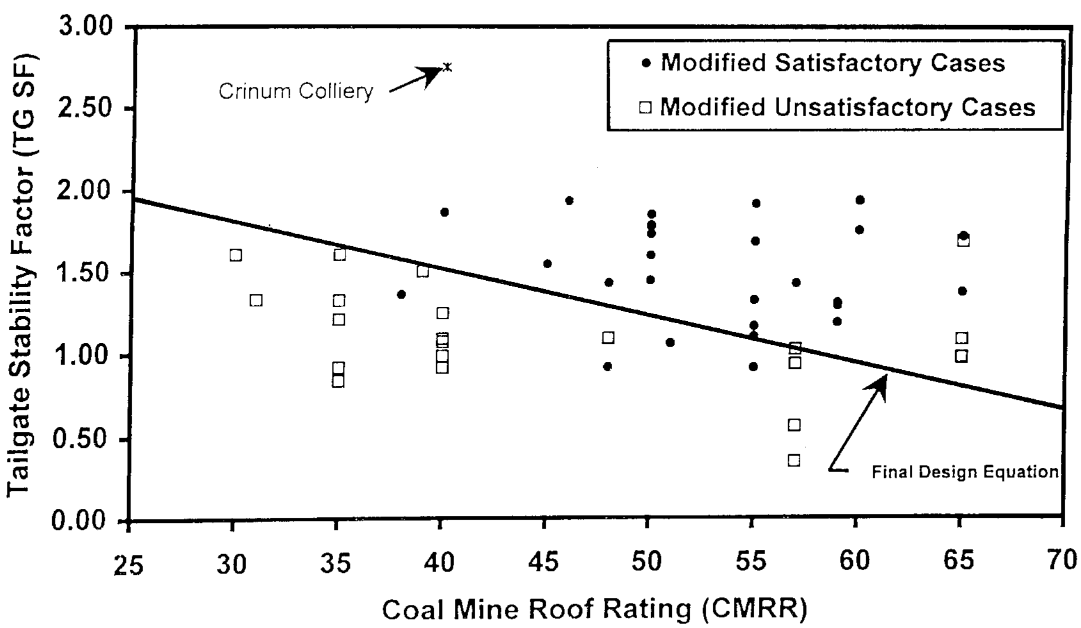


Figure 8.—The final design equation relating the CMRR to the TG SF.

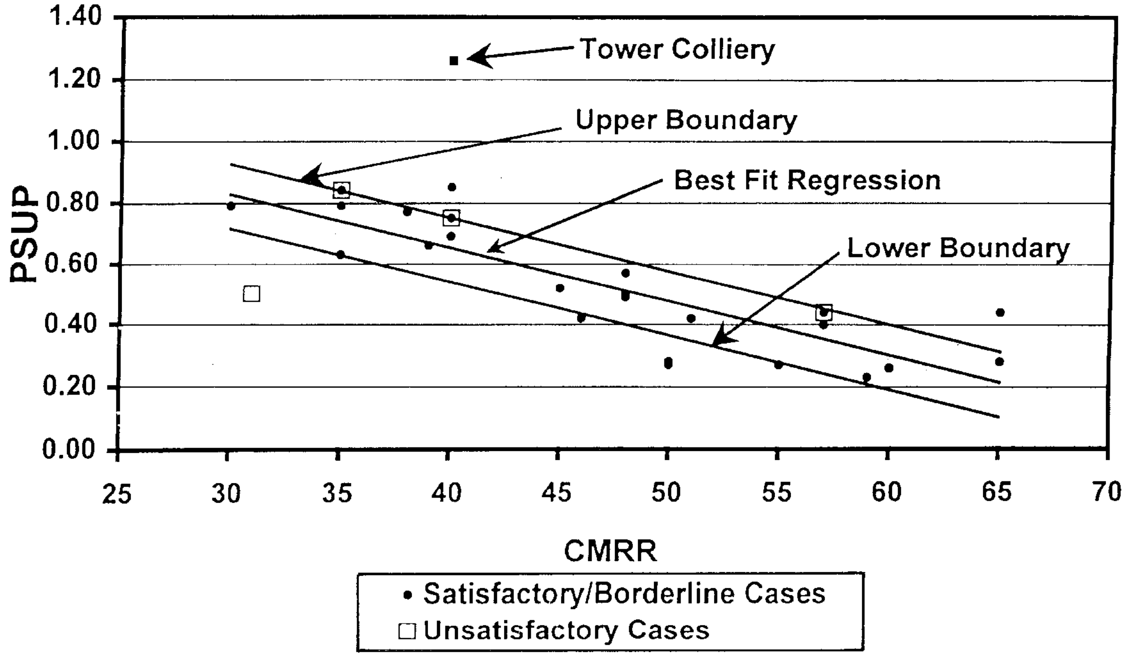


Figure 9.—Design equations for primary support based on the CMRR.

Australian coal mines have an unwritten (sometimes written) policy of no roof falls; U.S. multientry mining systems seem more tolerant of roof falls. Also, most Australian coal mines have an antipathy toward standing secondary support for reasons associated with a two-entry gate road system. It seems that the main way in which Australian operations prevent poor tailgate conditions is to install substantial primary support on development. Therefore, in Australia one would expect a strong relationship between the level of primary support and a reliable roof rating system. This is exactly what transpires, which adds to the credibility of the CMRR.

Additional statistical analyses tested whether the accuracy of ALPS could be improved by replacing the original Bieniawski formula with another pillar strength formula. Two formulas were trialed—the Mark-Bieniawski formula [Mark and Chase 1997] and Salamon's squat pillar formula [Madden 1988]. The Mark-Bieniawski formula had virtually no impact on the classification success rate. However, incorporating the squat pillar formula resulted in a significant decrease in the classification success rate. The conclusion was to remain with the original Bieniawski formula used in the "classic" ALPS.

ANALYSIS OF TAILGATE SERVICEABILITY (ALTS)

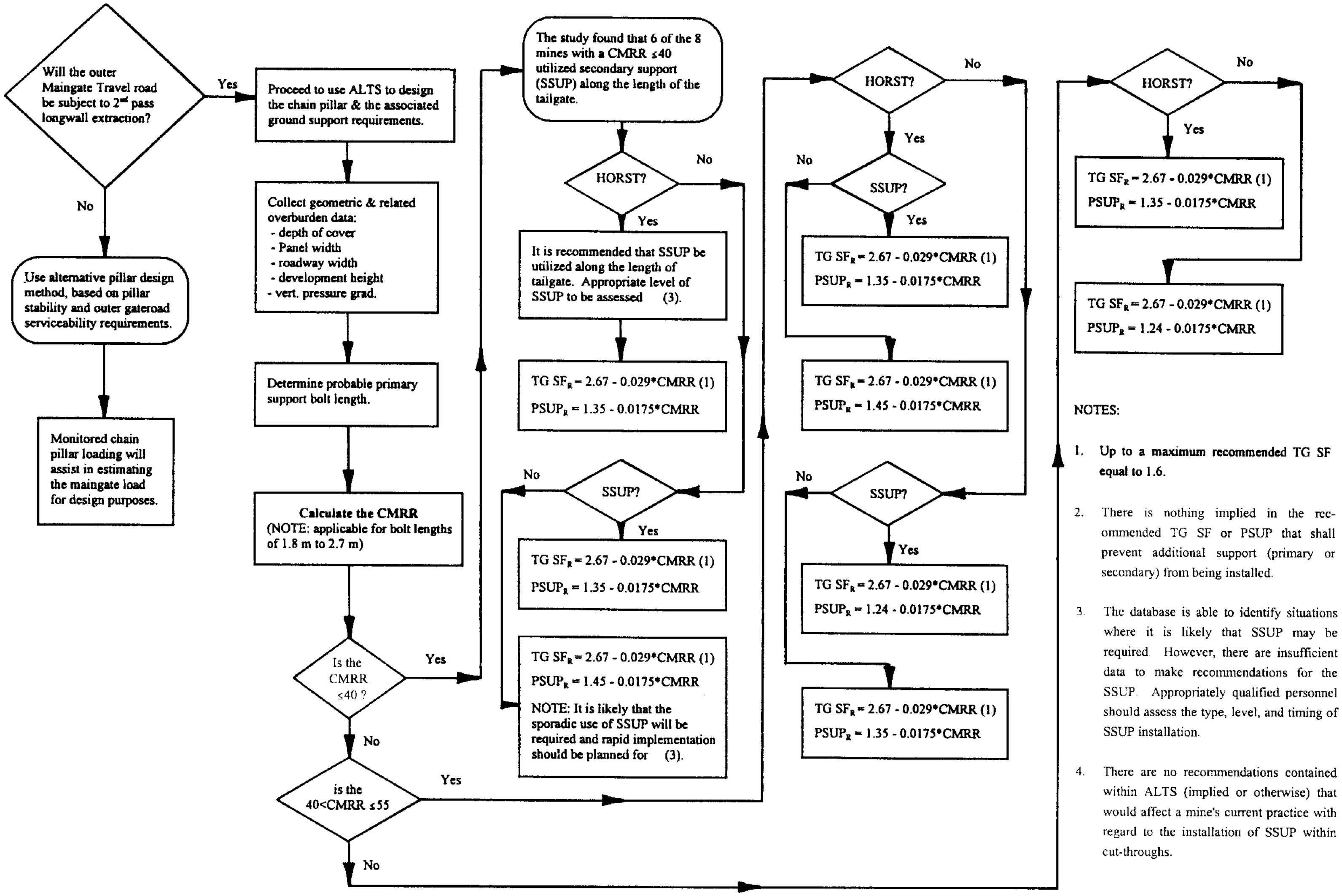
The chain pillar design methodology proposed by the project is referred to as "Analysis of Longwall Tailgate Serviceability" (ALTS). The design methodology recognizes the impact of ground support on tailgate serviceability and incorporates guidelines in relation to the installed level of primary support and the influence of standing secondary support on the design process.

A design flowchart (figure 10), Microsoft® Excel Workbook, and user manual have been developed. The spreadsheet workbook (*ALTS Protected.xls*) was formulated to facilitate the computational components of the design methodology.

The ALTS design process should only be employed in designing chain pillars that are subject to second-pass longwall extraction. If the chain pillars under consideration are not to be subject to second-pass longwall extraction, then an alternative pillar design method should be employed based on pillar stability and outer gate road serviceability requirements. The monitored chain pillar loading behavior (conducted as a part of the project) will assist in estimating the main gate load for design purposes.

The recommended chain pillar width (rib to rib) is contingent upon an appropriate level of primary support. That level of primary support (i.e., $PSUP_L$ to $PSUP_U$) is dependent on (1) the orientation of longwall retreat in relation to the magnitude and direction of the major horizontal stress and (2) the use of standing secondary support along the length of the gate road.

The database is able to identify situations where it is likely that standing secondary support may be required. However, there are insufficient data at this stage to make numerical recommendations for the SSUP similar to those made for the TG SF and PSUP. Appropriately qualified personnel should assess the type, level, and timing of SSUP installation.



NOTES:

1. **Up to a maximum recommended TG SF equal to 1.6.**
2. There is nothing implied in the recommended TG SF or PSUP that shall prevent additional support (primary or secondary) from being installed.
3. The database is able to identify situations where it is likely that SSUP may be required. However, there are insufficient data to make recommendations for the SSUP. Appropriately qualified personnel should assess the type, level, and timing of SSUP installation.
4. There are no recommendations contained within ALTS (implied or otherwise) that would affect a mine's current practice with regard to the installation of SSUP within cut-throughs.

Figure 10.—Flowchart for using Analysis of Longwall Tailgate Serviceability (ALTS).

CONCLUSIONS

The following main goals of the project were achieved:

• To establish a chain pillar design methodology that has widespread application to Australian longwall operations; and

• To quantify the probable variance in the chain pillar loading environment between collieries and mining localities and to incorporate this variance within the design methodology.

In addition, the study has been able to propose definitive guidelines with regard to the installed level of primary support and to conduct a subjective analysis regarding the impact of standing secondary support on the design process. This provides the Australian coal industry with a truly integrated design methodology with regard to tailgate serviceability that has been able to address the main factors controlled by the mine operator.

The initial benefit from this project is that mine managers and strata control engineers will be able to identify where chain pillars can be reduced in size and where increases may be necessary. They can make these decisions with the confidence that a credible Australian database is the foundation for the design methodology.

This project has identified that there is an opportunity for some mines that do not currently incorporate the routine installation of secondary support along their tailgate to make significant reductions in chain pillar width. It is an operational decision whether a reduction in pillar width is more or less beneficial to production output and costs than the introduction of secondary support along the length of the tailgate. This project simply highlighted that the opportunity exists.

The chain pillar monitoring exercises conducted at collieries under deep cover or with strong roof have found that the abutment load may be overestimated by using a generic abutment angle of $ ' 21°. However, the aggressive tailgate loading behavior monitored at West Wallsend Colliery (see figure 5) provided a warning, which emphasized the need to use great caution before making any sweeping changes to a proven chain pillar design tool. Although the way in which the load manifested itself at West Wallsend was significantly different from that proposed by ALPS, the resultant tailgate load was quite similar.

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