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BUREAU OF MINES
INFORMATION CIRCULAR/1990

Pillar Design Methods for Longwall Mining

By Christopher Mark

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UNITED STATES DEPARTMENT OF THE INTERIOR
Manuel Lujan, Jr., Secretary

BUREAU OF MINES
T S Ary, Director

Library of Congress Cataloging in Publication Data:

Mark, Christopher
Pillar design methods for longwall mining / by Christopher Mark.

p. cm. -- (Bureau of Mines information circular; 9247)

Includes bibliographical references p. 45-47.

Supt. of Docs. no.: I 28.27:9247.

1. Longwall mining. 2. Pillaring (Mining) I. Title. II. Series: Information circular (United States. Bureau of Mines); 9247

TN295.U4 [TN275] 622 s--dc20 [622'.334] 89-600380 CIP

CONTENTS

ILLUSTRATIONS

ILLUSTRATIONS–Continued

TABLES

UNIT OF MEASURE ABBREVIATIONS USED IN THIS REPORT

deg	degree	pct	percent
ft	foot	pcf	pound per cubic foot
ft^2	square foot	psf	pound per square foot
in	inch	psi	pound per square inch
lb	pound	ton/ft	ton per foot
lb/ft	pound per foot		

PILLAR DESIGN METHODS FOR LONGWALL MINING

By Christopher Mark[1]

ABSTRACT

Effective ground control in the gate entries is essential for safe and productive longwall mining. Longwall pillars protect the gate entries from the severe abutment loads that develop as the longwall retreats. This U.S. Bureau of Mines report summarizes 5 years of research aimed at improving longwall pillar design. Its goal is to provide longwall operators with practical procedures for maintaining longwall ground control.

The report focuses on the Analysis of Longwall Pillar Stability (ALPS) design method, which was developed largely by Bureau researchers. With ALPS, mining engineers can estimate the strength of longwall pillar systems and the load that will be applied to them. Several other methods that can be directly used to size longwall pillars are also described. The design methods are evaluated using a data bank of more than 100 mining case histories, and suggestions are given for using the methods in practical design. A step-by-step solution to a sample problem using ALPS is provided. The report also discusses the theory and practice of yield pillar design and suggests strategies for special conditions, including soft floor, excessive horizontal stresses, and multiple-seam interactions.

[1]Mining engineer, Pittsburgh Research Center, U.S. Bureau of Mines, Pittsburgh, PA.

INTRODUCTION

In recent years longwall mining has emerged as the most important technology to be applied to underground mining since the continuous mining machine. Currently longwall mines produce more than 30 pct of all underground coal, up from less than 5 pct just 15 years ago (*94*).[2]

The growth of longwall mining has been spurred by the increasing efficiency of the technology. Productivity on longwall faces has doubled since 1983 (*25*) and is expected to double again over the coming decade (*30*). Longwall mining is also responsible for significant safety advances. Research shows that on a per-ton basis longwall accident rates are one-third lower than room-and-pillar rates (*87*). Because of their better productivity and safety record, longwalls have been called "the salvation of the large deep mine in America" (*60*).

The longwall gate entries are the lifelines through which miners, supplies, and ventilating air reach the face, and through which the coal is transported toward the outside (fig. 1). Safe longwall mining depends on maintaining ground control in the gate entries. Miners working in the gate entries are not protected by powered supports as they are at the face and so may be exposed to much greater roof fall hazards. In addition, the gate entries contain the escape routes miners need in case of an emergency. Recently the Mine Safety and Health Administration (MSHA) introduced new regulations that require that roof control plans address the issue of maintaining safe travelways on the tailgate side of longwalls (*103*).

Instability in the gate entries can also have major economic impacts. One operator estimated that downtime on a longwall costs $200 per *minute*, about 8 times as much as downtime on a continuous miner section (*58*). A major roof fall in the headgate or tailgate entry can stop a longwall for days. As longwall productivity continues to skyrocket, the cost of gate entry falls will increase accordingly.

In multientry retreat longwall mining as practiced in the United States, rows of chain pillars are left to protect the gate entries. The design of these pillars is often the single most important element in gate entry ground control. Successful longwall pillar design must address the different stability requirements and different load conditions that develop during the progression of longwall mining. A typical chain pillar system will be subjected to two sets of longwall abutment loadings during its service life, first as a headgate and then as a tailgate. Other chain pillars will be subjected to only a single abutment loading as they protect bleeder entries. Barrier pillars must also be sized to protect the main entries from longwall loadings.

Two basic philosophies are available to guide longwall pillar design (*26*). The *conventional* design approach uses large pillars that are sized to carry all the abutment load to which they will be subjected. The *yielding* design approach, on the other hand, uses only very small pillars, which transfer load. Nearly every one of the 80 longwalls in the Eastern United States uses some form of conventional pillar design, while many of the 12 western longwalls have employed yielding designs.

The key assumption of conventional longwall pillar design is that unstable pillars will result in unstable gate entries. As this report will illustrate, there have been many examples of mines where undersized pillars were associated with intolerable entry conditions. In many cases, once the chain pillar sizes were increased, ground conditions in the gate entries improved dramatically. Larger pillars can improve ground conditions because they are better able to distribute abutment loads, resulting in lower average pillar stresses, lower bearing pressures, and smaller displacements.

Making pillars too large can be expensive and wasteful, however. A major problem in longwall mining is to keep development ahead of the longwall face. Larger pillars increase development time because they require longer crosscuts and are more difficult to ventilate and mine. Also, longwall pillars are seldom recovered, and so the coal that is locked up in them is a wasted resource. The potential economies from optimizing pillar size can be substantial. One operator estimated that improved pillar design saved more than $500,000 in direct development costs and made an additional 150,000 tons of coal available for longwall extraction, *per longwall panel* (*18*).

As part of its program to increase worker safety and improve efficiency in the U.S. coal mines, the Bureau of Mines has devoted a major research effort to the problem of designing effective pillar systems for longwalls. This report summarizes the results to date. The report focuses on a design method called Analysis of Longwall Pillar Stability (ALPS), which was largely developed by Bureau researchers. The ALPS method has now been verified by back-analysis of more than 100 mining case histories. Step-by-step guidelines for using ALPS to design longwall pillars are presented in appendix A.

Several other formulas using the conventional pillar design philosophy have been proposed for longwalls by other researchers. Three methods, those of Carr and Wilson (*19*), Choi and McCain (*22*), and Hsuing and Peng (*52*), are described in this report. Suggestions for using these methods are presented based on analyses of case histories.

The strata mechanics of yield pillars are more complex than those of conventional pillars, and as a result, there are few specific formulas for designing them. Considerable new experience has been gained over the past several years, however. A section of this report presents the issues associated with yield pillar design, the lessons of recent yield pillar trials, and some guidelines for sizing yield pillars.

[2]Italic numbers in parentheses refer to items in the list of references preceding the appendixes at the end of this report.

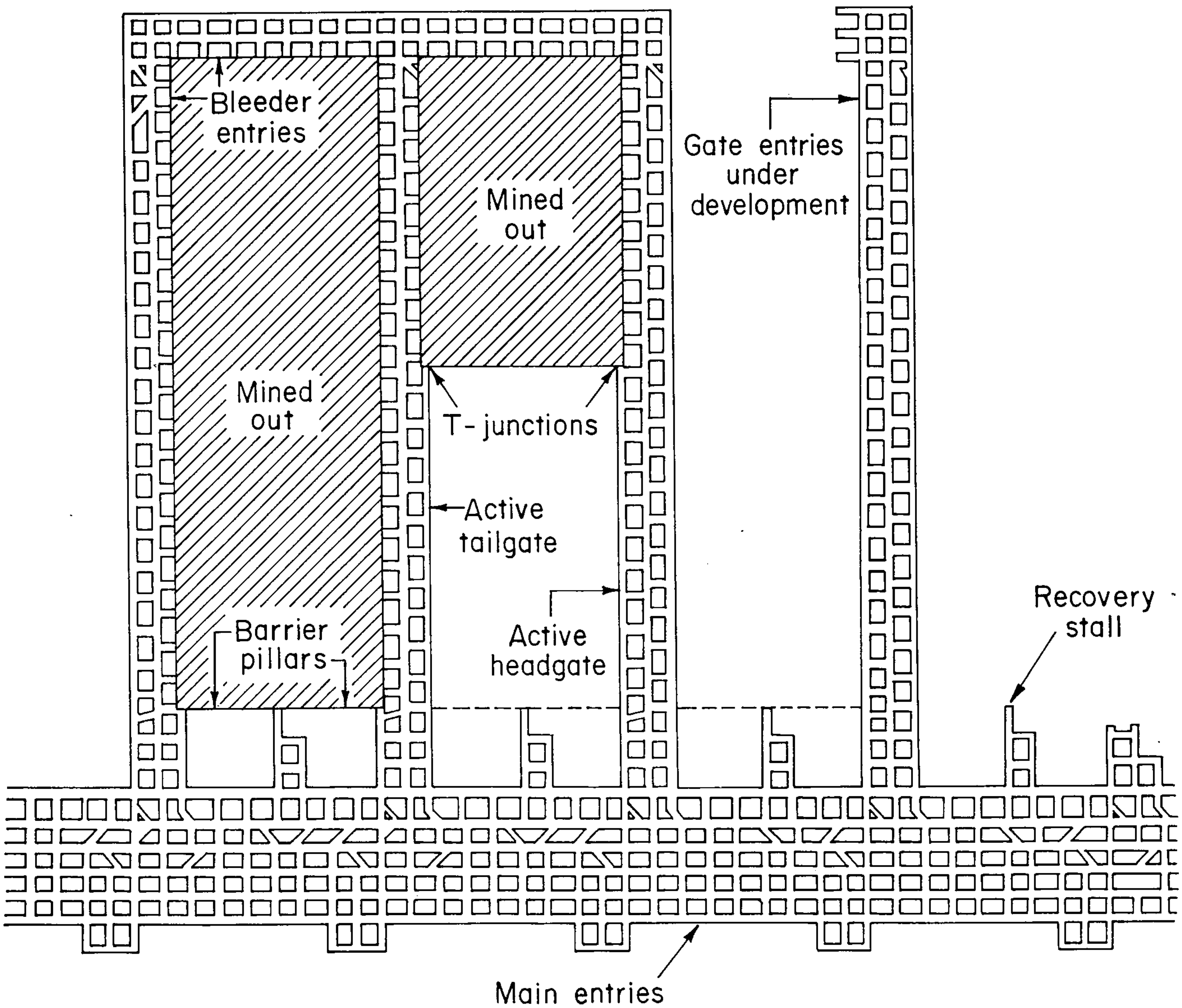

Figure 1.–Typical longwall panel layout.

Finally, the report briefly addresses some special conditions that can affect gate entry performance but are not directly included in the pillar design formulas. Some of the conditions discussed are roof rock quality, in situ horizontal stresses, the potential for pillar bumps, and longwall ventilation requirements. Strategies for incorporating these issues into effective longwall pillar design are presented.

THE ANALYSIS OF LONGWALL PILLAR STABILITY (ALPS) METHOD

The Analysis of Longwall Pillar Stability, or ALPS, method was originally developed by Mark and Bieniawski (77) at the Pennsylvania State University, and it has since been refined at the Bureau (75). ALPS was initially based on field data collected in studies conducted in nine longwall panels. Measurements from seven additional panels have now been incorporated into the data base, which currently covers five mines and four States. More important, the ALPS approach has now been verified by back-analysis of more than 100 case histories.

As a conventional pillar design method, ALPS consists of three basic elements:

- Estimation of the load applied to the pillar system.
- Estimation of the strength of the pillar system.
- Determination of a stability factor (SF) and comparison with a design criterion.

The next sections of this report discuss how each of the elements of ALPS was developed from field measurements and observations.

LONGWALL PILLAR LOADS

The loads applied to longwall pillars may be divided into two parts: *development loads*, which are present before longwall mining, and *abutment loads*, which occur during longwall panel extraction. Development loads are very similar to the loads applied to pillars in room-and-pillar mining. The accurate characterization of abutment loads is the key to longwall pillar design.

Development loads are due to the weight of the overburden directly above the pillars and the gate entries. The tributary area expression for the development load per foot of gate entry (L_d) is

$$L_d = (H)\,(w_t)\,(\gamma), \tag{1}$$

where H = depth of cover, ft,

w_t = width of pillar system, ft,

$= \Sigma\,[(w)] + (n - 1)\, w_e$,

w = width of individual pillars, ft,

w_e = entry width, ft,

n = number of entries in gate entry system,

and γ = unit weight of overburden, pcf.

To check the validity of the tributary area theory for longwall pillar design, a series of two-dimensional, linear-elastic, finite-element models were run using the finite-element computer program ANSYS (*74*). The parameters that were varied in the models included rock mass properties, horizontal stress, extraction ratio, and section geometry.

The model results indicated that tributary area theory provides a satisfactory estimate of the development loads on typical longwall pillars. The accuracy of the theory decreased as the extraction ratio increased, but the theory's predictions were within 10 pct of the model results for two-dimensional extraction ratios of up to 50 pct. The development loads observed in the model were also affected by less than 10 pct when the stiffness of the roof and floor was increased to 10 times that of the coal, when horizontal stresses twice as great as the vertical were applied, and when unequal-sized pillars were used. Based on these model results, equation 1 is used in ALPS to estimate longwall development loads. Recent field measurements also indicate that the tributary area theory gives a close approximation of the actual average development load for typical conventional longwall pillars (*102*).

Abutment loads occur as a portion of the weight of the overburden that had been supported by the excavated longwall panel is transferred to the pillars. Abutment load increases usually begin several hundred feet before the arrival of the face. From a mining standpoint, the most critical abutment loads are those experienced by the pillars at the face ends or T-junctions (fig. 1). When the first panel is mined, the pillars at the headgate T-junction must carry the first front abutment (L_{fh}). As the face continues to advance, the pillar load increases until it stabilizes at a final value called the side abutment (L_s or L_{ss}). If a second panel is mined, then the abutment load on the pillars at the tailgate T-junction includes the side abutment plus the second front abutment (L_{ft}). Pillars isolated in the gob are subjected to two side abutments.

The side abutment is much easier to analyze than the front abutment, because it may be treated in two dimensions. Two similar empirical approaches have been proposed for estimating the side abutment. Wilson (*107*) proposed that the vertical stress in the gob increases linearly, from zero at the rib to the original overburden pressure at some point within the gob. He estimated that the distance required for the gob pressure to return to cover load is typically 0.3 times the depth of cover. The side abutment may therefore be visualized as shown in the lower part of figure 2.

The second analytical approach for estimating the side abutment, proposed by King and Whittaker (*61*), begins with the concept of a shear angle that determines the pillar loading. As shown in the upper part of figure 2, the side abutment is represented as the wedge of strata defined by the shear angle ß. King and Whittaker presented two equations for quantifying the side abutment per foot of gate entry, one for critical and supercritical panels (equation 2), where the panel width P exceeds twice (H tan ß), and the other for subcritical panels (equation 3).

$$L_s = H^2\,(\tan \beta)\,(\gamma/2), \tag{2}$$

$$L_{ss} = \left[\frac{HP}{2} - \frac{P^2}{8\tan \beta}\right]\gamma. \tag{3}$$

King and Whittaker proposed that the shear angle might be equal to the angle of draw used in subsidence analysis, which they estimated at 25° for British conditions. Choi and McCain (*22*) slightly modified King and Whittaker's method, and presented subsidence data indicating that for the Pittsburgh Seam ß = 18°.

Although developed using different lines of reasoning, the methods of Wilson, King and Whittaker, and Choi and McCain are very similar in application. All assume that the weight of the overburden between the ribline and

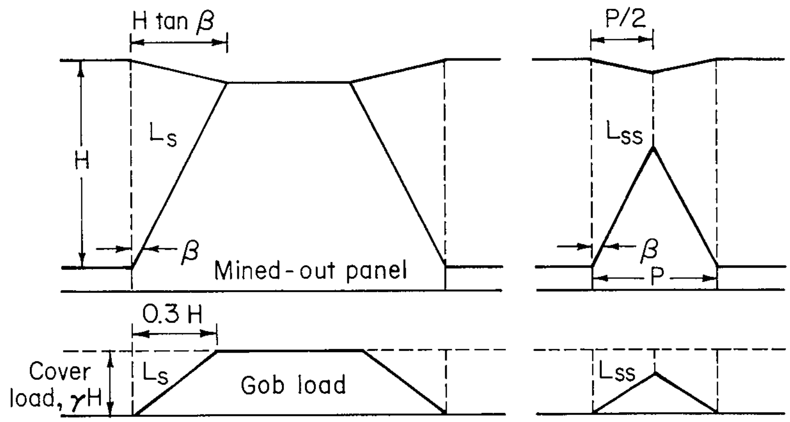

Figure 2.–Conceptualizations of the side abutment load proposed by King and Whittaker (*61*) (upper sketch) and Wilson (*107*) (lower sketch).

a point (H tan ß) into the gob is evenly split between the pillars and the gob. For Wilson's 0.3 H, ß works out to 16.7°, which is very near the 18° used by Choi and McCain.

While there undoubtedly is a relationship between subsidence and abutment load (*43*), that relationship may not be so straightforward as is implied by King and Whittaker. Nevertheless, the concept of an abutment angle equivalent to ß is very useful for design. The abutment angle should not be considered a physical reality, but an approximation that defines the side abutment in equations 2 and 3. To make these equations useful for ALPS, it was necessary to determine the value of ß appropriate for eastern U.S. conditions. Field measurements of longwall pillar loads, discussed in the next section, provided the means of determining ß.

The magnitudes of the front abutments (L_{fh} and L_{ft}) are much more difficult to determine analytically than that of the side abutment. The load transfer at the face ends is complicated in three dimensions, because the overburden load is shared among the pillars, the solid coal, and the gob. Some researchers have used three-dimensional numerical modeling to analyze this problem (*52*, *64-65*, *88*). The empirical approach used in ALPS begins with the concept that the front abutments are fractions of the side abutment and can be represented as

$$L_{fh} = F_h (L_s), \tag{4}$$

$$L_{ft} = F_t (L_s), \tag{5}$$

where F_h and F_t are front abutment factors with values of less than 1. The actual magnitudes of F_h and F_t are probably affected by local geology, but typical values should be sufficient for design purposes. Stress measurements were again used to determine appropriate values of F_h and F_t for ALPS.

A final aspect of the abutment load prediction problem is the distribution of the load. When the first panel adjacent to a gate entry system is mined, the pillars may not carry the entire abutment load because some portion may be transferred to the nearby barrier pillar or unmined panel. Peng and Chiang (*92*) analyzed field measurements to determine the width of the abutment influence zone (D), which they defined as the distance from the panel edge that abutment stress increases could be detected. They obtained equation 6:

$$D = 9.3 (H)^{0.5}. \tag{6}$$

The distribution of stress within the abutment influence zone must be known in order to estimate the percent of the total front or side abutment load that is carried by the chain pillars. Airey (*4*) proposed that within the influence zone the stress decays according to the inverse square of the distance from the panel edge. The field measurements of longwall pillar loads were used to test the hypothesis of an inverse-square stress decay.

FIELD MEASUREMENTS OF LONGWALL ABUTMENT LOADS

The design factors F_h, F_t, and ß can be easily calculated if the total front and side abutment loads are known. Unfortunately, pillar loadings cannot be measured directly, but must instead be inferred from measurements of *stress* made at distinct points within the pillar. Since the distribution of stresses within a pillar is generally nonuniform, an *array* of stressmeters is required to determine the average pillar stress. The pillar load is then the average pillar stress multiplied by the pillar's load-bearing area. A typical stressmeter array is shown in figure 3.

As the goal of the field studies was to measure *abutment* loads, rather than total loads, it was only necessary to measure the stress *changes* that occurred during longwall mining. Accordingly, vibrating wire stressmeters (*40*, *74*) were used at all the field sites. Typical results obtained from a stressmeter array, in the form of pillar stress distribution profiles, are shown in figure 4. As figure 4 illustrates, the initial abutment loads were usually observed in an instrumented pillar as stress increases that occurred near the pillar's ribs. Later, as the abutment load

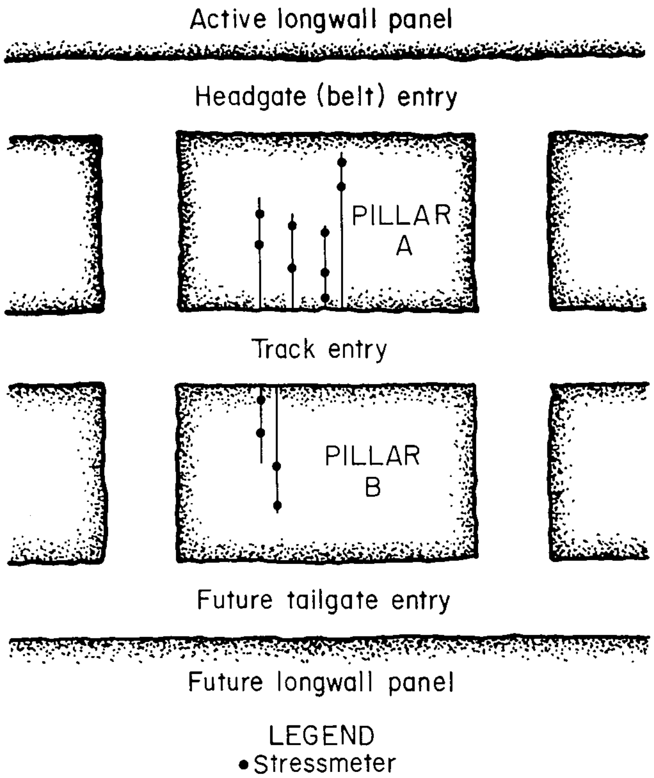

Figure 3.–Typical stressmeter array installed to measure abutment stress profiles.

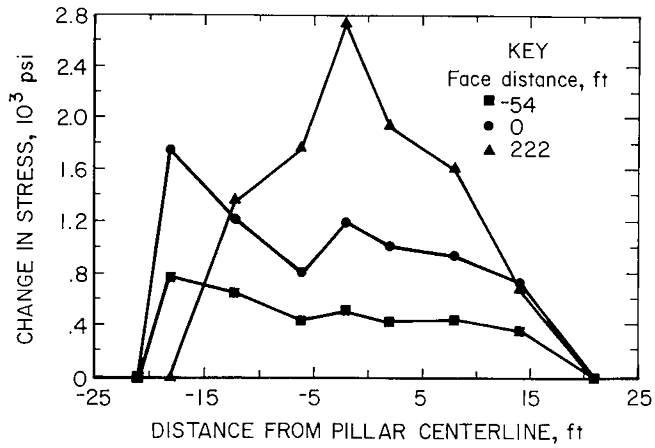

Figure 4.–Abutment stress profiles obtained from a 42-ft wide longwall pillar during panel extraction.

increased, the stress peaks shifted to the pillar's core while the stressmeters near the rib often lost load. These results confirmed that while no single stressmeter could have accurately reflected the loading history of the pillar, reasonable estimates of the average pillar stress change could be obtained by using arrays of stressmeters.

Field measurements were conducted at five separate mines. The Bureau conducted three of the studies, one in Ohio at mine A, and two in Pennsylvania at mines B and C. The fourth study was carried out by the Pennsylvania State University at mine D in northern West Virginia. U.S. Steel Research performed the fifth study in mine E, located in eastern Kentucky.

At mine A, arrays of 15 stressmeters were installed in four pairs of pillars as shown in figure 5. At all four locations the pillar design consisted of a 92-ft pillar next to a 32-ft block, under overburden ranging from 450 to 760 ft. Further details of the study were given by Allwes (*5*). At mine B, three different pillar arrangements were studied in four successive sets of gate roads. The study sites at mine B, shown in figure 6, were described by Listak (*69*). At mine C, two sets of pillars were studied in one four-entry headgate (*68*). All three of the Bureau's study mines were working the Pittsburgh Seam.

At mine D, in the Lower Kittanning Seam, pairs of headgate pillars in two successive longwall panels were monitored (*78-79*). The pillars in the first panel headgate were 42 ft wide, but the second panel had a design with 22-ft-wide pillars (fig. 7). At mine E, four pairs of longwall pillars in the Harlan Seam were instrumented with more than 80 stressmeters (*97*). The depth of cover over some of the instrumented pillars was as great as 1,560 ft (fig. 8).

In all, measurements from 16 stressmeter arrays were available for analyzing the magnitude of the abutment loads. In all 16 cases enough data were collected to characterize the headgate front abutment measured at the T-junction. Once the longwall had passed an array, individual meters were often destroyed and in some cases access to the entire array was lost. As a result, valid data on the side abutment stress, measured long after the face was passed, were available only from six arrays. Measurements of the tailgate abutment proved even more difficult to obtain and were available only in one case.

The first step toward characterizing the magnitude of abutment loads is determining the abutment angle ß. Data on the measured side abutment stress are shown in table 1. The side abutment stress (σ_s, in pounds per square inch) can be related to the side abutment load per foot of entry (L_s) as

$$\sigma_s = [(L_s)(l_p + w_e)]/[(A_t)(144)], \tag{7}$$

where A_t = total load-bearing area of pillars, ft^2,

and l_p = pillar length, ft.

After substituting from either equation 2 or 3 into equation 7, equation 7 can be rearranged to solve for the abutment angle ß. For the data shown in table 1, the abutment angles ranged from 25.2° to 10.7°. It was concluded that a value of ß = 21° would yield appropriately conservative estimates of the side abutment load for longwall pillar design.

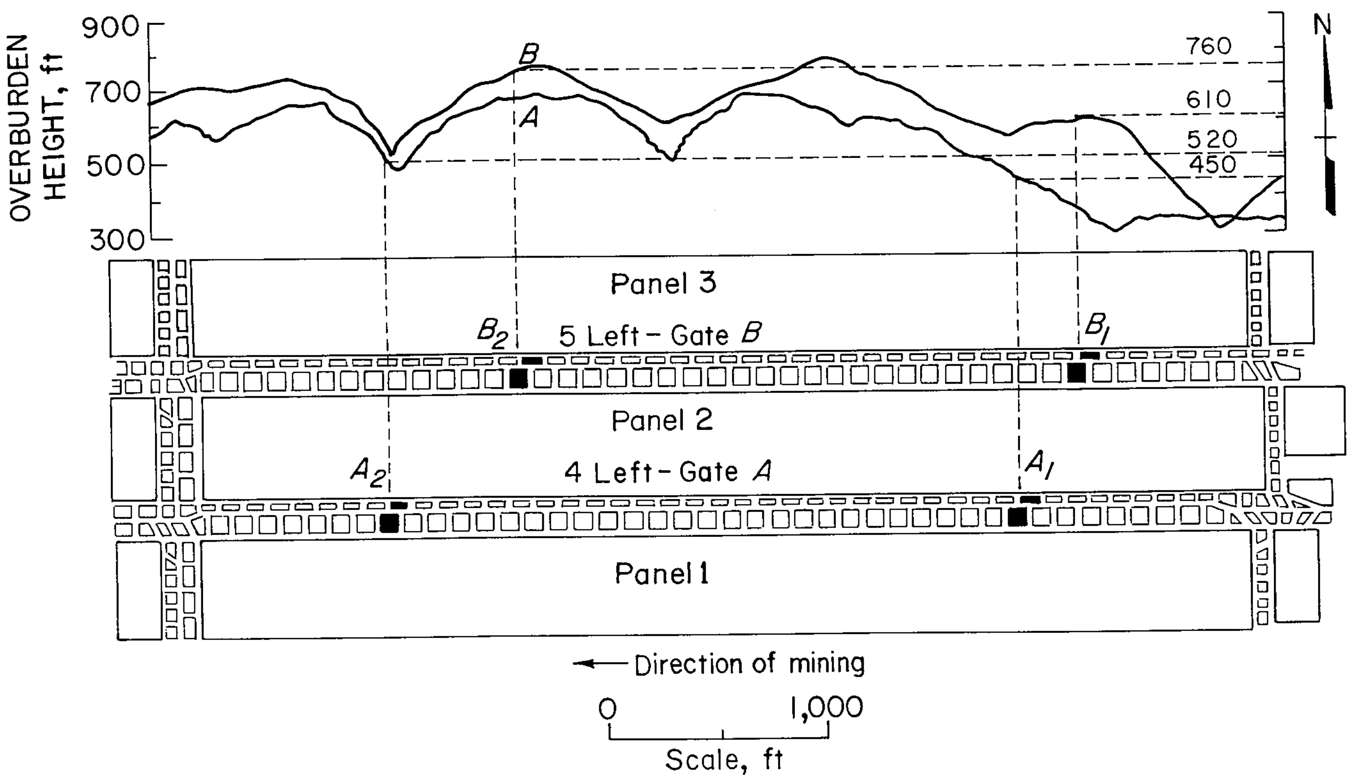

Figure 5.–Location of the four stressmeter arrays at mine A (after Allwes (5)).

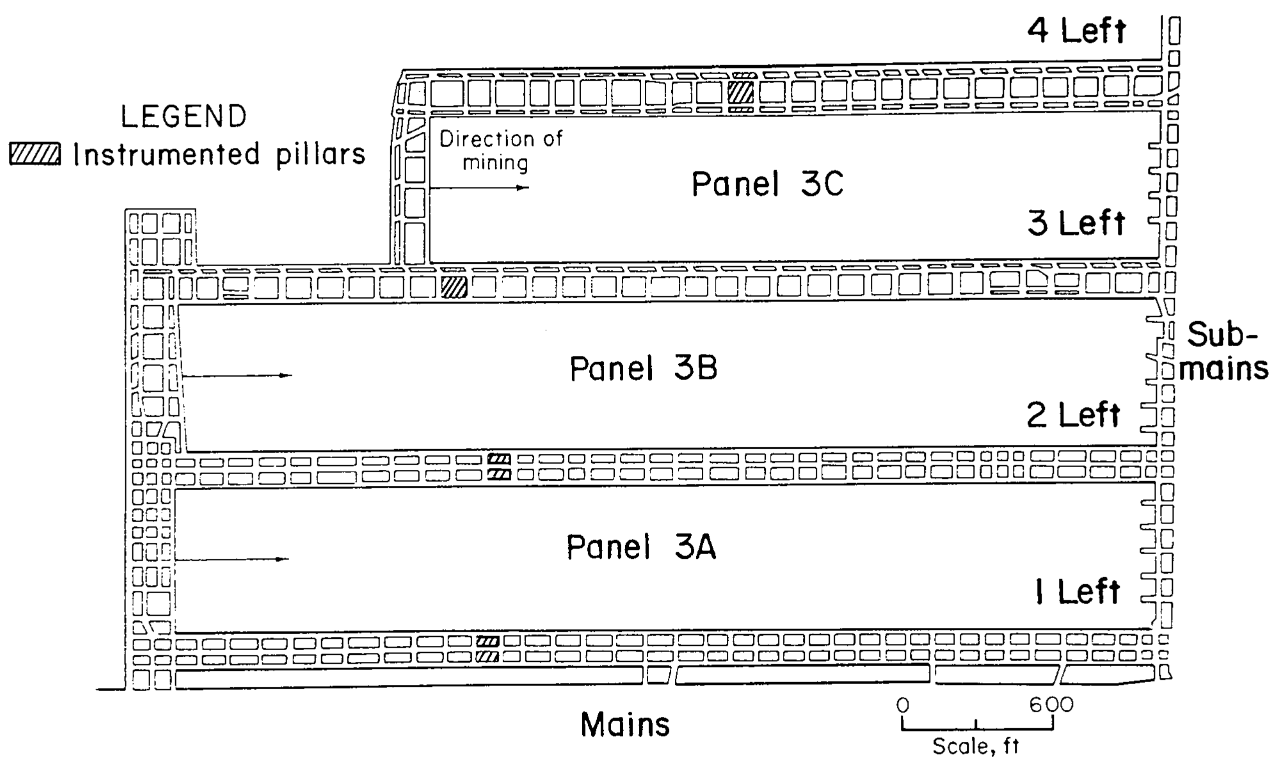

Figure 6.–Panel layout and instrumentation sites at mine B (after Listak (69)).

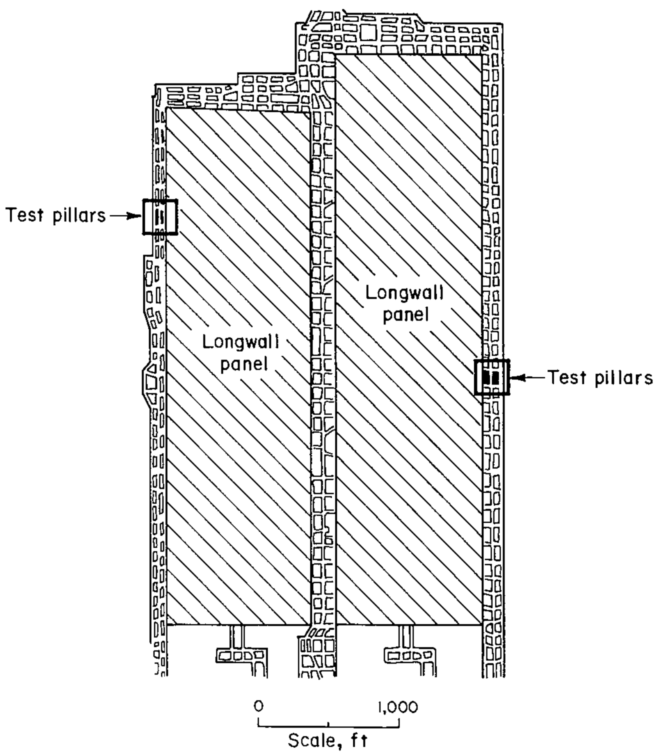

Figure 7.–Panel layout and instrumentation sites at mine D (after Mark (*74*)).

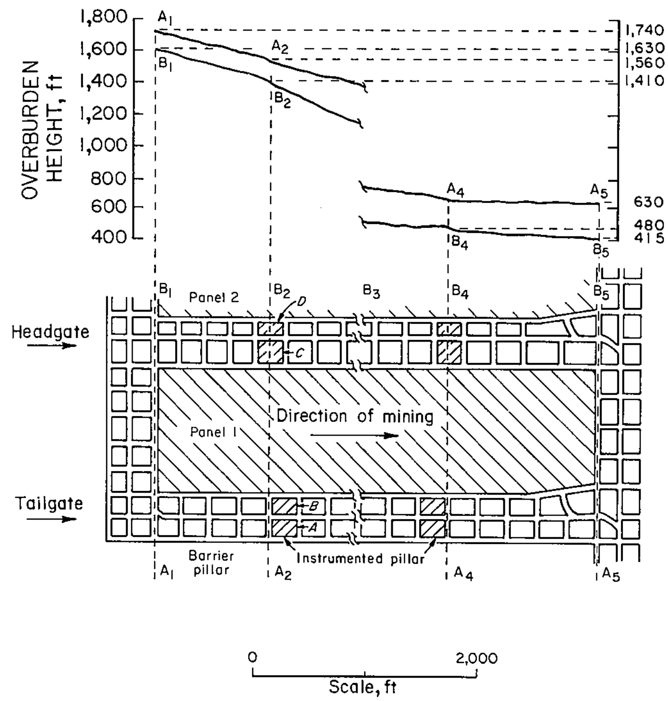

Figure 8.–Location of stressmeter arrays at mine E (after Scheurger (*97*)).

The next step is determination of the front abutment factor for headgate loading. Data on the measured average front abutment stresses are shown in table 2. Table 2 also contains estimates of the headgate front abutment factors (F_h), determined as the percent of the side abutment stress calculated assuming ß = 21°. Except for three anomalously low values obtained from mine B, the values of F_h are remarkably consistent for field data. A value of F_h = 0.5 appears to provide a reasonable estimate for design purposes.

Unfortunately, data for determining the tailgate front abutment factor (F_t) are available only from array 1 in mine A. Although insufficient for a strong conclusion, the data from this array indicated that a value of F_t = 0.7 should be used in ALPS as a first approximation.

Table 1.–Field measurements of the side abutment stress

Stressmeter array	Depth of cover, (H), ft	Pillar widths, (w), ft	Measured abutment stress (σ_s), psi	Abutment angle (β), deg
Mine A: 2 . .	520	32, 92	637	21.8
Mine B:				
2	650	45, 45	1,242	25.2
3	600	80, 20	403	10.7
4	455	20, 80, 20	344	17.3
Mine D: 1 . .	760	42, 42	1,380	18.5
Mine E: 3 . .	630	72, 72	663	20.3

Table 2.–Field measurements of the front abutment stress

Stressmeter array	Depth of cover, (H), ft	Pillar widths, (w), ft	Measured abutment stress (σ_s), psi	Front abutment factor (F_h)
Mine A:				
1	455	32, 92	241	0.61
2	520	32, 92	233	.38
3	620	32, 92	371	.52
4	760	32, 92	486	.47
Mine B:				
1	570	45, 45	318	.39
2	650	45, 45	164	.16
3	600	20, 80	111	.14
4	455	20, 80, 20	70	.17
Mine C:				
1	650	75, 75, 75	289	.59
2	650	75, 75, 75	250	.51
Mine D:				
1	760	42, 42	757	.48
2	630	22, 22	650	.40
Mine E:				
1	1,560	72, 72	1,720	.68
2	1,410	92, 52	1,290	.57
3	630	72, 72	328	.48
4	490	92, 52	184	.45

The final aspect of the abutment load problem is to determine the distribution of the abutment load between the chain pillars and the adjacent barrier pillar or future longwall panel (fig. 9). Because the front abutment data set is

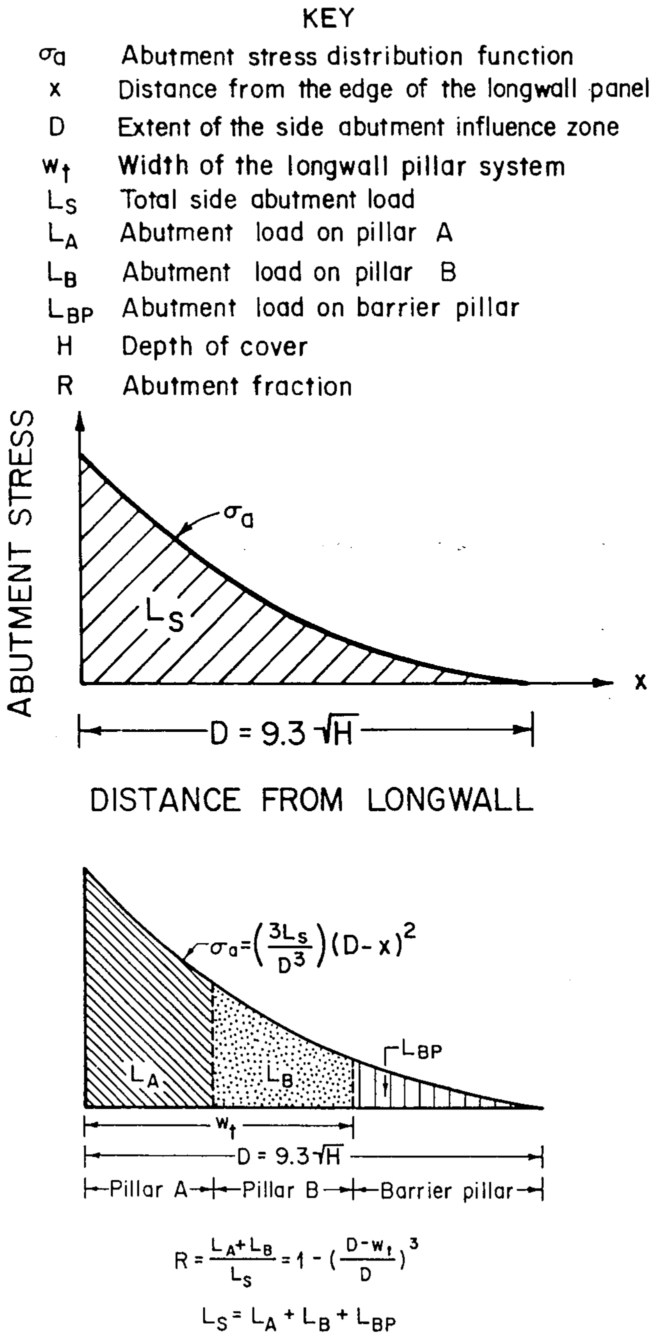

Figure 9.–Distribution of the abutment pillar load.

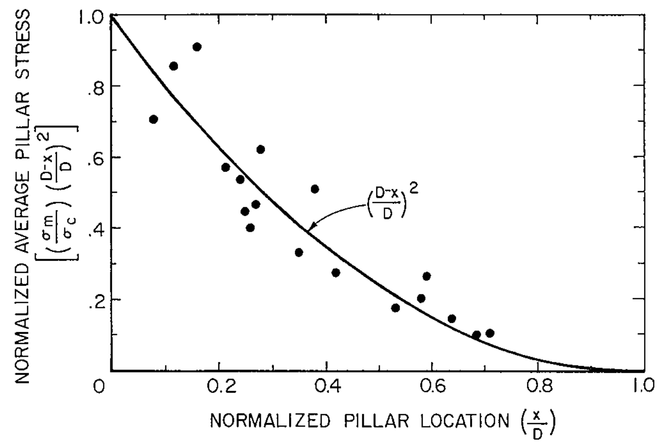

Figure 10.–Comparison between measured average front abutment pillar stresses and calculated abutment pillar stresses, assuming an inverse-square stress distribution function.

the most complete, it was used in the analysis. First, the extent of the abutment influence zone D was calculated for each array using equation 6. Then the normalized location of the pillar centerline, relative to D, was determined for each pillar. Next, the magnitude of the side abutment was calculated using equations 2, 3, and 4. Theoretical pillar loads were then calculated as shown in figure 9, assuming an inverse-square stress decay function. The theoretical pillar loads were converted to calculated average pillar stresses by dividing by the pillar widths. The measured average pillar stress changes were then determined from the field measurements. Figure 10 shows the ratios determined by dividing the measured average pillar stress (σ_m) by the calculated average pillar stress (σ_c), normalized with respect to the theoretical inverse-square stress decay function. The field measurements seem to follow the trend of the stress decay function very well. It was concluded that the fraction R of the total side abutment carried by the chain pillars may be estimated as

$$R = 1 - \left[\frac{D - w_t}{D} \right]^3, \qquad (8)$$

where w_t is less than D. Where w_t is greater than D, or where there is no adjacent unmined panel or barrier pillar, then R = 1.

LONGWALL PILLAR STRENGTH

The other component of the longwall pillar design problem is estimation of the pillar strength. For multientry gates, it is first necessary to determine the strength of the individual pillars used. Then the individual pillar strengths are used to derive an estimate of the load-bearing capacity of the longwall pillar system.

The strength of coal pillars has been the subject of much research. Many approaches to estimating pillar strength have been proposed, including analytical (*10, 108*), observational (*71*), and numerical (*67, 90*). Probably the most widely used methods have been empirical.

Empirical pillar strength formulas have been developed from numerous laboratory and in situ tests, and have been validated by mining experience in many coal regions (*49*).

Three empirical pillar strength formulas have been identified as being most applicable to U.S. mining conditions (*11*). These are the modified Obert-Duvall equation (equation 9), the modified Holland-Gaddy equation (equation 10), and the Bieniawski equation (equation 11).

$$S_p = S_1 \, (0.78 + 0.22 \; w/h), \tag{9}$$

$$S_p = S_1 \, (w/h)^{0.5}, \tag{10}$$

$$S_p = S_1 \, (0.64 + 0.36 \; w/h), \tag{11}$$

where S_p = pillar strength, psi,

S_1 = in situ coal strength, psi,

w = pillar width, ft,

and h = pillar height, ft.

The in situ coal strength S_1 used in equations 9 through 11 is defined as the strength of a full-scale cube of coal measuring 36 in on a side. In situ coal strength is generally much lower than the compressive strength of a laboratory-scale specimen of the same coal, because the full-scale specimen contains many more natural defects. It is widely accepted that the size effect can be expressed as

$$S_1 = S_c \left(\frac{d_{fs}}{d} \right)^{\alpha}, \tag{12}$$

where S_c = strength of laboratory specimen, psi,

d = least dimension of laboratory specimen, in,

d_{fs} = edge length of a full-scale coal cube, in,

and α = size effect scaling factor.

It has been commonly assumed that the scaling factor α for most coals is near -0.5 (*11*, *32*, *55*). The primary evidence for this conclusion is test results reported for the Pittsburgh Seam (fig. 11). Unfortunately, for many seams, using α = -0.5 to adjust laboratory data results in unrealistically low values of the in situ strength. In many seams, the size effect observed in laboratory tests is often much less pronounced. In their classic study of British coals, Evans and Pomeroy (*29*) found that α ranged between -0.17 and -0.32. Data presented by Wang (*104*) for the Pocahontas No. 3 seam in West Virginia indicate α = -0.17 (fig. 11). Similar findings for the Upper Freeport Coal in Pennsylvania were recently reported by Mrugala and Belesky (*85*). Mrugala and Belesky also speculated that the actual value of α might be related to the cleat density of the coal. Blocky coals with widely spaced cleats, like the Pittsburgh, tend to have higher

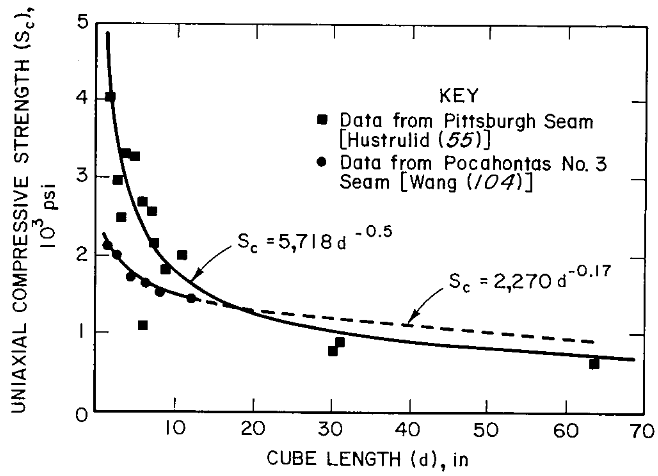

Figure 11.–Effect of specimen size on compressive strength observed in two coal seams.

specimen strengths and larger absolute values of α, while highly cleated coals like the Pocahontas No. 3 have lower laboratory strengths but experience less of a size effect.

While the strength of laboratory-size specimens varies widely from seam to seam, in situ coal strengths may fall within a relatively narrow range. The author has found that meaningful results can usually be achieved with empirical pillar strength formulas if an average value of the in situ strength, taken as S_1 = 900 psi, is used. More accurate estimates of the in situ coal strength may be obtained by back-calculation if well-documented examples of failed and unfailed pillars are available.

If laboratory testing is used to estimate S_1, the scaling factor must be determined for the seam in question. This can be done by testing statistically significant numbers of specimens of several sizes. Great care must also be exercised in conducting a testing program, because laboratory tests on coal are often unreliable because of sampling bias, integrity loss during specimen preparation, and platen effects during testing. Misleading results from a poorly conducted or incomplete program of coal strength testing can do more harm than good.

Because the empirical pillar strength formulas were developed primarily for room-and-pillar mining at shallow depth, they have been used mostly for pillars with moderate width-to-height ratios. Conventional longwall pillars, which are typically at greater depth and subject to abutment loads, often have width-to-height ratios of 8 or more. The field studies described in the previous section offered an opportunity to determine the strength of wide pillars. Eight of the instrumented pillars apparently reached their ultimate load-bearing capacity during the time that measurements were being conducted. Failure was indicated by significant drops in the average pillar stress, yielding of the pillar edges, and clear shifts in the stress peaks toward the pillar core.

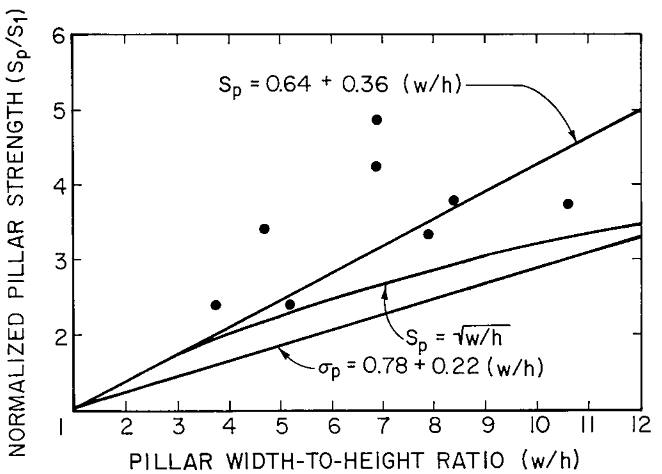

Figure 12.–Comparison of observed strength of eight longwall coal pillars with predictions of three empirical formulas.

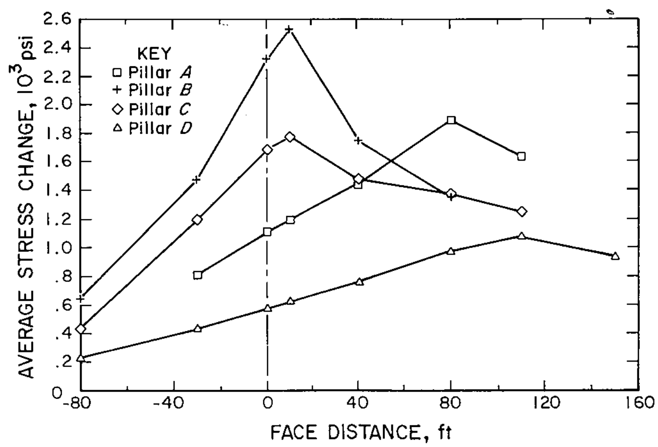

Figure 13.–Average pillar stress changes measured in four deep-cover pillars at mine E (pillars referenced to figure 8).

The total load on these pillars at failure was the sum of the longwall abutment loads, as measured by the stressmeters, and the development loads. Two-dimensional finite-element modeling was again used to determine the development loads (*74*). Figure 12 compares the measured pillar strengths, normalized relative to the in situ coal strength, with the predictions of the three empirical pillar strength formulas (equations 9-11). The average value of the in situ coal strength, 900 psi, was used in all the analyses. Although there is considerable scatter, all three equations seem to follow the trend of the data. The best fit is apparently obtained with the Bieniawski equation, so the decision was made to employ it in ALPS.

Once the strength of the individual pillars has been calculated, the next step is determination of the load-bearing capacity of the longwall pillar system. First, the load-bearing capacity of the individual pillars (B_p, in pounds per foot of gate entry) is calculated by multiplying the pillar strength by the load-bearing area:

$$B_p = \frac{S_p \, w \, l_p \, (144)}{(l_p + w_e)} \, . \tag{13}$$

In ALPS, the load-bearing capacity of the pillar system (B, in pounds per foot of gate entry) is then taken as the simple sum of the individual pillar resistances:

$$B = \Sigma \, B_p \, . \tag{14}$$

Because longwall loadings are not in general evenly distributed between the pillars, equation 14 rests on two hypotheses. The first is that a pillar's load-bearing capacity remains essentially constant after the pillar has been loaded to its limit. For wide pillars that do not fail violently, the coal near the pillar edge is expected to provide enough constraint to maintain most of the load-bearing capacity of even a yielded inner core (*108*). Field measurements seem to support this hypothesis. Figure 13 shows the stress histories of the four pillars under deep cover at mine E. These pillars apparently yielded, but none apparently suffered a loss in strength of more than 20 pct during the time the measurements continued.

The second hypothesis is that overloading an individual pillar will not lead to instability in the entries adjacent to that pillar. There are many examples of successful longwall mining with combinations of large and small pillars (*6, 22, 35, 44*), and in several cases the pillars are known to have yielded without adversely affecting entry stability (*79*). The conclusion appears to be that the loading on individual pillars is less important than whether the entire pillar system is strong enough to resist the applied load.

CALCULATION OF STABILITY FACTORS

The third aspect of the ALPS approach is the determination of the stability factor (SF). The stability factor is simply the load-bearing capacity of the pillar system (B) divided by the design loading (L):

$$SF = B/L \, . \tag{15}$$

One of three design loadings may be used, depending upon the proposed use of the gate entry system. The loading experienced by pillars at the T-junctions in the headgate, or in the tailgate during first panel mining, is called headgate loading. Headgate loading (L_H) consists of the development loads plus the first front abutment:

$$L_H = [L_d + (L_s)\,(F_h)\,(R)] \, . \tag{16}$$

Pillars that are expected to protect bleeder entries will be subjected to the development load and the first full side abutment, or bleeder loading (L_B):

$$L_B = [L_d + (L_s)(R)] . \quad (17)$$

Barrier pillar loads may also be determined from equation 17.

The most severe longwall service loading is tailgate loading (L_T), experienced during the mining of the second and subsequent panels. Tailgate loading consists of the development load, the first side abutment, and the second front abutment:

$$L_T = [L_d + (1 + F_t)(L_s)] . \quad (18)$$

Once the pillars are isolated in the gob, they are subjected to two side abutments.

The final step in the analysis is to determine whether the stability factor calculated in equation 15 is adequate. Selection of appropriate design criteria is discussed in the next section.

VERIFICATION OF THE ALPS METHOD

Design criteria have been established for ALPS through back-analysis of two groups of case histories. The first group, which contains 52 pillar designs used in 31 different mines, was obtained from the literature and from personal observations. As shown in figure 14, the case histories included in this group are distributed all over the eastern coalfields.

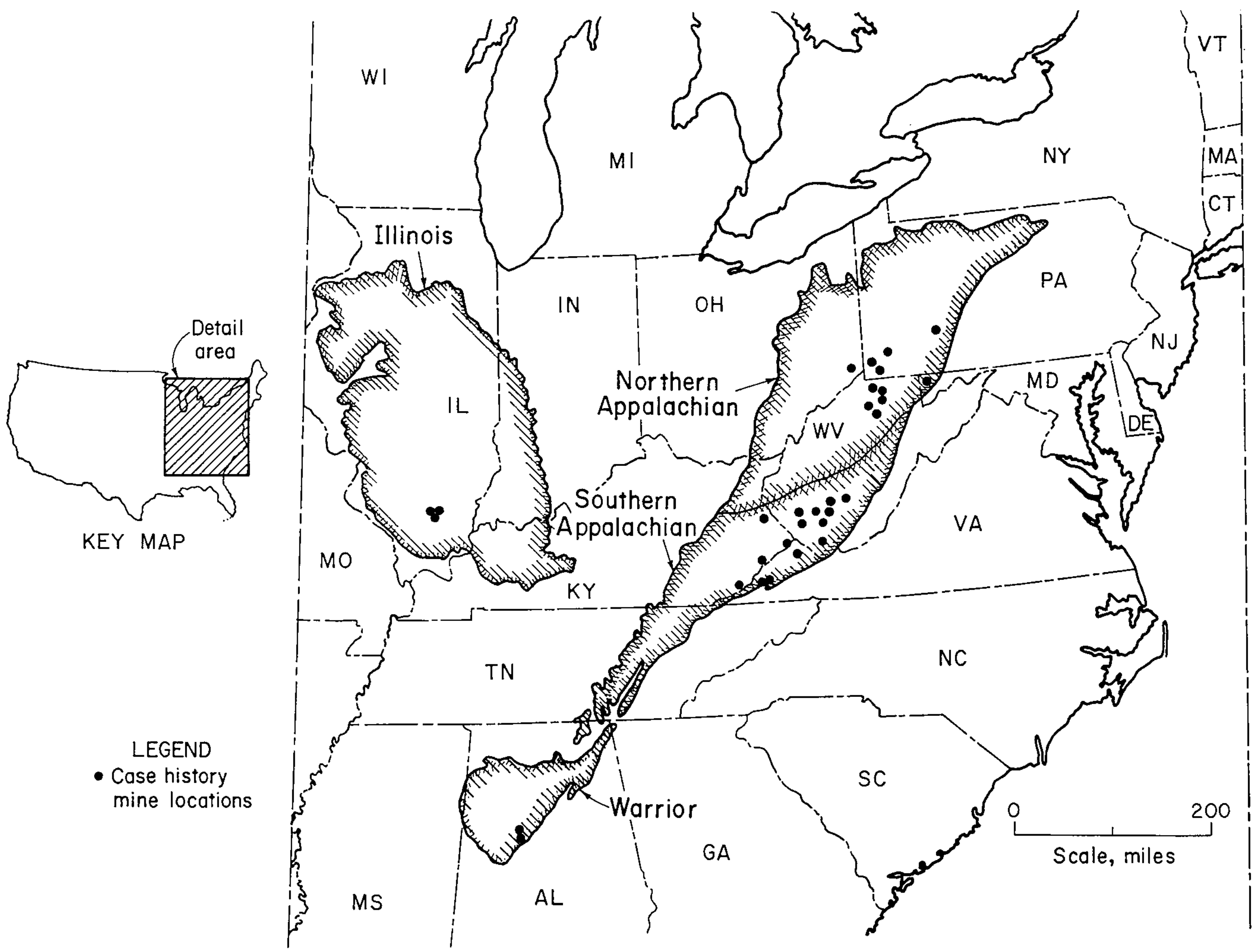

Figure 14.–Coalfields of the Eastern United States and the locations of the first set of case histories.

The details of each case history are contained in tables 3 and 4. In these tables the case histories have been separated into "unsuccessful" and "successful" designs. The unsuccessful designs (table 3) were those in which intolerable entry conditions occurred, including roof deterioration and falls, severe pillar sloughing and floor heave, and in one case even pillar bumps. In all cases the problems could be attributed to excessive abutment stresses. Often the mine subsequently changed the gate entry design, either by increasing the pillar size or by installing more supplemental support.

The successful designs, shown in table 4, are ones in which minimal ground control problems were reported. Most of these designs have been in use for many years. About half of the successful designs are from mines that also reported unsuccessful designs described in table 3. In these cases the improved ground conditions are attributable to a change in pillar design or to a decrease in the depth of cover.

Tables 3 and 4 also show the ALPS stability factors calculated for each case history. The average in situ coal strength of 900 psi was used in all the analyses, as was the tailgate loading criterion (except where the design was used only in the headgate as noted).

The most important conclusion from the case histories is that the failed cases almost all had stability factors of less than 1.0, while the successful cases usually had stability factors of 1.0 or better (fig. 15). Based on this observation, stability factors of 1.0 to 1.3 are currently recommended as appropriate for use with ALPS.

A second observation is that ALPS seems to work well over a wide range of locations, mining geometries, and depths of cover. Six case history mines work the Pittsburgh Seam at depths of less than 1,000 ft, while three other mines in the southern Appalachians operate under 2,000 ft of cover. Both tables contain examples of three-entry systems, four-entry systems, designs that employed equal-sized pillars, and designs that used pillars of different sizes.

There does appear to be some variation in the required stability factor from mine to mine. For example, one successful mine in Virginia uses a design with SF = 0.67, while most of the successful Pittsburgh Seam designs have SF = 1.3 or greater. Analysis of the second set of case histories offers some possible explanations.

The second set of case histories was obtained from the survey of U.S. longwall faces reported by Agbede and Whitehead (*2*). Nearly 70 longwalls, representing about 65 pct of the operating longwalls at the time, responded to the survey by filling out detailed questionnaires covering ground conditions, mine design, and equipment. After elimination of a handful of cases in which the mine was dissatisfied with the performance of the gate entry design, ALPS stability factors were determined for the remaining, presumably successful, cases. The average in situ coal strength and the tailgate loading criterion were again used in all the analyses.

Table 3.-ALPS results for unsuccessful case histories

Seam	Location	Depth of cover (H), ft	Pillar widths (w), ft	Pillar height (h), ft	Panel width (P), ft	ALPS SF
Blue Creek	AL	1,500	80, 80	6.0	500	0.70
Do	AL	1,500	64, 64, 64, 64	6.0	450	.76
Do	AL	2,000	105, 105, 105	7.0	600	[1].84
Do	AL	2,000	115, 115, 115	7.0	600	.94
Campbell Creek ..	WV	1,100	62, 62	7.3	480	.56
Do	WV	900	41, 41, 41	6.0	650	.58
Do	WV	1,050	41, 41, 41	7.0	700	[1].60
Do	WV	1,000	53, 53, 53	6.0	600	.73
Do	WV	1,000	81, 41	7.0	700	.75
Eagle	WV	1,250	71, 51	5.5	520	.62
Elkhorn No. 2	KY	1,100	57, 57	5.0	550	.71
Harlan	KY	1,400	92, 52	11.0	500	.43
Do	KY	2,000	92, 52	11.0	500	[1].57
Imboden	KY	1,800	100, 28	7.0	650	.40
Pittsburgh	WV	800	32, 32, 32	6.5	480	.86
Do	PA	650	42, 42	7.0	570	.86
Do	OH	800	52, 62	6.5	500	.87
Do	PA	850	68, 43	6.0	500	.90
Do	WV	900	68, 68	7.5	420	1.03
Pocahontas No. 3 .	VA	2,000	31, 81, 31	5.0	600	.37
Do	WV	1,400	42, 42, 42	4.0	360	.60
Do	WV	1,400	42, 42, 72	4.0	360	.89
Powellton	WV	800	41, 41	6.0	580	.53
Do	WV	650	50, 30, 30	4.5	365	.75
Taggart	VA	1,500	95, 95	7.0	685	.59
Upper Freeport ...	MD	650	55, 55	7.5	600	.90
Warfield	KY	800	50, 30, 30, 50	5.0	600	.92

SF Stability factor. [1]Headgate failure.

Table 4.–ALPS results for successful case histories

Seam	Location	Depth of cover (H), ft	Pillar widths (w), ft	Pillar height (h), ft	Panel width (P), ft	ALPS SF
Blue Creek	AL	2,000	20, 200	7.0	600	1.16
Do	AL	1,500	20, 180	7.0	600	1.56
Do	AL	1,500	130, 60, 130	6.0	500	1.58
Campbell Creek ..	WV	1,000	81, 41	6.0	600	.78
Do	WV	700	41, 41, 41	6.0	650	.86
Do	WV	850	73, 53	6.0	600	1.02
Dorchester	VA	750	70, 50	5.5	625	1.07
Eagle	WV	1,050	73, 73	6.0	700	.90
Do	WV	800	71, 51	6.0	520	1.04
Elkhorn No. 2	KY	575	37, 57	5.0	550	1.29
Herrin No. 6	IL	650	44, 64	7.5	750	1.14
Do	IL	650	56, 64	7.5	750	1.27
Do	IL	650	44, 42	7.5	750	[1]1.30
Imboden	KY	1,000	100, 28	7.0	650	.87
Lower Kittanning .	WV	720	32, 32	5.0	980	[1]1.11
Do	WV	720	62, 62	5.0	980	1.37
Do	PA	750	62, 62	5.0	585	1.38
Pittsburgh	PA	1,000	34, 74, 84	6.0	750	1.20
Do	OH	800	34, 94	6.5	500	1.32
Do	WV	750	32, 92	6.0	450	1.33
Do	WV	900	85, 85, 85	7.0	520	1.47
Do	WV	900	93, 93	6.4	420	1.48
Do	PA	600	20, 93	7.0	570	1.52
Pocahontas No. 3 .	VA	2,000	21, 121, 21	5.0	600	.67
Do	WV	1,400	42, 42, 107	4.0	360	1.43

SF Stability factor. [1]Design used in headgate only.

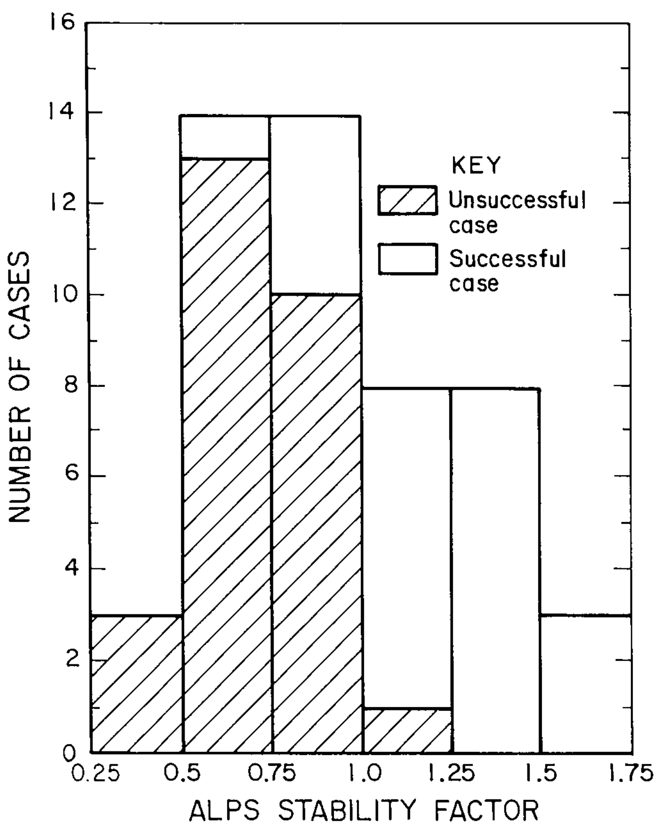

Figure 15.–ALPS stability factors (SF) calculated for the first set of case histories.

The results are shown in figure 16. A large group of faces, representing 30 pct of the total, fell within the stability factor range of 1.0 to 1.5. Slightly more than 50 pct of the faces were apparently overdesigned, with stability factors in excess of 1.5, suggesting that many mines could benefit from using ALPS to optimize their pillar sizes.

Approximately 20 pct of the mines in the survey were using designs with stability factors of less than 1.0. To help determine what other factors might allow these mines to use lower stability factors, the data were analyzed using multiple regression. The statistical analysis related ALPS stability factors to a number of parameters, including roof quality, floor quality, the presence of water, gate entry width, bolting plan, and supplemental support.

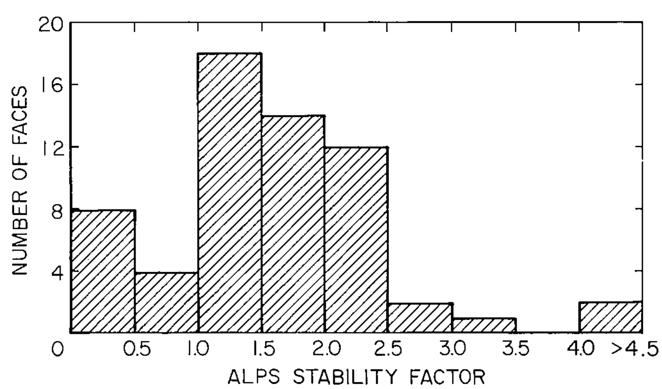

Figure 16.–ALPS stability factors (SF) calculated for the BCR data bank case histories.

The analysis found the strongest correlation between ALPS stability factors and roof quality. Mines with exceptionally strong sandstone roof tended to use pillar designs with low stability factors, while mines with weak, slickensided roof used designs with higher stability factors. Correlations were also found between ALPS stability factors and gate entry width, bolting plan, and floor quality. All of the correlations became much weaker when designs with SF > 1.5 were included in the analysis, indicating again that there are probably few benefits to using oversized pillars.

ALPS appears to provide very good first approximations of the pillar sizes required for gate entry stability. It is therefore well suited for initial feasibility studies. In an operating mine, past experience can be directly incorporated into ALPS. ALPS stability factors can be back-calculated for both successful and unsuccessful areas, and the trend of the back-analyses should reveal the minimum stability factor that provides adequate ground conditions. That minimum should then be maintained in subsequent panels as changes occur in the depth of cover, coal thickness, or entry layout. Where no experience is available, operators should begin with a stability factor in the range of 1.0 to 1.3 and then adjust as they observe pillar performance.

Further refinements are possible by considering the effect of artificial support and rock mass quality. Current Bureau research is directed toward quantifying the relationships among ground conditions in the gate entries, pillar design, roof and floor quality, and entry support. The ultimate goal is the development of a complete gate entry design package, of which ALPS will be a central part.

OTHER METHODS FOR CONVENTIONAL LONGWALL PILLAR DESIGN

ALPS is not the only available conventional pillar design method. Computer-assisted design procedures for longwall mining, based on sophisticated numerical models, have also been developed by Bureau researchers (*65*), as well as observational techniques that use pressure measurements to evaluate the stability of longwall pillars (*71*). These approaches have been described in detail in other Bureau reports.

In addition, methods for designing longwall pillars have been proposed by Carr and Wilson (*19*), Choi and McCain (*22*), and Hsuing and Peng (*52*). Like ALPS, these methods can be used to provide *quantitative* estimates of the pillar sizes required to support the abutment loads. Unfortunately, these methods have not found wide use, in part because the original references are often difficult to obtain.

The purpose of the present section is to describe these three methods and show how they might be used in conjunction with ALPS to design longwall pillars. All of the necessary equations are presented, along with discussions of the assumptions used in their development. Predictions from the three methods are also compared with the results of the same case histories described in the previous section. Based on the analysis of the case histories, suggestions are given regarding the selection of input parameters and design criteria.

CARR AND WILSON'S METHOD

In the early 1970's, A. H. Wilson of the British National Coal Board began to develop his innovative approach to estimating the strength of coal pillars (*107*). He also developed the equations for predicting the magnitude of longwall abutment loads that have already been described in the section "Longwall Pillar Loads." These two formulas were used in Great Britain to size the barrier or rib pillars that are often left between British longwall panels. Before Wilson's approach could be applied to the U.S. longwalls, it needed to be adapted for multientry longwall development.

In 1982 Carr and Wilson presented a modified version of Wilson's theory, which included an approach for estimating the abutment load distribution across a multientry gate (*19*). Carr and Wilson's method has been used extensively by Jim Walter Resources (JWR) Mining Division to size pillars at its longwalls.

To use Carr and Wilson's method it is necessary to estimate the pillar strength using Wilson's theory. Wilson's theory may be summarized as follows. When a pillar is initially developed, it consists of two zones, an outer "yield" zone and an elastic inner core. The yield zone has failed and can take no more load, but it provides constraint to the core, which usually provides most of the load-bearing capacity of the pillar. The constraint is in the form of horizontal confining stresses generated by the frictional strength of the yielded coal.

Initially, the greatest stresses in the pillar are found at the boundary between the yield zone and the core (fig. 17). As additional longwall loadings are applied, the average stress in the pillar core increases until it equals the peak stress at the yield zone boundary. Up until this point, which Wilson calls the Limit of Roadway Stability (LRS), both the pillar and entries adjacent to it are expected to be stable. Further loading of the pillar causes the yield zone to expand, resulting in increased horizontal stresses that can damage the nearby roadways. Finally, the Ultimate Limit (UL) is reached when the entire core has yielded. Any additional loads will now be transferred to adjacent pillars.

Wilson provides equations for determining the stress distributions in the pillar at both the LRS and the UL for two different boundary conditions, one in which the surrounding rock is rigid (rigid roof and floor, or RRF, conditions) and the other in which yielding takes place all around the entry (yielding roof and floor, or YRF, conditions). When RRF conditions are assumed, the UL's are

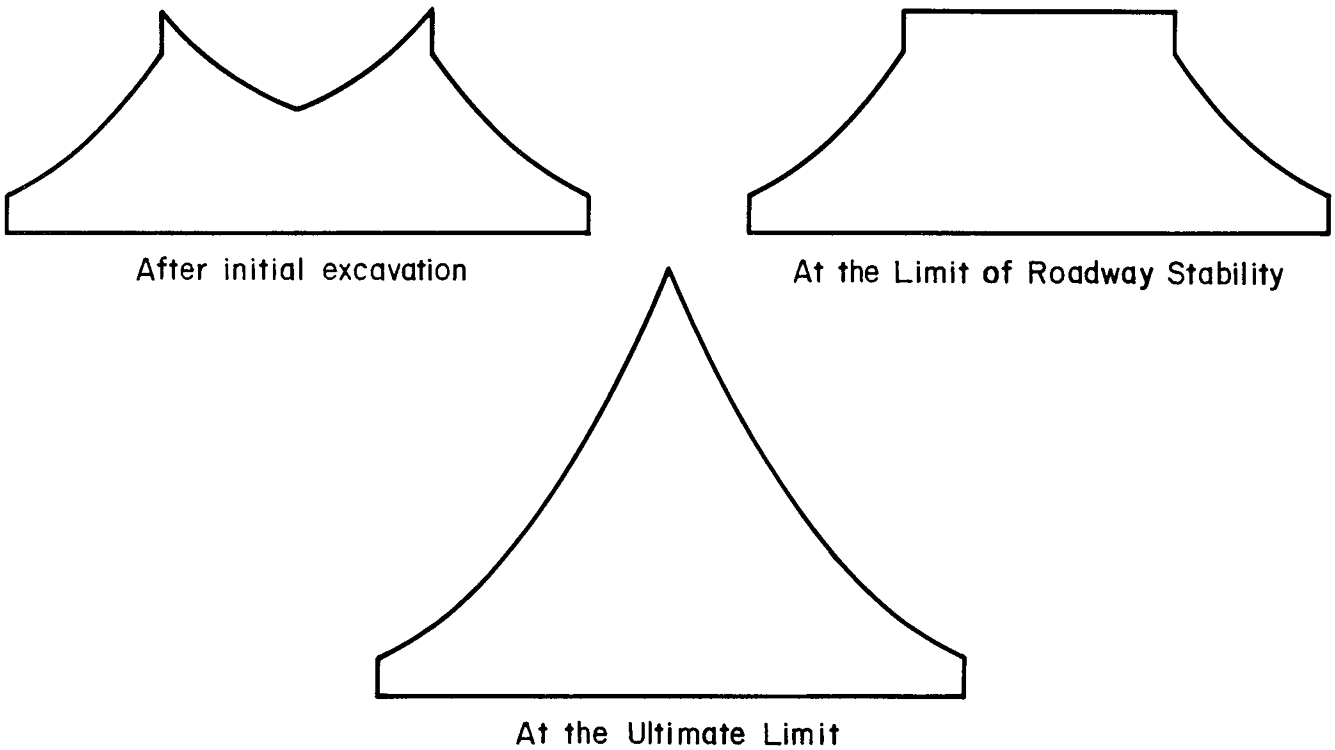

Figure 17.–Theoretical vertical stress distribution profiles in coal pillars (after Wilson (*108*)).

usually unrealistically large, so the more conservative YRF conditions are almost always employed for coal mine applications. The load-bearing capacity of the pillar can be determined by integrating these stress distributions over the area of the pillar. To the best of the author's knowledge, Wilson never published solutions for the pillar strength integrals. The author therefore developed table 5, from which the Wilson pillar strength may be determined.

Wilson's pillar strength concept has been the basis for much modern research in pillar mechanics. In 1989, Wilson received the prestigious Rock Mechanics Award from the Society of Mining Engineers of AIME in recognition of his contributions. His method is not always easy to use in practical design, however. One difficulty is that the method requires three separate material properties, and the results are highly sensitive to the values used. The most important material property is the triaxial stress factor k, which is directly related to the angle of internal friction. Theoretically, k may be determined from laboratory triaxial tests, but recent studies (*79*) suggest that in situ friction angles may be considerably lower than those determined in the laboratory. In his published work, Wilson relied on engineering judgment rather than laboratory tests to estimate k. He typically used k values ranging between 3.0 and 3.5, corresponding to friction angles of 30° to 34°.

The other two material properties are the unconfined compressive strength of fractured or *failed* coal, p', and the in situ strength of *intact* coal, S_1. The strength of failed coal would be very difficult to determine through laboratory testing, but Wilson simplified matters by assuming that p' = 14 psi. The intact coal strength is the least significant of the three material properties, and the same S_1 = 900 psi suggested for use in ALPS may be used in Wilson's method as well.

Another issue is that stress measurements do not seem to support Wilson's assumption that the LRS can be identified as a distinct point in a pillar's loading history (*74*). Wilson's method also tends to predict UL's that are unrealistically low for narrow pillars, but which increase exponentially once the initial elastic core reaches significant size. For these reasons the method's predictions should be carefully checked against previous experience.

Table 5.–Solution of Wilson pillar strength integrals

Boundary condition	Width of yield zone (x_b), ft	Total pillar resistance, lb
YRF	$\left(\frac{h}{2}\right)\left[\left(\frac{8}{p}\right)^{1/k-1} - 1\right]$	LRS = 8(Y1) + 2(Y2) + 3(Y3) + Y4
	w/2	UL = 8(Y1) + 2(Y2)
RRF	(h/F) ln (q/p')	LRS = 8(R1) + 2(R2) + 2(R3) + R4
	w/2	UL = 8(R1) + 2(R2)

LRS Limit of Roadway Stability. UL Ultimate Limit.
RRF Rigid roof and floor. YRF Yielding roof and floor.

$$Y1 = \left(\frac{hp'}{2}\right)\left[\frac{\left(\frac{2x_b}{h} + 1\right)^{(k+1)} - 1}{(k+1)\ (2/h)} - x_b\right],$$

$$Y2 = (l_p - 2x_b)\left(\frac{h\,p'}{2}\right)\left[\left(\frac{2x_b}{h} + 1\right)^{k} - 1\right],$$

$$Y3 = (w - 2x_b)\left(\frac{h\,p'}{2}\right)\left[\left(\frac{2x_b}{h} + 1\right)^{k} - 1\right],$$

$$Y4 = (l_p - 2x_b)\ (w - 2x_b)\ (kq + S_1),$$

$$R1 = \frac{kp'h}{F}\left[\left(\frac{h}{F}\right)\exp\left(\frac{x_bF}{h}\right) - \frac{h}{F} - x_b\right],$$

$$R2 = (l_p - 2x_b)\left(\frac{kp'h}{F}\right)\left[\exp\left(\frac{x_bF}{h}\right) - 1\right],$$

$$R3 = (w - 2x_b)\left(\frac{kp'h}{F}\right)\left[\exp\left(\frac{x_bF}{h}\right) - 1\right],$$

$$R4 = (l_p - 2x_b)\ (w - 2x_b)\ (kq + S_1),$$

where F = a function of $k = \left(\frac{k-1}{k}\right)^2 + \left(\frac{k-1}{k}\right)\tan^{-1} k$,

h = extraction height, ft,

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

l_p = greatest pillar dimension, ft,

p' = uniaxial strength of fractured coal, psf,

q = cover load, psf, = γH,

S_1 = in situ intact coal strength, psf,

w = least pillar dimension, ft,

x_b = width of yield zone, ft,

γ = unit weight of overburden, pcf,

and ϕ = angle of internal friction, deg.

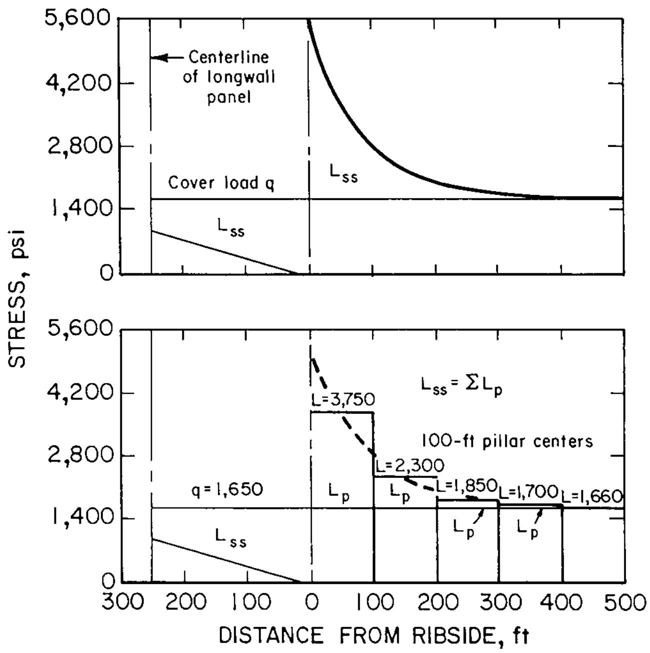

Figure 18.–Determination of the abutment load applied to longwall pillars (after Carr and Wilson (*19*)). (H = 1,500 ft, P = 500 ft.)

Once the strength of each of the chain pillars has been determined, the next step in using Carr and Wilson's method is to estimate the loads that are applied to each pillar. The step-by-step procedure that follows is illustrated in figure 18.

The first step is to calculate the total side abutment load (pounds per foot of gate entry). For panel widths (P) greater than 0.6 times the depth of cover (H), the side abutment load (L_s) is calculated as

$$L_s = (0.15)(\gamma)(H^2), \tag{19}$$

and for $P < 0.6(H)$,

$$L_{ss} = (0.5)\, P\, \gamma\, [H - (P/1.2)], \tag{20}$$

where γ = unit weight of the overburden, pcf.

Next, the peak abutment stress, $\hat{\sigma}$ (pounds per square foot), and the shape constant C (feet) are calculated:

$$\hat{\sigma} = kq + S_1, \tag{21}$$

$$C = \frac{L_s}{\hat{\sigma} - q}, \tag{22}$$

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

ϕ = angle of internal friction, deg,

q = cover stress, psf, = γH,

and S_1 = intact coal strength, psf.

Now the average abutment stress before any load transfer may be calculated for each pillar. If pillar A is bounded at roadway centers of x_1 and x_2, (expressed in feet from the extracted panel), then the average abutment stress, σ_A, (pounds per square foot), may be calculated as

$$\sigma_A = \frac{\hat{\sigma} - q}{x_2 - x_1}\left[C\left(e^{\frac{-x_1}{C}} - e^{\frac{-x_2}{C}}\right)\right]. \tag{23}$$

The total initial average pillar stress, σ_P (pounds per square foot), is the average abutment stress plus the cover stress:

$$\sigma_p = \sigma_A + q \tag{24}$$

The final step is to determine if any load transfer occurs because of pillar yielding. First, the loads applied to individual pillars must be compared with the pillar load-bearing capacities determined using table 5. Both the LRS and the UL must be calculated.

Comparison of stress to strength begins with the pillar located nearest the mined-out panel. Three cases are possible. If the applied stress is less than the LRS, then no load transfer occurs and the adjacent entry furthest from the gob side of the pillar is presumed stable. If the applied stress exceeds the LRS but is still less than the UL, then the entry may be damaged but still no load transfer occurs. Finally, if the applied stress is greater than the UL, the additional load is transferred to the pillar in the next row. Because a failed pillar is assumed to maintain all of its peak load-bearing capacity, only the additional load, called the transferred remnant load (TRL), is carried over to the adjacent pillar.

The analysis is then repeated for the pillar in the next row, except that any TRL must be added to the total initial pillar load already calculated. The process continues until the last pillar is reached. In longwall pillar design, the stability of the future tailgate entry is generally of greatest concern. Therefore, if the tailgate pillar load, including TRL, exceeds the LRS of the tailgate pillar, then the design is assumed to be acceptable.

Several other design criteria have been proposed in addition to the LRS. In their 1982 paper (*19*), Carr and

Wilson suggested that the degree of entry damage may be related to the TRL transferred *from* the tailgate pillar after its UL is reached. In a later paper, Carr and Martin (*16*) proposed using a "pillar resistance to load ratio," or stability factor, for "yield-abutment" designs. They suggested that a stability factor of 1.4 should be used for abutment pillars subjected to tailgate loading, while 1.0 is adequate for single-use pillars.

Several mining companies have used Carr and Wilson's method in actual practice. Its first application in the JWR mines is described in Carr and Wilson's 1982 paper. The method was apparently successful in explaining the poor conditions that had been encountered in several cases, and it indicated that additional support would be necessary to maintain stability in the tailgate unless very large pillars were used. Design criteria were proposed suggesting that if the TRL from the tailgate row of pillars was less than 10,000 ton/ft along the length of the tailgate, then tailgate damage would be limited. The rather large values of TRL can be attributed in part to the low strength values that were assigned to the coal, k = 3.0 and S_1 = 300 psi.

Carr and Wilson's analysis indicated that the ground conditions at JWR could be greatly improved through the use of a single large abutment pillar, rather than several equal-sized pillars. The improvement would be due to the exponential increase in pillar strength predicted by Wilson's theory for very wide pillars. Based on the analyses, JWR began using yield-abutment designs, and these have now been standard at JWR's mines for several years. Carr and Wilson's method is used to size the abutment pillars, but not the yield pillars, in these designs. JWR's successful experience with yield-abutment pillar systems is discussed in more detail in the section "Experience With Yield Pillars in Longwall Mining."

Another example of the application of Carr and Wilson's method to a practical problem is provided by Artler (*6*), writing about Quarto Mining Co.'s longwalls in the Pittsburgh Seam. The method apparently adequately explained both the failure of an early design using equal-sized pillars and the success of a later design using large and small pillars. It was also used to back-calculate conditions in a detailed study of entry conditions in one tailgate, and the results were considered very accurate. Because of the confidence that Quarto developed in the Carr and Wilson approach, the company used it to optimize a proven longwall pillar design.

To help users of Carr and Wilson's method choose appropriate coal strength values and design criteria, the longwall mining case histories described in the previous section were analyzed. Initially, the first set of case histories was analyzed using three different values of k, ranging from 3.0 to 3.5. The other material properties were fixed at p' = 14 psi and S_1 = 900 psi. Results were calculated for the tailgate pillar load and the stability factor design criteria. YRF boundary conditions were used in all the analyses.

The results of the analyses are shown in the form of histograms (figs. 19-24). In the tailgate pillar load histograms (figs. 19, 21, 23) the cases are plotted according

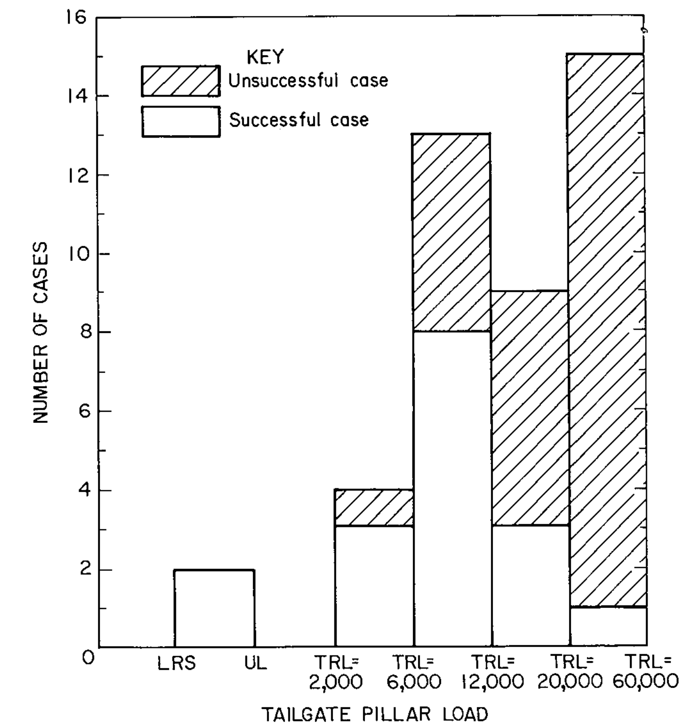

Figure 19.–Tailgate pillar loading calculated using Carr and Wilson's method for the first set of case histories, k = 3.0.

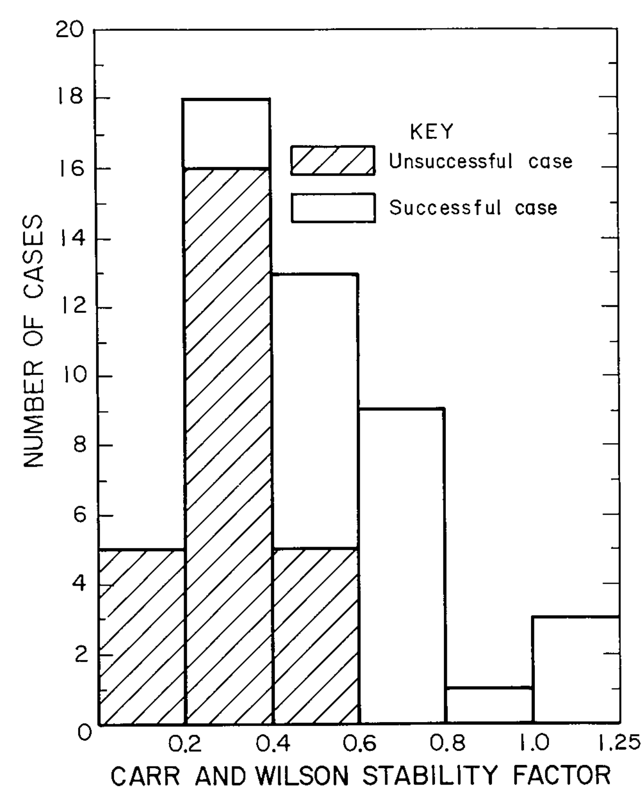

Figure 20.–Carr and Wilson stability factors calculated for the first set of case histories, k = 3.0.

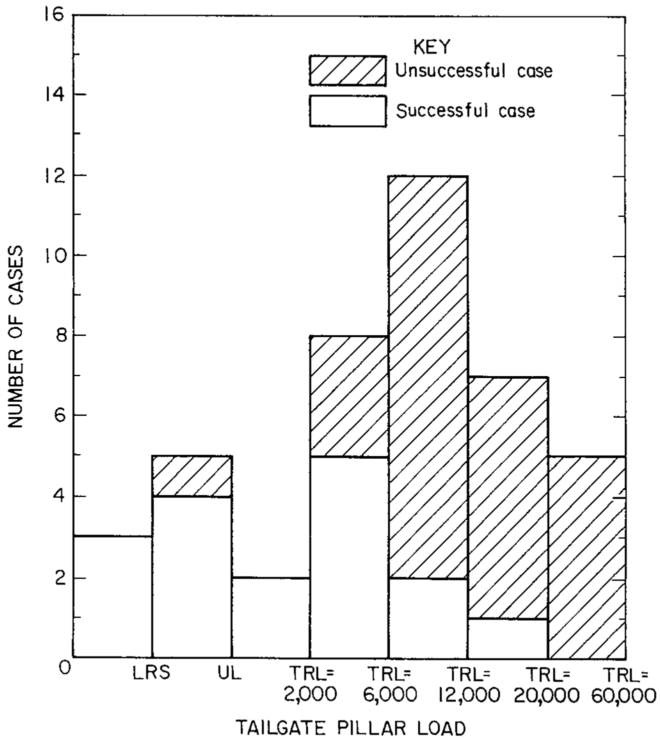

Figure 21.–Tailgate pillar loading calculated using Carr and Wilson's method for the first set of case histories, k = 3.25.

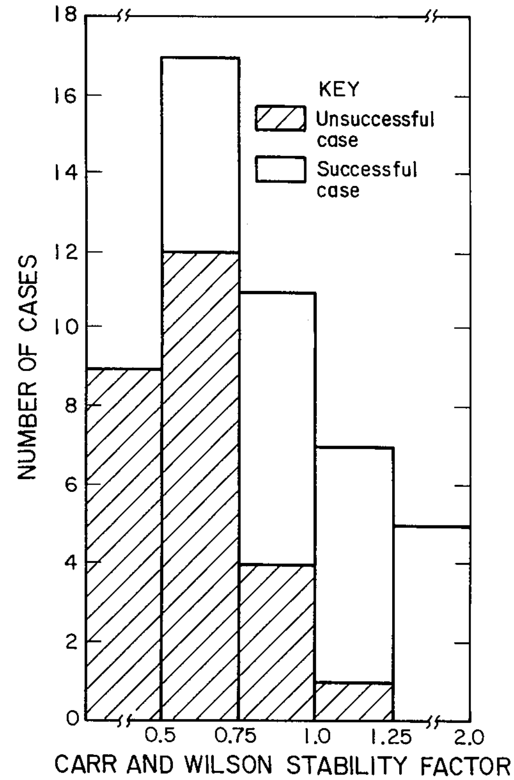

Figure 22.–Carr and Wilson stability factors calculated for the first set of case histories, k = 3.25.

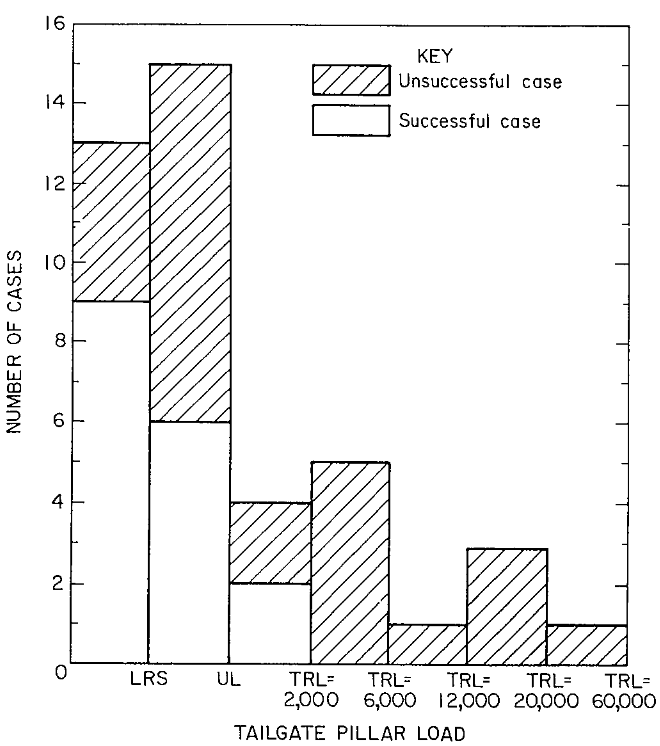

Figure 23.–Tailgate pillar loading calculated using Carr and Wilson's method for the first set of case histories, k = 3.5.

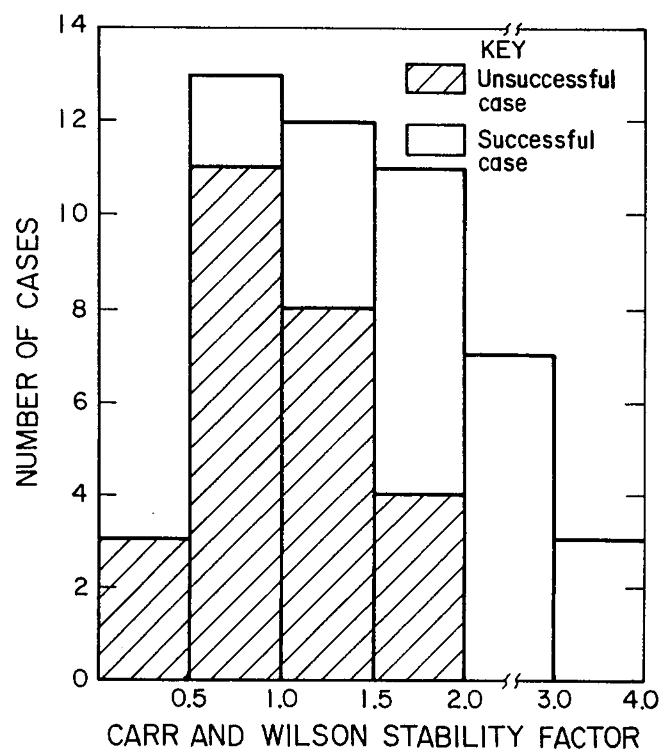

Figure 24.–Carr and Wilson stability factors calculated for the first set of case histories, k = 3.5.

to the loading on the pillar closest to the tailgate. If the loading was less than the LRS, the case was plotted to the left of the LRS. When the loading exceeded the LRS but was less than the UL, then the case was plotted to the left of the UL. The case was plotted according to the calculated TRL when the loading exceeded the UL. Cases in which the design employed a small yield pillar next to the tailgate were excluded from the tailgate pillar load analysis. The stability factors shown in figures 20, 22, and 24 were calculated by dividing the total load-bearing capacity of the pillar system by the load applied to it.

The results show that when k was set at 3.5 (figs. 23 24), Carr and Wilson's method seemed to overpredict the pillar strength. Of the unsuccessful designs, 52 pct were calculated to have no TRL, and 16 pct even met the LRS criterion. Nearly half of the unsuccessful designs also had stability factors in excess of 1.0. Figure 24 indicates that the stability factor approach could be used for designs with k = 3.5, but only if the design criterion was SF > 2.0.

When k was set at 3.0 (figs. 19-20), the opposite problem developed. The method accurately predicted all of the failures, but now none of the successful designs met the LRS criterion. A bare handful of the successful designs achieved SF = 1.0, but none of the unsuccessful designs exceeded SF = 0.6.

The picture improved greatly when k was fixed at 3.25 (figs. 21-22). Now 40 pct of the successful cases showed no TRL, and more than three-quarters had TRL < 6,000 tons/ft of gate road. In contrast, TRL exceeded 6,000 tons/ft of gate entry in 84 pct of the unsuccessful cases. In the stability factor analysis, nearly half of the successful designs exceeded SF = 1.0, while only 4 pct of the unsuccessful designs did.

These trends were confirmed in the analysis of the second set of case histories, the presumably successful designs from the BCR data bank (2). When k was set at 3.0, only a handful of the designs met the LRS criterion. For k = 3.25, slightly more than half of the designs met the LRS criterion, while only 15 pct of the designs were predicted to have TRL greater than 6,000 ton/ft.

In conclusion, it appears that the best results can be achieved with Carr and Wilson's method if k is set at 3.25. Either the LRS or SF = 1.4 may be used as a conservative criterion for preliminary design, and an appropriate lower bound criterion may be TRL < 6,000 ton/ft or SF = 1.0.

CHOI AND MCCAIN'S METHOD

Choi and McCain's method, presented in 1980 (*22*), was the first longwall pillar design method developed specifically for the United States. Their method combined results from field studies, numerical modeling, and practical experience from Consolidation Coal Co.'s working longwalls in the Pittsburgh Seam. In addition to including a technique for sizing longwall pillars, Choi and McCain also addressed the location of different-sized pillars in a multientry longwall gate.

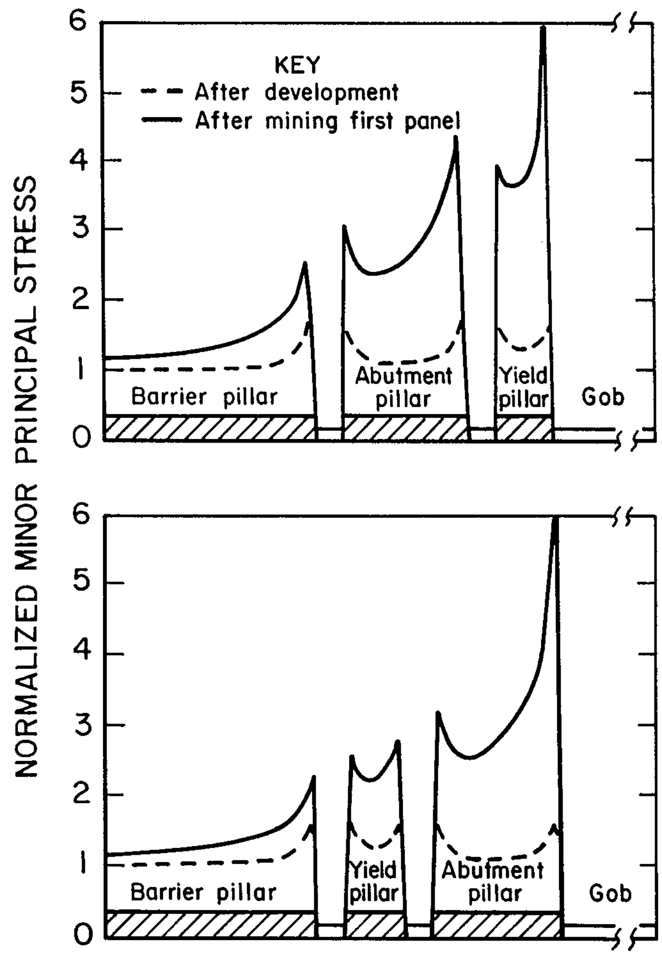

Figure 25.–Finite-element modeling results showing lower stresses on the future tailgate when the abutment pillar is located near the headgate (after Choi (*21*)).

An earlier paper (*21*) contains some important background to the pillar design method. The 1975 paper describes tailgate problems occurring at one Consol longwall. The mine was using a three-entry system, with large abutment pillars next to the tailgate and small yield pillars next to the headgate. Results obtained from numerical modeling indicated that stresses near the tailgate could be reduced by reversing the pillars (fig. 25). Placing the larger, stiffer pillar next to the headgate would be expected to induce a clean break in the overburden, reducing the abutment load resulting from first panel mining. The new pillar placement was apparently successful and was incorporated into the 1980 chain pillar design method.

The 1980 paper discusses in detail the design of a three-entry, abutment-yield longwall pillar system. Most of the paper is devoted to sizing the abutment pillar to

- Support the side abutment pressure,
- Limit the influence of the active panel on the unmined panel, and
- Maintain the stability of the yielding pillar.

Choi and McCain's approach to estimating the magnitude of the abutment load was described in the section "Longwall Pillar Loads." Like Wilson's, their approach is limited to the two-dimensional side abutment. To calculate the pillar strength (S_p), they use the Holland-Gaddy equation (equation 10). The pillar load and pillar strength estimates are combined into a single equation:

$$P = 0.6\,H - 1.2\left[\frac{H^2}{4} - \frac{5}{3}\left\{\left(\frac{w_A l_p}{l_p + w_e}\right)\left(\frac{S_p}{24.9\,SF}\right) - (w_A H) - \left(\frac{w_e H}{2}\right)\right\}\right]^{0.5}, \qquad (25)$$

where w_A = abutment pillar width, ft,

l_p = pillar length, ft,

and SF = safety factor.

Choi and McCain suggest that a safety factor of 1.3 be employed with their method.

In equation 25 the size of the abutment pillar is considered independently of the size of the yield pillar, which is assumed to be 32 ft. The flexibility of Choi and McCain's method is somewhat limited because of this, and because the method can be used only to size abutment pillars for tailgate loading.

Also, equation 25 actually solves for the panel width rather than the pillar width. For sizing pillars, Choi and McCain presented some design curves, which are reproduced in figure 26. These curves were developed using equation 25, assuming that the crosscut spacing would remain constant and that the crosscut spacing would determine the critical pillar dimension once $l_p < w$. In figure 26, the break in the design curves for depths in excess of 800 ft indicate that the entry spacing exceeds the crosscut spacing, and so the pillar length is used to determine the pillar strength. Figure 26 also predicts very small abutment pillars for shallow depths–less than 20 ft for H = 300 ft.

In order to make Choi and McCain's method more generally applicable, equation 26 was rewritten to solve explicitly for pillar width:

$$0 = w^{1.5}\,(C_1) - w(C_2) + C_3, \qquad (26)$$

where $$C_1 = \frac{l_p K}{(l_p + w_e)\,(h)\,(24.9)\,(SF)},$$

$$C_2 = H,$$

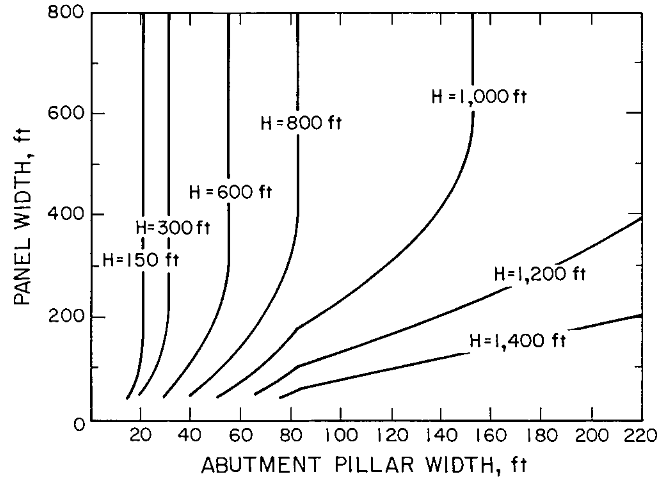

Figure 26.–Design chart for sizing abutment pillars using Choi and McCain's method (after Choi and McCain (22)).

$$C_3 = \frac{3}{5}\left[\left(\frac{0.6\,H - P}{1.2}\right)^2 - \frac{H^2}{4}\right] - \frac{w_e H}{2},$$

and K = Gaddy strength constant, which Choi and McCain say is equal to 7,800 psi $in^{0.5}$ for the Pittsburgh Seam.

Equation 26 is correct only for $w < l_p$. When the required pillar width is greater than the desired pillar length, it is usually necessary to use square pillars to maintain pillar strength. If square pillars are used, then $l_p = w$ and equation 26 must be rewritten as

$$0 = w^{2.5}\;C_1' + w^{1.5}C_1 - w\,C_2 + C_3, \qquad (27)$$

where $$C_1' = \frac{K}{(h)(24.9)(SF)}.$$

Finally, Choi and McCain's method can be used for back-analysis of case histories by rearranging equation 25 to solve for the safety factor:

$$SF = \frac{w^{1.5}C_1''}{w\,C_2 - C_3}, \qquad (28)$$

where $$C_1'' = \frac{l_p K}{(l_p + w_e)\,(h)\,(24.9)}.$$

Equations 26 and 27 can be easily solved using root-finding algorithms (*50*). When the panel width is supercritical, meaning that $P > 0.6\ H$, then a value of $P = 0.6\ H$ must be used in equations 25 through 28.

In their 1980 paper (*22*), Choi and McCain describe the application of their design formula to a mine in the Pittsburgh Seam. The depth of cover was 700 to 800 ft, and the abutment pillar width was 83 ft. The design was judged to be a success because few entry stability problems were encountered. In addition, subsidence measurements taken after the mining of the adjacent panels indicated that the entire pillar system had yielded, meaning that long-term subsidence problems would be minimal.

Choi and McCain believe that their formula has demonstrated its validity for typical Pittsburgh Seam conditions, but that it might need to be reevaluated for other seams, particularly when the overburden exceeds 1,000 ft.

Some additional insight into the performance of Choi and McCain's method can be obtained from the case histories. Only yield-abutment-type designs can be handled by the method, so a total of 28 cases were available from the 2 data sets. Of these, 21 were successful designs and 7 were failures.

The safety factors predicted for these 28 cases are shown in figure 27. At shallow depth, safety factors in the range of 1.0 to 1.3 appear to be appropriate. At greater depth, both the successes and the failures appear to have safety factors that are significantly lower than 1.0. One reason for the declining trend of the safety factor with depth is that the pillar strength formulation used in Choi and McCain's method, the Holland-Gaddy formula, predicts that once pillars are very wide further increases in width have little effect on pillar strength (see figure 12).

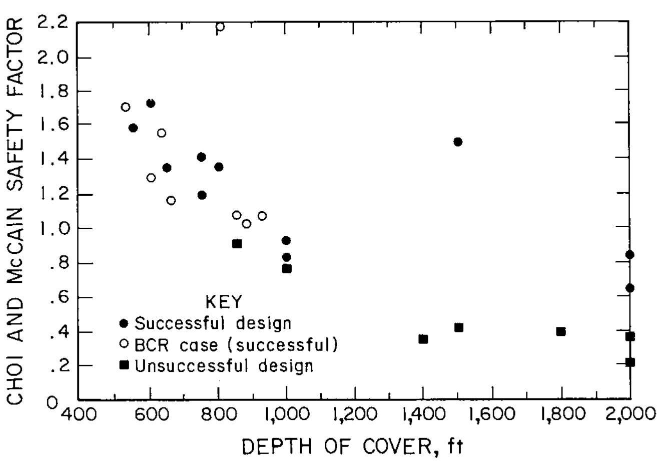

Figure 27.–Back-analysis of case histories using Choi and McCain's method.

HSUING AND PENG'S METHOD

Hsuing and Peng's method is unique in that it directly incorporates some properties of the roof and floor into the pillar design. The method was developed from numerical modeling described by Hsuing in his 1984 Ph.D. dissertation (*51*). In its finished form, it was first presented in 1985 (*52*) and it was later included in the second (1986) edition of Peng's book "Coal Mine Ground Control" (*91*).

In developing the method, Hsuing used three-dimensional finite-element models to represent pillars at various stages of longwall mining. The three-dimensional models allowed Hsuing to evaluate pillar stability at the critical headgate and tailgate T-junctions. Important design and rock mechanics parameters were varied within the models, and the model results were then analyzed using statistics. The final result is a simple equation that predicts pillar width as a function of seven geologic and geometric parameters.

Hsuing's models simulated three-entry longwall systems using equal-sized pillars. The roof, floor, coal, and gob materials within the models were assumed to be elastic, homogeneous, and isotropic, but yielding of individual elements could be simulated by reducing their elastic properties in later iteration steps.

The parameters varied by Hsuing in the models included overburden depth, pillar width, compressive strength of coal specimens, panel dimensions, stiffness of the roof and floor, and thickness of the main and immediate roofs. The large number of parameters required a large number of different models and runs. It also meant that some parameters had to be fixed in the analysis. For example, the height of the pillar elements was held constant throughout the study and was assumed to represent an 8-ft-thick seam. Also, the most important strength parameter, the angle of internal friction, was fixed at 37°.

As with the other conventional pillar design approaches, Hsuing's design criterion is pillar stability. Hsuing performed multivariate statistical analyses to relate the modeled geologic and geometric parameters to pillar performance. The results indicated that pillar stability increased as the stiffness of the roof and floor increased. Stiffer rock helps by providing more confinement to the pillars and by transferring load away from the pillars. Some other parameters, such as the thickness of the roof layers, were found to be insignificant.

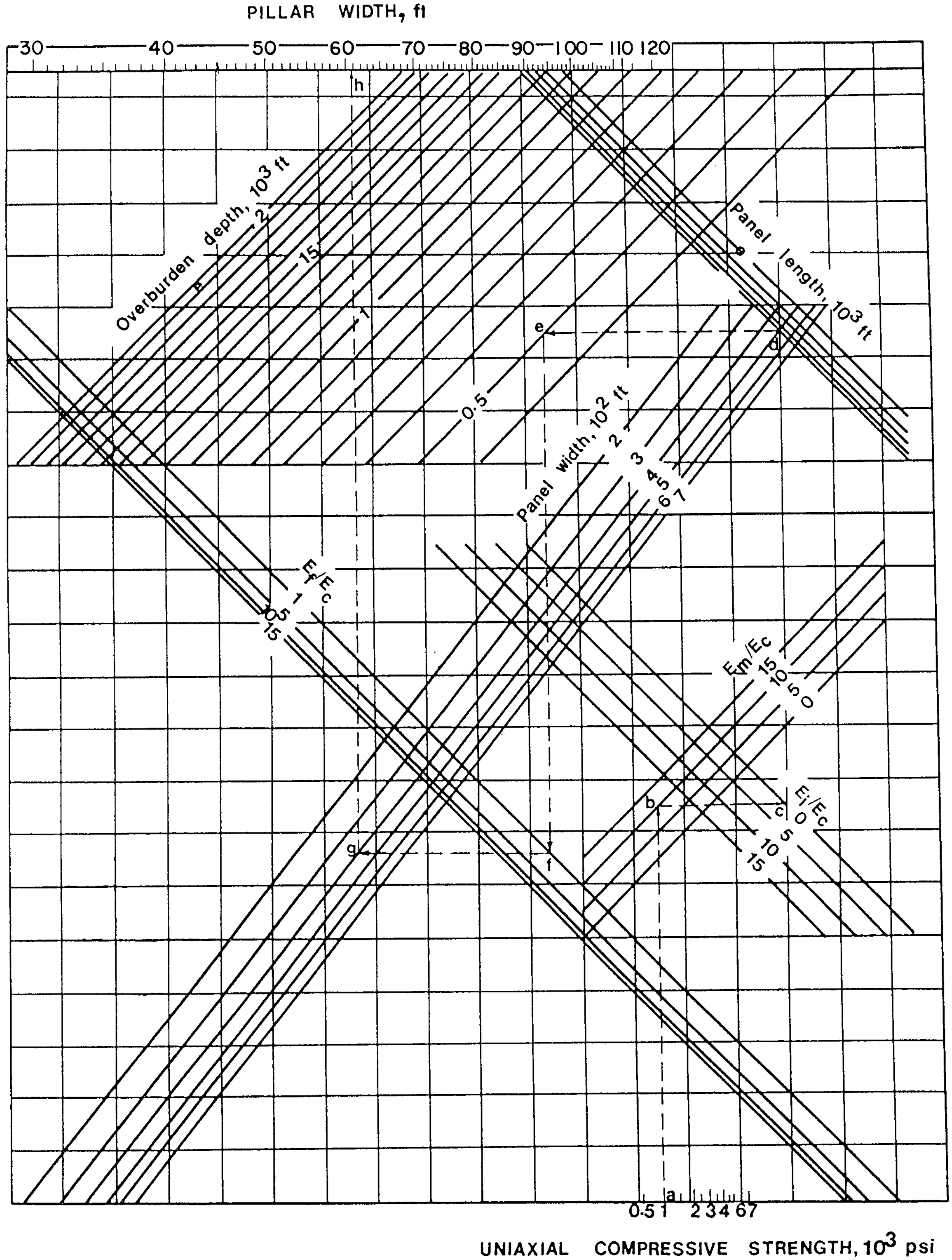

Figure 28.–Nomograph for determining the chain pillar size using Hsuing and Peng's method (after Hsuing and Peng (52)).

The final result of the modeling is equation 29, which predicts the required chain pillar width (w) for use in three-entry gate systems:

$$w = C_1 (E_i/E_c) + C_2 (E_m/E_c) + C_3 \log (E_f/E_c) + C_4 \log S_c + C_5 \log H + C_6 \log P_l/2 + C_7 \log P, \quad (29)$$

where E_c = elastic modulus of coal, psi,

E_f = elastic modulus of floor, psi,

E_i = elastic modulus of immediate roof, psi,

E_m = elastic modulus of main roof, psi,

S_c = compressive strength of laboratory coal specimen, psi,

H = depth of cover, ft,

P_l = longwall panel length, ft,

P = panel width, ft,

and where $C_1 = -4.676 \times 10^{-3}$,

$C_2 = -4.04 \times 10^{-3}$,

$C_3 = -3.33 \times 10^{-2}$,

$C_4 = -7.89 \times 10^{-2}$,

$C_5 = 0.5144$,

$C_6 = 4.94 \times 10^{-2}$,

and $C_7 = 0.1941$.

Hsuing and Peng provided a nomogram to aid in the use of their equation, which is reproduced in figure 28. They also provided a formula, based on other finite-modeling results, which they used to convert a rectangular pillar to a square pillar of equivalent strength:

$$w_p = w_r^{0.85}\, l_r^{0.15}, \quad (30)$$

where w_p = square pillar width, ft,

w_r = rectangular pillar width, ft,

and l_r = rectangular pillar length, ft.

The factors in Hsuing and Peng's equation (equation 29) that have the most effect on the pillar size are the depth of cover and the panel width. The three modulus ratios together can affect the required pillar size more than 20 pct. Hsuing and Peng suggest that for preliminary design purposes the lowest possible modulus ratios be used, or $E_f/E_c = 1$, $E_m/E_c = 0$, and $E_i/E_c = 0$.

Hsuing and Peng's equation was used to analyze all the case histories from designs using three-entry systems with equal-sized or nearly equal-sized pillars. A total of 23 cases were available from the 2 data sets. Fifteen designs were successes, but only two of these were located at depths exceeding 900 ft.

Actual test data on the stiffness of the roof and floor rock were seldom available, but modulus ratios could be assumed based on qualitative descriptions of the roof and floor. A strong roof or floor was assigned a modulus ratio of 10, very weak roof or floor was given a 1, with other values falling in between.

The pillar width predicted by the design formula was then divided by the actual pillar width reported in the case histories. Where necessary, equation 30 was used to adjust for pillar length.

The results are plotted in figure 29. The plot shows that Hsuing and Peng's design equation correctly predicted the failure of all eight of the unsuccessful cases. On the other hand, about two-thirds of the successful cases also used pillars smaller than predicted by the equation.

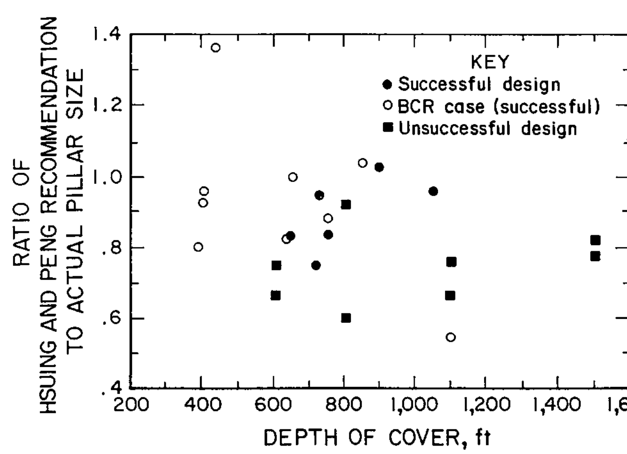

Figure 29.–Back-analysis of case histories using Hsuing and Peng's method.

COMPARISON OF LONGWALL PILLAR DESIGN METHODS

The preceding sections have described the several conventional longwall pillar design methods. Making a meaningful comparison between them is not a simple task, because each method makes different assumptions about gate entry geometry, pillar loading, and design criteria. It is therefore worthwhile to review some of these assumptions.

First, all the methods are *conventional* pillar design methods whose goal is to size pillars to carry the abutment loads. They all assume that stable gate pillar systems will result in stable gate entries. The experience at many mines, as documented in the case histories, has shown that this is generally a valid assumption.

The methods differ in the ways that they estimate the loads that are applied to the pillars. Three of the methods, Carr and Wilson's, Choi and McCain's, and ALPS, use empirical formulas to estimate the side abutment load. The only real differences are that they use three different values of the abutment angle ß, and only ALPS directly addresses the front abutment loads experienced at the headgate and tailgate face corners. Hsuing and Peng determined the pillar loading through three-dimensional numerical modeling, and their criteria are explicitly for tailgate loading conditions.

The methods fall into two broad categories regarding their approach to pillar strength estimation. Two of the methods, those of Carr and Wilson and Hsuing and Peng, use analytical approaches that place great emphasis on the frictional properties of the coal. Predictions using Carr and Wilson's method are extremely sensitive to the user's choice of friction angle. The frictional coal strength is fixed in Hsuing and Peng's equation. The other two methods, Choi and McCain's and ALPS, use empirical pillar strength formulas. The empirical formulas are sensitive to the unconfined compressive strength of the coal (S_1), but both ALPS and Choi and McCain suggest using fixed values of S_1.

The methods also assume different gate system geometries. Choi and McCain's method was developed for three-entry gates with a large abutment pillar next to a 32-ft-wide yield pillar. Hsuing and Peng's method is also for three-entry gates, but with equal-sized pillars. In addition, Hsuing and Peng's equation was derived assuming the height of the coal seam to be 8 ft. Both Carr and Wilson's method and ALPS can be used to analyze designs using any number of entries and any combination of pillar sizes and seam heights.

Finally, the methods differ in how they employ design criteria. The least flexible method in this regard is Hsuing and Peng's, because their design equation provides only a recommended pillar width. The opposite extreme is illustrated by Carr and Wilson's method, for which several design criteria have been suggested. ALPS and Choi and McCain's formulation employ the familiar stability or safety factor concept.

The characteristics of the four longwall pillar design methods are summarized in table 6.

Table 6.–Characteristics of longwall pillar design methods

Characteristic	ALPS	Carr and Wilson	Choi and McCain	Hsuing and Peng
Evaluation of tailgate pillar loading	x			x
Analytical pillar strength method		x		x
Empirical pillar strength method	x		x	
Variable entry configuration	x	x		
Variable coal height	x	x	x	
Roof and floor characteristics				x

To make a quantitative comparison between the methods, their predictions were compared for a base case longwall panel design problem. The base case was a longwall panel with a length of 5,000 ft and a width of 675 ft. The depth of cover and the seam thickness were varied in the analyses. When the depth was varied, the seam height was fixed at 6 ft; and when the seam height was varied, the depth was fixed at 1,000 ft. All entries were assumed to be 18 ft wide. The crosscut spacing was fixed at 120 ft, unless the predicted pillar width exceeded 102 ft, in which case the pillars were assumed to be square, with the crosscut spacing equal to the entry spacing.

Two different pillar configurations were analyzed. The first was a three-entry system with equal-sized pillars. The second was also a three-entry system, but with a single large abutment pillar and a 32-ft yield pillar.

The most conservative design criterion suggested for each method was used in the analyses. For ALPS and for Choi and McCain's method, the stability or safety factor was set at SF = 1.3. For Carr and Wilson's method, the LRS was used for the equal-sized pillar case, and SF = 1.4 was used for the abutment pillar design. The lowest possible stiffness ratios were used in Hsuing and Peng's method. In all cases the pillars were sized to be dual-use pillars subject to tailgate loading.

The results of the analyses are presented in figure 30. Figures 30*A* and 30*B* are for the equal-sized pillar base case and compare the predictions of Carr and Wilson's method, Hsuing and Peng's method, and ALPS. Figures 30*C* and 30*D* compare Carr and Wilson's method, ALPS, and Choi and McCain's method for the abutment pillar case.

The first conclusion is that all the design methods agree that the depth of cover has a great effect on the predicted pillar size (figs. 30*A*, 30*C*). The effect is most pronounced with Choi and McCain's method, because the empirical pillar strength formula it uses implies that the strength of very wide pillars approaches a constant value. Choi and McCain's formula indicates that for a fourfold increase in depth (from 500 to 2,000 ft), the required abutment pillar width increases more than six times (from 42 to 258 ft).

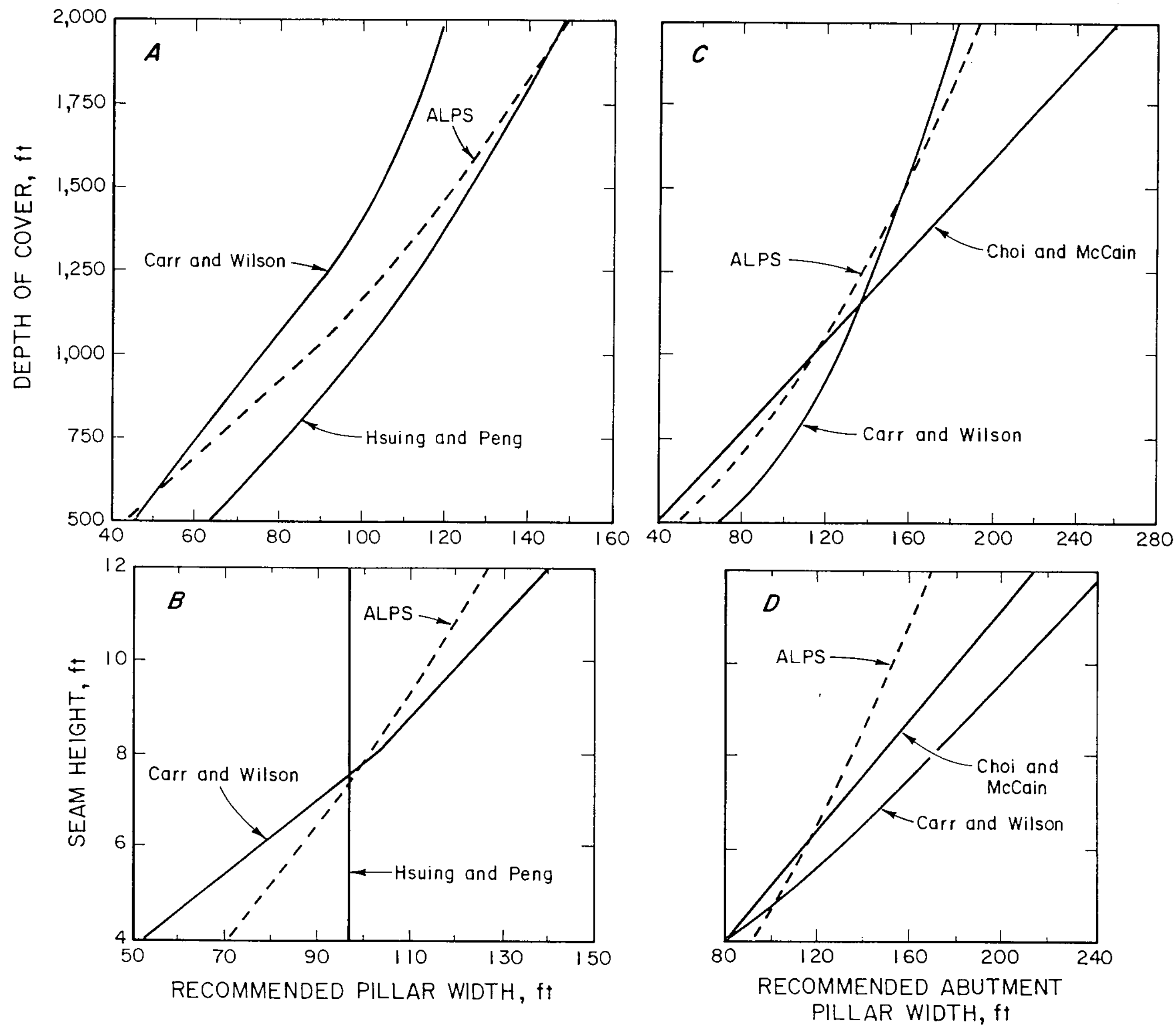

Figure 30.–Comparison of the longwall pillar design formulas. *A*, Three-entry system, equal-sized pillars, 6-ft seam thickness; *B*, three-entry system, equal-sized pillars, 1,000 ft of cover; *C*, three-entry system, abutment-yield design, 6-ft seam thickness; *D*, three entry system, abutment-yield design, 1,000 ft of cover.

For the same increase in cover, ALPS predicts that the pillar width must increase by a factor of 3.5, while the methods of Carr and Wilson and Hsuing and Peng predict increases of approximately 2.5 times. The least effect of depth is seen with the latter two methods because they employ analytical pillar strength formulas that presume that pillar strength increases rapidly with increasing width-to-height ratio.

Most of the methods also indicate that seam height is a critical parameter for longwall pillar design (figs. 30*B*, 30*D*). With Carr and Wilson's method, the required pillar size increases almost in direct proportion to the seam height. Again, this can be attributed to the sensitivity of Wilson's pillar strength formula to width-to-height ratio. Lesser effects are observed in the two methods that employ empirical pillar strength formulas.

Although not illustrated in the figures, the effects of the crosscut spacing and the panel width can be discussed. All of the methods imply that increasing the crosscut spacing improves stability. This is true because the available load-bearing area is increased for a given pillar width. In addition, rectangular pillars are generally stronger than square ones of the same least dimension, because they contain relatively more confined core. Wilson's pillar strength formula explicitly includes this effect. Crosscut spacing is not included as a parameter in Hsuing and Peng's method, but its effect could be considered using equation 30.

All of the formulas also imply that increasing the panel width increases the abutment load, and thereby the required pillar size. As panel widths grow larger relative to the depth of cover, however, the effect of further increases in panel width become less and less. In fact, all of the methods except Hsuing and Peng's incorporate the concept of a critical panel width, beyond which further increases have absolutely no effect. The effect of increasing panel width is most significant when the depth of cover is more than twice as great as the panel width.

Based on the analyses, some final observations can be made regarding the best use of the design methods. First, the methods consider many of the same basic design variables, and give generally reasonable predictions of the required pillar width when appropriate input parameters are used. Therefore, it is usually desirable to check the predictions of ALPS with one or several of the other methods. Each of the other methods has its own individual characteristics, which should be considered in the design process.

Apparently, Choi and McCain's method generally predicts the most conservative pillar size, and at very great depths of cover, it predicts widths that seem unrealistically large. This method therefore appears to be most appropriate for three-entry, yield-abutment designs at depths of less than 1,500 ft.

The biggest advantage of Hsuing and Peng's method is that it is currently the only one that may be used to evaluate the effect of roof and floor geology. Its most significant disadvantage is that it does not consider the effect of the seam height. The analysis indicates that it is best suited for three-entry designs using equal-sized pillars in seams 6 to 10 ft thick.

Carr and Wilson's method and ALPS are the only ones that can be used to analyze two-, four-, and five-entry systems. Carr and Wilson's method is very sensitive to a number of parameters that affect the pillar strength, but this can be overcome if a consistent set of values is used. A potentially more serious problem is that determining the most appropriate design criterion can be difficult. When the LRS was used for the equal-sized pillar design (fig. 30*A*), Carr and Wilson's method predicted the least conservative pillar widths. On the other hand, the method predicted the most conservative pillar widths when a stability factor of 1.4 was used with the abutment pillar design. Users should be cautious with this method until they have gained some experience with its use.

YIELD PILLARS AND LONGWALL MINING

The previous two sections described longwall pillar design methods that employ the *conventional* approach. The goal of these methods is to size pillars that are large enough to carry all the loads that develop during longwall mining. The conventional approach is usually quite effective in providing ground control, but it has one serious disadvantage. Longwall chain pillars are rarely extracted, and the amount of coal that is lost in them can be substantial. For a longwall under 2,000 ft of cover, conventional methods predict that pillars measuring up to 200 ft square would be required. A single row of such pillars in a 6-ft seam would contain more than 250,000 tons of coal per 6,000-ft panel.

An alternative to the conventional design approach is the use of very small yield pillars combined with extra entry support. The idea of yield pillars for ground control is not new; it was originally popularized in the United States by Holland (*48*) as part of the "pressure arch concept." In the pressure arch approach, the yield pillars are expected to redirect the overburden stresses to the solid abutments, thereby allowing greater extraction ratios within the panels. In longwall mining, yield pillars have been proposed for several purposes, including reducing floor heave (*72*, *80*), improving tailgate stability (*26*, *82*), eliminating pillar bumps (*39*), and reducing stress-related roof falls during development (*99*).

Yield pillars are thought to improve ground conditions in the following manner. When an entry is excavated, it creates a relatively small zone of stress-relieved ground surrounded by a zone of stress concentration, as shown in figure 31 (*1*). If the entry can be made wider, the zone of stress relief expands. In coal mines, entries cannot be made too wide because tensile failure at midspan may develop (*7*, *27*), and because mining law limits entry widths to 20 ft unless a combination roof control plan is used. Yield pillars are a means of simulating a wide opening without excessive spans.

Unfortunately, effective application of the yield pillar concept is a fairly complex rock mechanics problem. While some successful uses of yield pillars have been well documented, other applications remain little more than theoretical possibilities. There has even been considerable confusion over the definition of the term "yield pillar." In some cases, conventional longwall pillar designs using large abutment pillars in combination with smaller pillars have been called yield pillar designs, with the smaller pillars designated as "yield pillars." This definition does not require that any pillars actually yield. A more meaningful definition, and the one used here, states that a pillar whose load exceeds its load-bearing capacity *upon development* is a yield pillar. A yielding longwall pillar design is one that consists *only* of yield pillars. A design that includes both yield and abutment pillars is still a conventional pillar design.

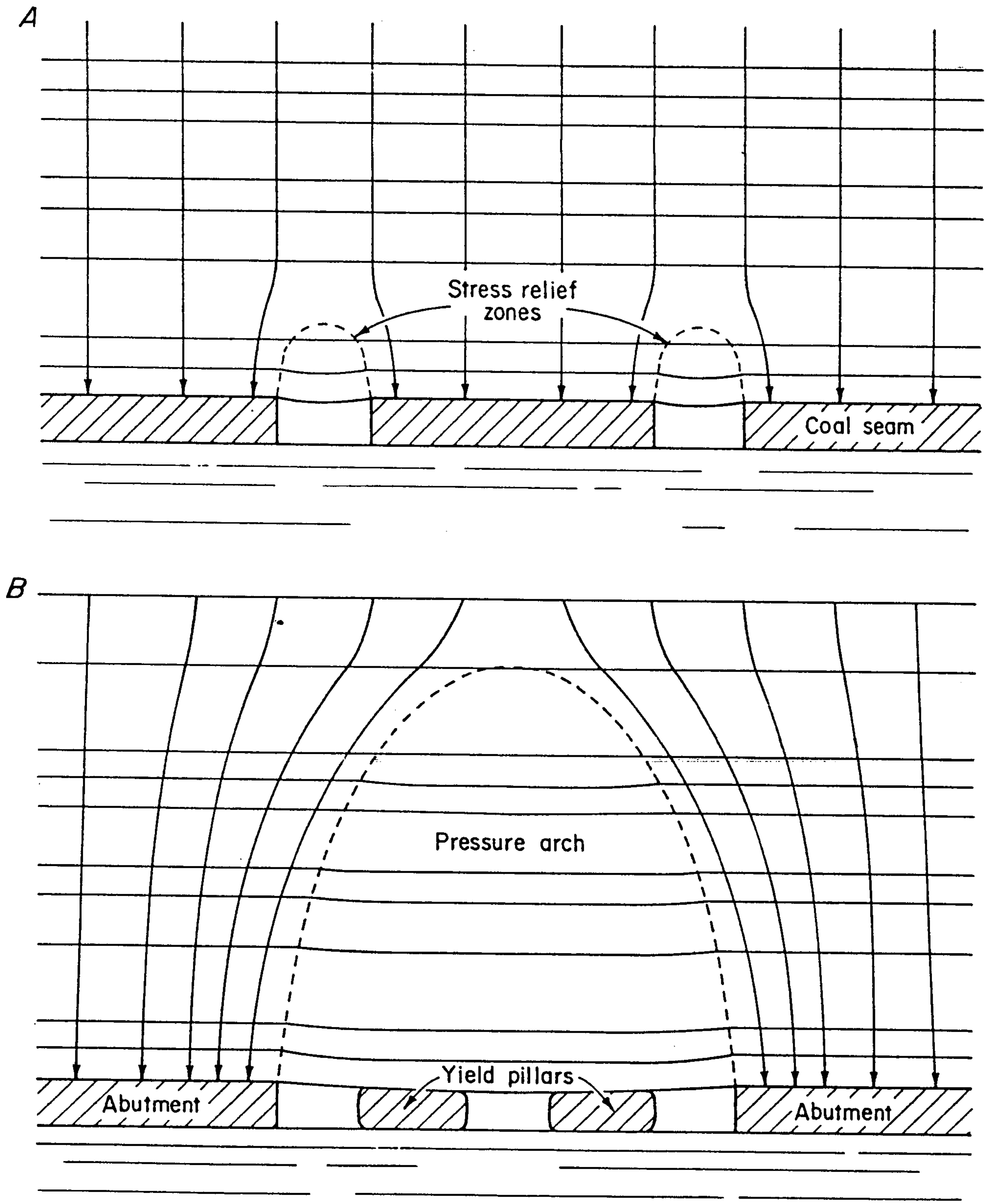

Figure 31.–Redistribution of ground stresses due to entry development. *A*, Minimal stress relief obtained with nonyielding pillars; *B*, large pressure arch created through use of yield pillars (after Adler (*1*)).

YIELD PILLAR THEORY

The yield pillar approach rests on three fundamental assumptions. First, the yield pillars must deform enough that the main roof bridges them, transferring some load and creating a pressure arch. Second, there must be solid abutments nearby to receive the load that is shed by the yield pillars. Finally, the yield pillars must fail in a nonviolent manner and maintain enough residual strength to support the weight of the rock within the pressure arch.

The load transfer required by the yield pillar concept depends upon the yield pillars deforming considerably more than the nearby abutments. If the pillars were perfectly rigid and did not deform at all, then no load transfer would occur, and the load would be the original tributary area load. Elastic theory indicates that some load transfer occurs from narrow (small width-to-height ratio) pillars even if they do not yield, simply because they are more free to expand laterally than are squat (large width-to-height ratio) pillars. Far greater load transfer can occur when a pillar yields, because a yielded pillar will continue to deform until the applied load equals its residual load-bearing capacity.

These concepts are illustrated using the ground reaction curve shown in figure 32. The load applied to a rigid pillar is represented by point A. An elastic pillar deforms by amount b, and carries load B. The yield pillar deforms much more and carries its residual load C. If no pillar were present and the pressure arch remained stable, then the pillar load would be zero and the sag of the arch would be d. The total load transfer to the abutments is zero for the rigid pillar, A minus B for the elastic pillar, A minus C for the yield pillar, and A for the no-pillar case.

The actual mechanism of load transfer is that stiff units in the upper roof deflect less than the yield pillars, thus bridging from abutment to abutment. The maximum width of a yield pillar system is therefore limited to the span the upper roof can effectively bridge. If the span is too great, the central yield pillars may be too highly loaded and the whole system can collapse.

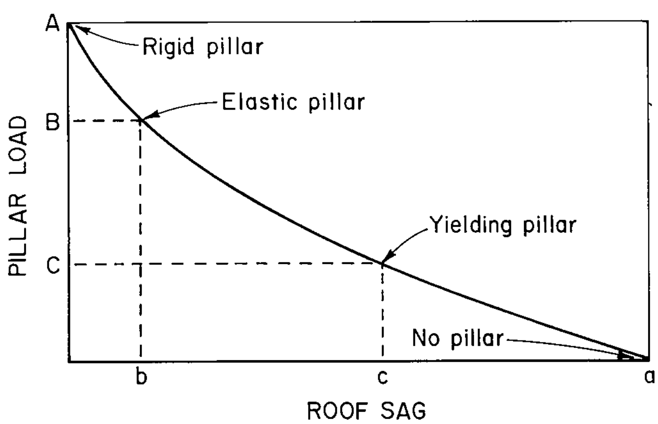

Figure 32.–Ground reaction curve showing trends in roof deformation and pillar load for decreasing pillar stiffness.

The performance of a yield pillar system evidently depends upon the presence of nearby strong abutments. The abutments may be either large pillars or unmined longwall panels. If the abutments are too small, or if they are weakened or removed, the yield pillars may be reloaded, possibly leading to the collapse of the pressure arch (*7*). Weight on the abutments is increased by the loads transferred from the yield pillars. In multientry yield pillar systems, the central entries that are flanked by yield pillars are expected to be the most stable (*76, 80, 98*). This characteristic of the yield pillar technique may reduce its power for longwall applications, since the headgate and tailgate entries are adjacent to the longwall panels that act as the abutments in a yielding longwall pillar design.

The concept that pillars can fail nonviolently and maintain a degree of resistance even after failure represented an important advance in rock mechanics thinking. Laboratory tests established that violent failure occurs only when the loading system is less stiff than the specimen and when the specimen-platen interfaces supply enough confinement (*8, 12*). In the United States, coal bumps, or violent pillar failures, seem to occur in retreat mining areas under deep cover where the roof and floor rocks are both massive and strong (*15*). In the Northern Appalachian and Illinois coal basins, which are characterized by softer roof and much softer floor, serious bumps have been very rare (*37*).

The use of yield pillars to improve entry stability can properly be called a stress control technique. The concept of stress control methods has been popularized recently (*7, 98*), based primarily on experiences in Canadian potash mines. There are several stress control techniques, but all attempt to modify the stress field around the entries by adjusting the room width, the pillar or entry geometry, or the excavation sequence. In coal mining, the use of rib slotting, roof slotting, or caving entries to relieve high horizontal stresses causing roof failure are stress control methods (*63*).

According to Serata (*98*), incorrect sizing of yield pillars could result in worsened entry conditions. Based on studies in potash mines, Serata indicates that between yield and abutment pillar sizes there may be an intermediate pillar size that is too stiff to yield but too small to effectively redistribute stresses within itself. Such "critical" pillars maximize disturbance of the surrounding ground. There have been no studies of the critical pillar phenomenon in U.S. coal, but studies in British coal mines have indicated that closure rates of gate roadways may be greatest when rib pillars are between 15 and 90 ft in width (*106*). Figure 33 shows that while better conditions were more common when very large conventional pillars were used, yield pillars were at least as successful as undersized conventional pillars in preventing gate road closure.

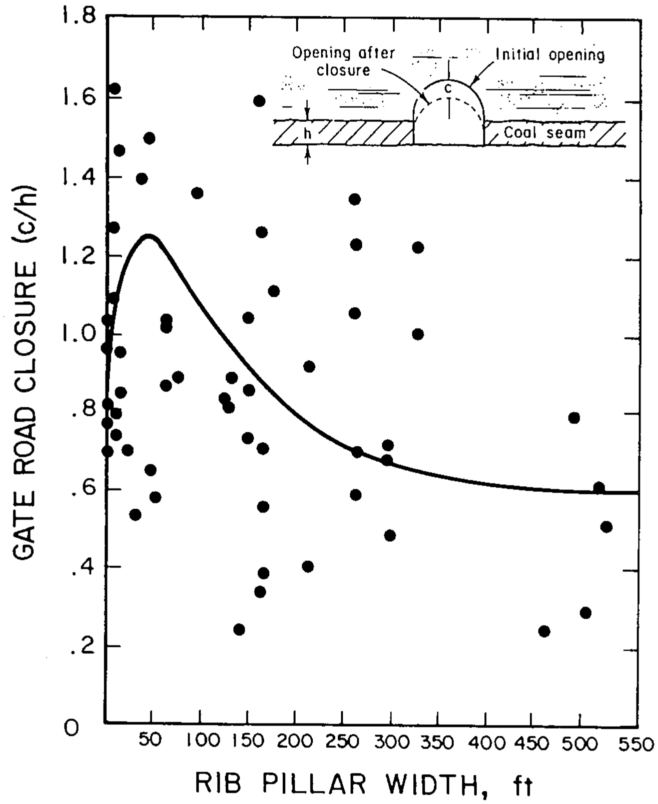

Figure 33.–Measured vertical gate road closure in British longwall mines, for depths in excess of 1,000 ft (after Whittaker and Singh (*106*)).

EXPERIENCE WITH YIELD PILLARS IN LONGWALL MINING

Yield pillars have been used in many of the two-entry longwall pillar systems employed by some western U.S. mines. At most of these mines, the yield pillar systems evolved as a means of controlling severe bump problems (*39*). Recently, several ground control studies conducted in western mines (*28*, *62*, *66*, *70*, *72*, *73*) have added considerably to the understanding of the use of yield pillars for ground control.

In one Bureau study, the performance of a three-entry, conventional pillar design was compared with that of a two-entry, yielding design at a western longwall (*28*, *66*). The study site is shown in figure 34. The conventional pillars were only 50 ft wide, resulting in an ALPS stability factor of 0.3. Measurements taken as longwall mining progressed showed that the conventional pillars became highly stressed, resulting in floor heave, roof sag, and roof falls. It seems that the 50-ft pillars were too small to carry the abutment loads, but too large to function effectively as yield pillars. In the two-entry area, the stresses measured in the 20-ft yield pillars were considerably reduced, and conditions in the adjacent tailgate entry were also improved. The results of this study give credence to the argument that a well-designed yield pillar system can outperform undersized conventional pillars. It also seems likely that the 50-ft pillars may have been critical pillars at this site.

One lesson of the western experience with yielding pillar systems is that extensive supplemental support is usually required to maintain stability in the tailgate entries. At the study site described above, yielding steel arches on 5-ft centers were used in the tailgate. At another nearby mine, tailgate support consists of two rows of 36-in-long, three-layer wood cribs on 5-ft centers, with metal straps and wire mesh.

Unfortunately, the western experience with yield pillars may not be universally applicable, because eastern mines typically have softer roof and floor rocks and are found at shallower depths. In addition, many eastern mines must use multiple-entry gates because of high methane inflows (*93*). At the present time, only a handful of eastern longwall mines employ true yield pillars, and even fewer have experimented with total yielding designs.

The most extensive use of yield pillars in the Eastern United States began at Jim Walter Resources (JWR) in 1982 (*44*). JWR operates four Alabama coal mines at relatively deep cover, averaging 1,500 to 2,200 ft. JWR first introduced yield pillars in a room-and-pillar section that was experiencing severe floor heave (*80*). Pillar sizes were reduced from 100 to 20 ft in a three-entry test section, and floor heave was greatly reduced.

This success encouraged JWR to experiment with systems of one, two, and three yield pillars. Ground conditions were satisfactory in all but the four-entry yield pillar system (*16*), perhaps indicating that four entries approached the maximum width for a yield pillar system at that mine. JWR also found that ground control problems could develop in the transition zones between conventional and yield pillar sections. These transition zones receive some portion of the load that is transferred away from the yield pillar area and need to be carefully designed and supported.

JWR's next step was to investigate the use of yield pillars in longwall pillar design. JWR had experienced severe tailgate stability problems in its early longwall panels, which had been attributed to insufficient load-bearing capacity of the three rows of 85-ft-wide pillars used in the gates (*19*). Analysis indicated that conditions would be improved if the 85-ft pillars could be replaced by a single row of 180-ft pillars. An experimental two-entry development confirmed the ground control advantages of the large pillars, but large pillars proved extremely difficult to develop in the gassy Blue Creek Seam (*16*, *100*). Yield pillars provided a means to develop the large pillars. By placing 20-ft-wide yield pillars on either side of the abutment pillars, and by driving crosscuts through the yield pillars on 100-ft centers, the 180-ft-square abutment pillars could be developed without excessive lengths of face ventilation curtain. In Carr and Martin's (*17*) words, the yield pillars functioned primarily as "inexpensive ventilation partitions." JWR has since made the yield-abutment-yield

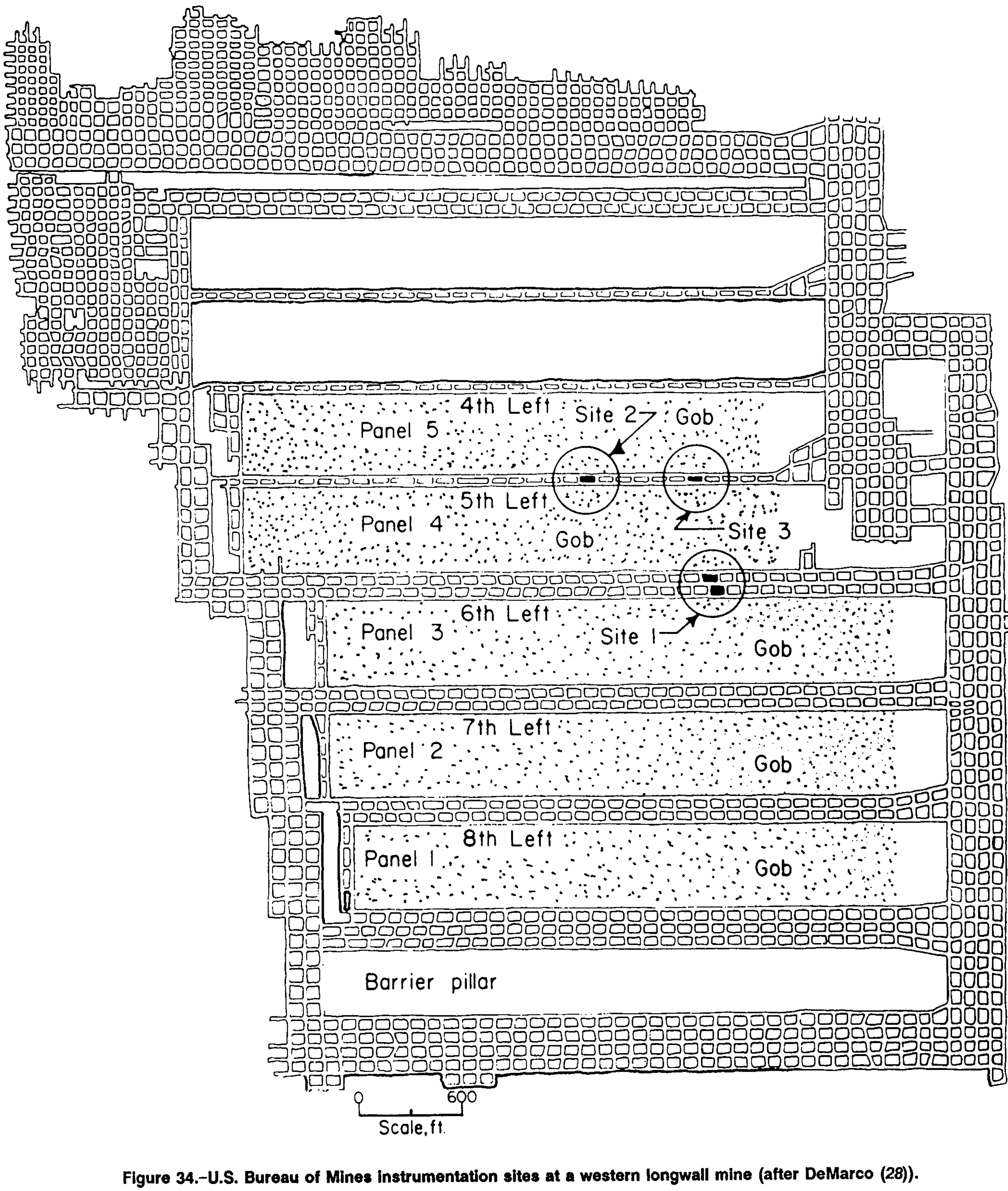

Figure 34.–U.S. Bureau of Mines instrumentation sites at a western longwall mine (after DeMarco (28)).

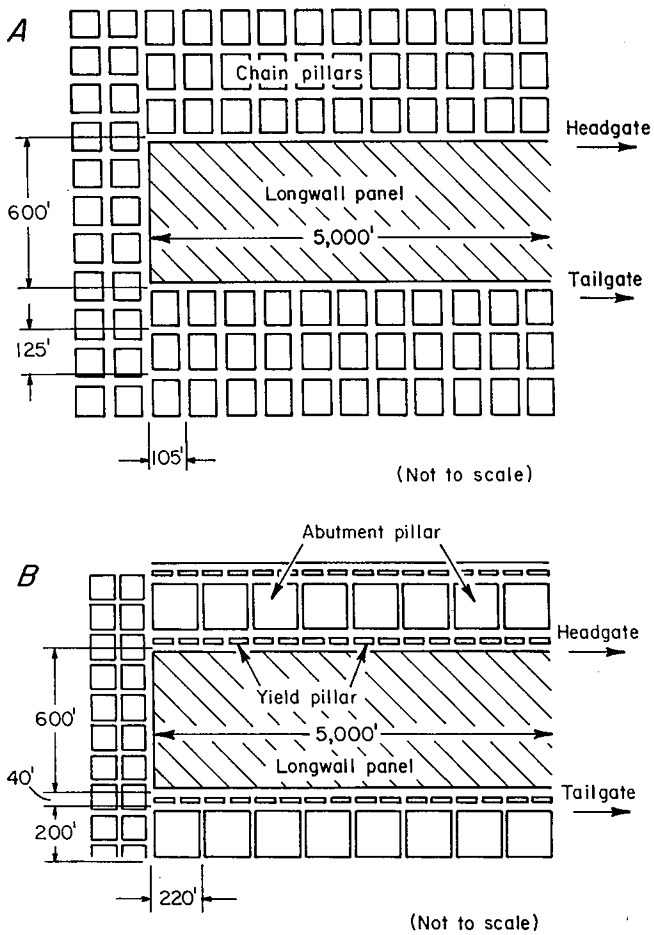

Figure 35.–Evolution of longwall pillar designs at Jim Walter Resources (after Carr (*18*)). *A*, Original design with equal-sized pillars; *B*, improved yield-abutment-yield design.

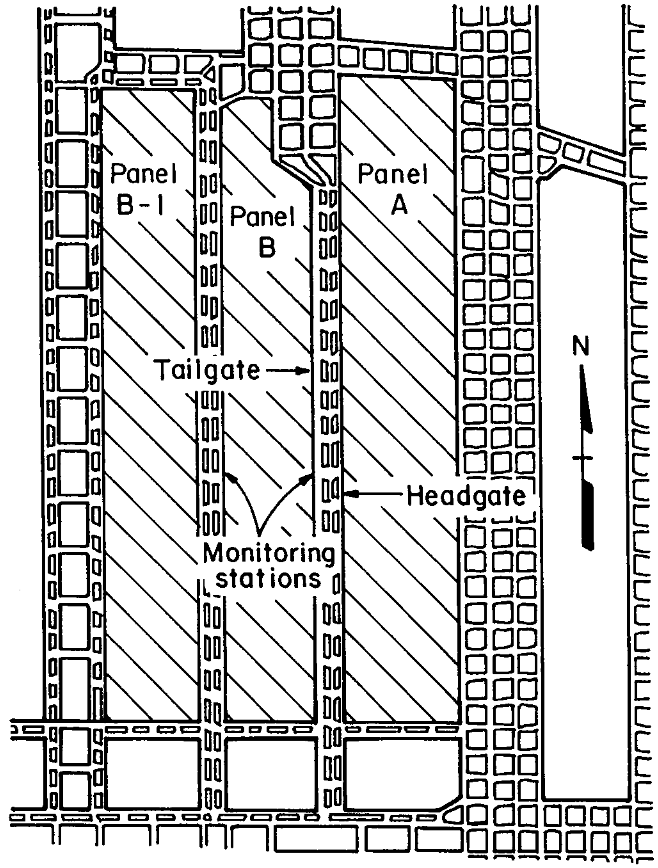

Figure 36.–Total yield pillar test panel development at Jim Walter Resources (after Martin and Carr (*81*)).

(Y-A-Y) system, with one abutment pillar flanked by two yield pillars (fig. 35), standard on its longwalls (*16*). A very similar design has also been adopted in the deep-cover longwalls operated by Island Creek Coal Co. in Virginia (*35*, *42*).

Most recently, JWR developed three longwall panels using a three-entry, total yield pillar design (fig. 36). Monitoring showed that the entries were stable during the extraction of the first panel, but closures of up to 10 in occurred in the future tailgate entry (*34*). In anticipation that reloading of the yield pillars would occur when the second panel was mined, extra supplemental support consisting of a double row of three-layer wood cribs and wire mesh was installed in the future tailgate. Extraction of the second panel was completed without adverse effects on strata control (*81*). In fact, it was reported that both the cribbed tailgate and the center entry remained open for several hundred feet between the longwall gobs. Ventilation between the gobs was hindered, however, discouraging JWR from further use of total yield pillar designs. Nevertheless, this project demonstrated that multientry, total yielding pillar designs could be successfully used on deep-cover longwalls.

A second total yielding pillar system, this one at a Kentucky longwall, was the subject of a Bureau study (*76*). At this site a 400-ft length of a five-entry, all-yield system was developed between three- and four-entry conventional pillar designs (fig. 37). Analysis with ALPS indicated that even the conventional pillars in the three-entry system were undersized for the 1,800 ft of cover present at the site, with a stability factor of 0.45. As the longwall was mined past the test site, entry convergence, roof sag, and changes in roof quality were recorded.

The study found that while mining conditions were adequate for all three designs during first panel mining, none of the designs would have provided acceptable tailgate stability without considerable artificial support. The four-entry system in particular suffered severe floor heave and roof falls along its entire length. The ALPS stability factor for four-entry design was 0.25, indicating that perhaps the 60-ft pillars used were critical pillars. Severe damage was also encountered within the transition zones located adjacent to the all-yield section, again emphasizing the importance of careful design of transition areas.

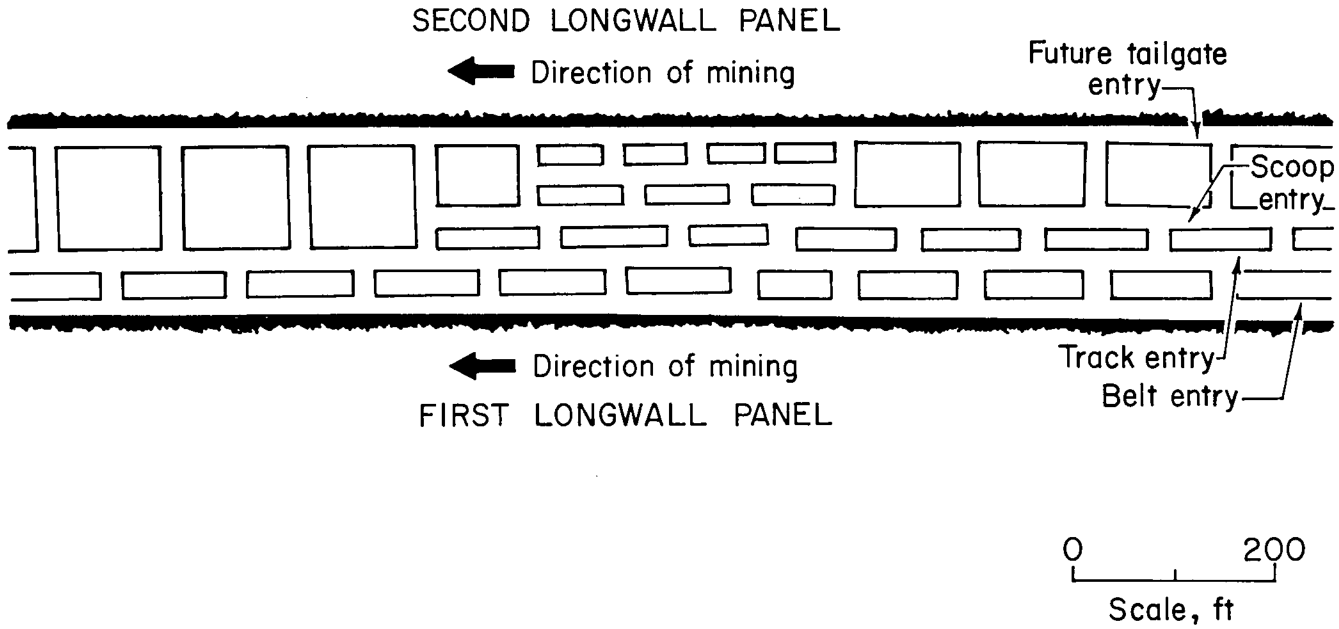

Figure 37.–Pillar designs studied at a deep-cover Kentucky longwall (after Mark and Barton (*76*)).

The total yield pillar system performed nearly as well as the three-entry conventional system. Although serious floor heave and roof sag were measured in the tailgate portion of the yield pillar area, very little roof degradation occurred and the second face was mined past the area with little difficulty. The key to the success of both the total yield and the three-entry systems was the heavy secondary support, consisting of two rows of 4- by 4-ft cribs, installed in the tailgate.

A final yield pillar trial in an eastern longwall was conducted at Beth Energy Mines, Inc., Eighty-Four Complex in Pennsylvania. Conditions of high horizontal stress led Beth Energy to try 8-ft yield pillars to control cutter roof that was occurring as the gate entries were being developed (*86*). The yield pillars were located in a 6.5-ft seam under 525 ft of cover. Mixed results were achieved in improving ground control on development.

DESIGN OF YIELD PILLARS

The design of yield pillars should be based on quantitative guidelines using site-specific information about pillar strength and strata loading. There are, however, no true quantitative guidelines for yield pillar design (*72*). One semiquantitative approach that has been used is the rough guidelines presented by Holland (*48*). These were developed through experience and research conducted in northern England during the 1940's and are primarily suited to the pressure arch design of room-and-pillar workings. The experience at JWR led Gauna (*36*) to conclude that Holland's guidelines oversize yield pillars for longwall applications.

Choi and McCain (*22*) proposed that small pillars used with large abutment pillars could be "sized to yield before the roof or floor break." These pillars are not required to yield on development. The key to ground control with conventional pillar designs that use small or yield pillars, such as Choi and McCain's approach or the Y-A-Y designs used at JWR, is the load-bearing capacity of the abutment pillar. The size and location of the small pillar or pillars appear to be a secondary issue. By designing the small pillars to be weaker than either the roof or floor, the problems caused by critical pillars that concentrate excessive stresses are avoided.

Total yield pillar systems, which contain no abutment pillars, are more difficult to design successfully. *Oversized* yield pillars that concentrate stresses can be as hazardous as undersized pillars that fail to support the overburden within the pressure arch. In addition, yield pillar systems do not isolate the tailgate from the abutment loads transferred from the previous panel. Therefore, yield pillars should be used only when the immediate roof is competent, and it is usually necessary to provide heavy supplemental support in the tailgate entry.

At present, the most common way of sizing yield pillars is by experiment. For example, JWR experimented with 20- and 25-ft pillars in the longwall test sections shown in figure 38. Stress measurements indicated that the 20-ft pillars yielded on development, while the 25-ft pillars later showed some increase in load as the longwall face approached (*89*). Other measurements indicated that both pillar sizes expanded laterally by the same amount, about 12 in. Based on the tests, JWR made 20 ft its standard yield pillar width.

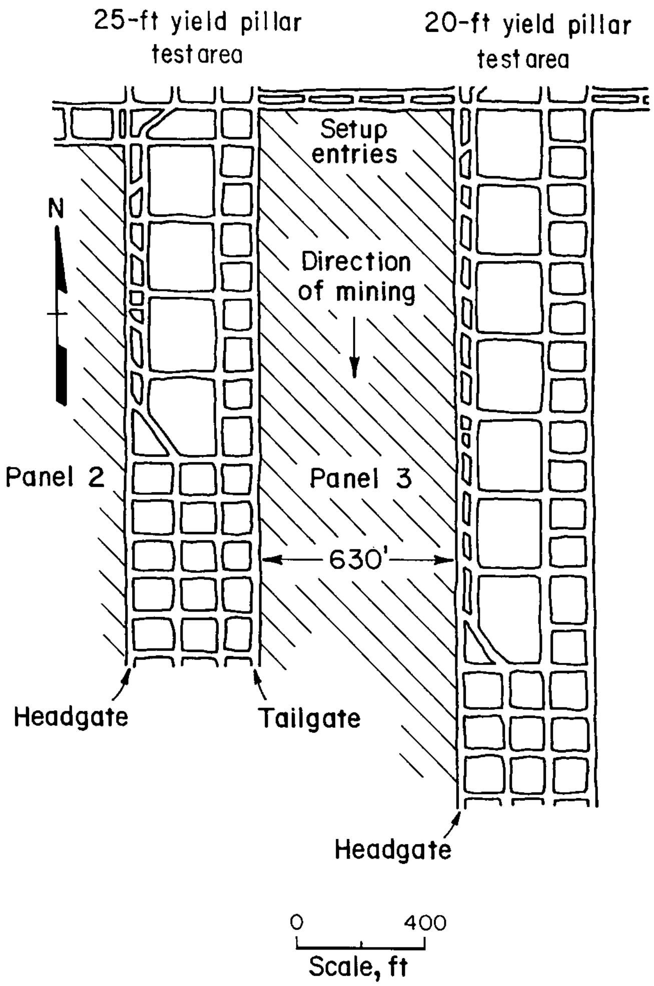

Figure 38.–Yield pillar test areas at Jim Walter Resources (after Gauna (36)).

Malecki (*72*) described a yield pillar test that was performed in the setup rooms of the Plateau Mine in Utah at a depth of 1,500 ft. The test yield pillar was 27 ft wide, and the seam was 9 ft high. Over 10 ft of fracturing was observed in the ribs of the yield pillar, as compared with 3 ft in the adjacent conventional pillars. Based on the study, yield pillars of 20 to 30 ft were considered appropriate for the mine, and a 30-ft pillar was actually used in the gate roads so as to limit spalling. Later, stress measurements indicated that the 30-ft pillars received very little additional load during longwall panel extraction, indicating that they did indeed yield on development (*28, 73*).

Stress measurements are also available from two of the 8-ft-wide yield pillars that were tested at Beth Energy's Eighty-Four Complex (*69*). While local stress increases of up to 1,000 psi were measured in the core of these pillars as the longwall approached, the average pillar stress increase was only 300 psi in one case and 100 psi in the other. It therefore appears that the 8-ft pillars were close to yielding on development.

Analysis of these three case histories can provide some guidance to other mines that would like to size yield pillars. Table 7 contains the relevant design parameters from the cases. The calculated stability factors range between 0.34 and 0.44, indicating that SF = 0.5 might be used as an upper limit for yield pillar design.

Two factors may explain why the stability factors calculated for the yield pillars are so much less than 1.0, when by definition any pillar with a stability factor less than 1.0 would be expected to fail. The first is the pillar geometry. The empirical pillar strength formulas assume the pillars are square, while the test yield pillars were all very long rectangles. Rectangular pillars are considerably stronger than square ones because they contain a greater area of confined core. Hsuing and Peng's formula (equation 30) indicates that a pillar whose length was 10 times its width would be approximately 40 pct stronger than a square pillar of the same width.

The second factor is that the actual pillar loadings were probably considerably less than the tributary area estimates. As discussed in the section "Longwall Pillar Loads," the tributary area approximation becomes less and less accurate as the extraction ratio increases and as the panel width diminishes. For the Beth Energy case history, where the local extraction ratio was approximately 75 pct, elastic analysis indicates that the actual loading may have been little more than half of the value calculated using tributary area theory (*96*).

Table 7.–Yield pillar tests

Test Site	Depth of cover (H), ft	Coal seam height (h), ft	Yield pillar width (w_y), ft	Estimated pillar strength (S_p), psi	Calculated SF
Beth Energy	600	6.5	8	975	0.35
Jim Walter Resources	1,500	6.0	20	1,655	.42
Plateau Mining	1,500	9.0	25	1,550	.44

SF Stability factor.

In summary, some preliminary formulas for selecting the width of yield pillars (w_y) are as follows. The test cases in table 7 indicate that an *upper bound* design criterion can be obtained by combining the tributary area expression for the pillar loading (equation 1) with the Bieniawski equation for pillar strength (equation 11), and using 0.5 as the maximum allowable stability factor:

$$0.5 > \frac{S_1 \; [0.64 + 0.36 \, (w_y / h)]}{\gamma \, H \left[\dfrac{(w_y + w_e) \, (l_p + w_e)}{(w_y) \, (l_p)} \right]} . \tag{31}$$

The *lower bound* design criterion for yield pillars is determined by the necessity that they maintain enough load-bearing capacity to support the overburden within the pressure arch. Therefore, yield pillars should maintain an effective width-to-height ratio of at least unity, because more slender pillars could fracture and lose all load-bearing capacity. It should also be considered that mine entries are often off centers by as much as 2 ft, so the suggested minimum yield pillar size is the seam height plus 4 ft:

$$w_y > h + 4. \tag{32}$$

In shallow mines or mines with thick seams, it may not be possible to satisfy both the criteria in equations 31 and 32. It should also be noted that equation 31 must be solved iteratively for the yield pillar width.

Two other important aspects of yield longwall pillar design are the span of the system and the selection of artificial support. For a yield pillar design to be successful, the span between the abutments must be small enough that a pressure arch can be maintained. In general, the number of entries in a yield pillar system should be minimized. The preliminary experience at JWR indicates that a three-entry yield pillar system with a total span of 80 ft may perform adequately, but that wider spans may be less stable. Studies of multiple-seam mining have found that stiffer, more competent roof strata are more able to support an arch (*41*).

Finally, experience has shown that total yield pillar systems usually require considerably more artificial support than do adequately sized conventional pillar systems. In fact, the decision to adopt the yield pillar approach may be largely economic, balancing the savings associated with smaller pillars against the costs of additional support. It is also important to select the proper *type* of artificial support. Yield pillar systems can be expected to undergo considerable deformation, and so stiff supports such as fiber-reinforced concrete cribs are probably not appropriate (*9*). Closely spaced wooden cribs are often a good choice when large deformations are anticipated, because they can withstand up to 40 pct vertical closure without losing their support capacity. Yielding steel supports, either arches or posts, are expensive but combine very high support loads with excellent deformabilities.

PILLAR DESIGN STRATEGIES FOR SPECIAL CONDITIONS

The preceding sections have provided general guidelines for longwall pillar design. For the most part these guidelines are based on evaluations of pillar strength and pillar loading, which in turn are determined primarily from the seam height, depth of cover, and longwall panel width. The case histories showed that in many instances improved design can be achieved using these methods alone.

Each longwall panel is unique, however, and there are a number of additional, special conditions that may have to be considered in the development of a fully optimized longwall design. Some are geologic conditions, including the roof rock quality, the floor strength, the in situ horizontal stress field, and the potential for coal bumps. Other conditions are mining induced, such as multiple-seam interactions. A third group of factors may affect stability *requirements*, such as a need to control the severity of subsidence or to maintain stable ventilation airways.

The discussion that follows will briefly describe each condition and give suggestions as to how it might be factored into longwall pillar design.

ROOF ROCK QUALITY

The initial quality of the roof rock seems to have a large effect on the ultimate stability of gate entries (*19*, *106*). The statistical analysis of the BCR data (*2*), described in the section "Verification of the ALPS Method," supports the observation that mines with more competent roof may successfully use pillar designs with lower stability factors, while less competent roof requires more conservative pillar design.

A geologic evaluation can provide a qualitative assessment of the structural competence of mine roof. A valuable summary of the factors that should be considered in such an evaluation was recently presented by Moebs and Stateham (*84*). They identified four key factors that affect roof rock quality:

- Rock strength,
- Bedding planes,
- Minor structures,
- Moisture sensitivity.

Rock strength is defined as the strength of intact rock *material*, determined on laboratory-size specimens. Moebs and Stateham found that problem roof is associated with rocks whose compressive strength is less than 2,500 psi. Such rocks include weak shales, underclays, and some claystones. The strongest roof rocks found in coal mines are typically massive sandstones or limestones, with compressive strengths exceeding 15,000 psi.

Rock strength is by no means the most important factor affecting the structural competence of mine roof, however. Natural weakness planes, or discontinuities, can dramatically reduce the integrity of roof consisting of even very strong rock material. Bedding planes are the most common type of discontinuity found in sedimentary rocks. Closely spaced bedding planes, or laminations, are most likely to be found in shales, but thinly laminated sandstone ("stackrock") also occurs. When the bedding planes are closely spaced and the bonding across them is weak so that they separate easily, roof quality can be expected to be poor. Conversely, massive rocks with few bedding planes normally provide competent roof.

Minor structures include such features as sandstone channels, clay dikes, slickensides, slumps, rolls, slips, and small faults. These features disrupt the normal beamlike structure of the roof. Where they are present they can cause problems even in otherwise competent ground.

The initial strength of many shales and claystones can be dramatically reduced when they come in contact with the humid mine environment. Fortunately, the detrimental effects may take several years to develop, so gate roads that are used and abandoned relatively quickly may not be affected. Moisture sensitivity can be more of a problem in long-term entries like mains or bleeders.

If a geological evaluation indicates that the roof in planned gate entries is of poor geotechnical quality, several steps may be taken. The first is to use a conservative pillar design (ALPS stability factor near 1.3) to reduce the transferred abutment loading to a minimum. A total yield pillar design might be ruled out, because the roof might not be able to withstand the deformations associated with such a design. Where possible, entry widths should be minimized. Additional artificial support may also be required, so that the spacing between supports is kept low.

FLOOR STRENGTH

Excessive floor heave in the gate entries poses a major threat to longwall operation. Floor heave can impede travel, obstruct airflow, destroy roof supports, and ultimately result in roof falls. It can also seriously impair equipment performance in the T-junction areas. Many longwall ground control failures, including many of the unsuccessful designs reported in the section "Verification of the ALPS Method," can be largely attributed to floor heave.

Floor failure occurs when the stresses applied to the floor exceed its bearing capacity. Underclays, which are highly fractured claystones containing many slickensides and fossilized root casts, typically have low bearing capacities. The bearing capacity of underclays is further reduced when the moisture content is high, when a large percentage of swelling clay minerals is present, or when the underclay layer is thick (*23*, *95*).

Since there is seldom a cost-effective method for increasing the bearing capacity of mine floor, the usual solution to the problem of excessive heave is to reduce the applied stress. There are two ways in which pillar design can be used to do this. The first approach is to use larger pillars, which distribute the load better and thereby reduce the stress. The second approach is to use yield pillars, limiting the applied stress to the residual strength of the pillars. Two recent studies, one in a longwall area (*76*) and the other in a room-and-pillar area (*80*), found that floor heave is minimal in the center entries of a total yield pillar section. These studies also found that more than 1 ft of heave occurred in the outside entries, due to load transfer from the yield pillars to the adjacent barrier pillar. In both cases, however, the heave in the outside entry was less than occurred in nearby areas where larger, nonyielding pillars were used.

A numerical model study reported by Hsuing and Peng (*53*) indicates that floor heave may best be controlled by combining yield pillars with a large abutment pillar. Their results show that a yield-abutment-yield pillar system might reduce floor heave in the future tailgate entry by as much as 50 pct compared with a conventional design using equal-sized pillars.

IN SITU HORIZONTAL STRESSES

The basic purpose of longwall pillar design is to control the vertical stresses that result from the weight of the overburden. Horizontal ground stresses are often present as well, and when they are excessive they may affect roof stability even where the pillar size appears to be adequate. Recent studies have found that in many cases the magnitude of in situ horizontal stresses can exceed the vertical stress, sometimes by a factor of 3 or more (*47*). The cutter roof that occurs in many coal mines has often been largely attributed to the presence of high horizontal stresses (*13*, *45*, *63*).

High horizontal stresses and cutter roof can severely impact the functioning of a longwall. Cutter roof usually occurs when entries are first mined, and if it is not controlled, gate entry development can be slowed severely. Even when falls do not result immediately, the competence of the roof can be so reduced by cutters that longwall front abutment stresses result in collapse. Finally, there is growing evidence that under some circumstances the process of longwall mining itself may concentrate horizontal stresses, resulting in gate entry stability problems, particularly in the headgate (*33*, *74*). A general rule of thumb is that if ground conditions are consistently worse in the headgate than in the tailgate, then excessive horizontal stresses are probably involved.

A distinguishing characteristic of cutter-roof-type failures is that they tend to occur in a directional pattern. Stress measurement programs conducted in the northern Appalachians (*45*) and in the Illinois Basin (*57*) have found

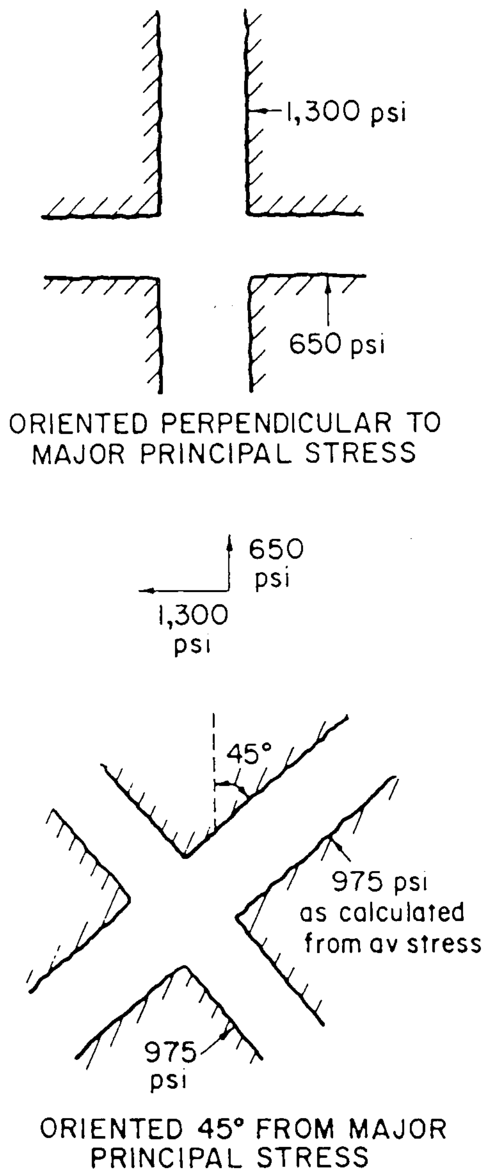

Figure 39.–Effect of entry orientation on in situ horizontal stresses (after Hill (*45*)).

that the major principal horizontal stress direction is approximately east-west. Mines in those areas have reported that cutter roof problems are often most severe in entries oriented perpendicular to the major principal horizontal stress, or north-south. Other studies have found that excessive horizontal stresses may develop beneath stream valleys (*46*, *83*).

Often the simplest solution to the problem of horizontal stresses is to reorient the entries. For example, where the regional stress field is known, gate entries oriented parallel to the major principal horizontal stress should suffer the least distress (fig. 39). Other methods that have proved successful in some instances are the use of truss bolts and high-strength combination bolts installed at very high torques (*13*). Stress control methods, including pillar softening, roof slotting, and yield pillars, have also been proposed as methods to control cutter roof, but the results so far have been inconclusive (*45*, *63*). One longwall mine found that the use of a sacrificial caving entry dramatically improved conditions both during development and during longwall mining, but the costs of the technique proved prohibitive (*3*, *74*).

MOUNTAIN BUMP POTENTIAL

Coal mine bumps are the rapid, violent failure of highly stressed coal. These powerful events have the potential to inflict severe injury on mining personnel and equipment. In the Western United States, bumps have occurred in many mines (*38*) and recently forced the closing of one longwall mine (*24*). At least three longwall mines in the southern Appalachian field comprising western Virginia, eastern Kentucky, and southern West Virginia have also experienced bump problems over the last several years.

Research has found that two conditions are normally present when bumps occur: (1) The coal seam is sandwiched between competent roof and floor strata, and (2) excessive vertical stresses are concentrated by retreat mining at depths exceeding 750 ft (*15*). Other factors that can contribute to bumps are load transfer from multiple-seam mining, faulting and other geologic disturbances, and the presence of strong, massive strata near the coal (*38*).

Mine operators may alleviate the bump hazard through the use of mine design, destressing techniques, or both. A pillar layout that has proven to be effective in bump-prone mines in the southern Appalachians is shown in figure 40. This design employs a large abutment pillar that absorbs considerable abutment load, thereby reducing the load carried by the tailgate corner of the longwall panel and minimizing the potential for face bumps. The abutment pillar is flanked by yield pillars that are too small to bump but that screen the gate entries from the possible effects of violent failure of the abutment pillar (*35*). Recent Bureau studies have indicated that the key to the success of this design is proper sizing of the abutment pillar (*14*, *56*).

Where the longwall face is prone to bumping, it may be necessary to resort to destressing techniques. The basic goal of destressing is the transfer of stress concentrations further into the panel, away from the face. This is achieved by fracturing or softening the coal through volley firing or auger drilling. Great care must be exercised in designing a destressing program, because the destressing operation itself may initiate a bump. Extensive Bureau research into destressing technology used in western U.S. coal mines was recently summarized by Haramy and McDonell (*38*).

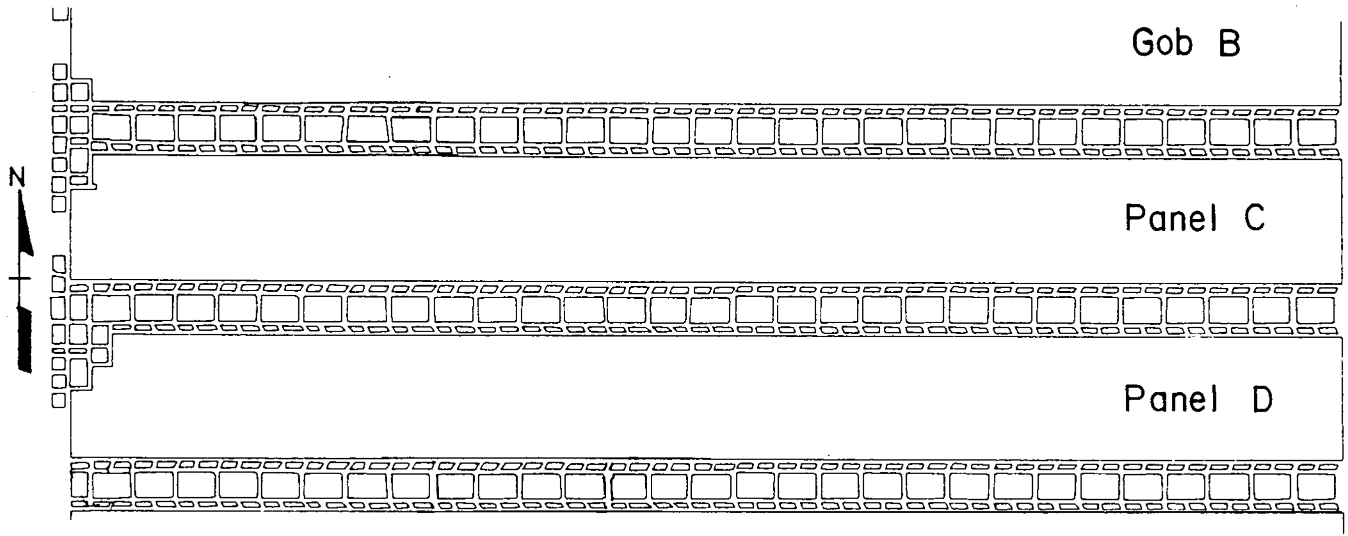

Figure 40.–Pillar layout for bump control (after Campoli and Heasley (*14*)).

MULTIPLE-SEAM INTERACTIONS

Vast quantities of coal reserves in the United States reside in multiple-seam configurations, and as the most accessible coal seams are mined out, more longwalls will be multiple-seam operations. Interactions caused by the presence of previously mined seams can greatly complicate ground control during longwall mining.

Full-extraction mining has two primary effects on the surrounding strata (fig. 41). First, zones of stress concentration are created in the vicinity of remnant pillars, while partially destressed zones develop above and below gob areas. The second effect, fracturing and subsidence, occurs in the rock above the extracted seam as it moves to fill the mined-out void.

Stress concentrations, or pillar load transfer effects, may be encountered either above or below previous mine workings. A comprehensive study of room-and-pillar retreat operations found that load transfer effects are usually not excessive unless the interburden is less than 110 ft thick (*41*), but others indicate that with longwalls such effects may be experienced at greater seam separations (*20*). Subsidence effects, on the other hand, are commonly observed all the way to the surface (*101*). They are most severe within the fracture zone (fig. 41), which typically extends to a height of 30 to 60 times the seam height above the workings.

The first decision mine planners face in a multiple-seam situation is which seam to extract first. A top-down mining sequence is nearly always preferable, because the severe interactions that can be associated with interburden subsidence affect only the overlying strata. One potential exception arises if the upper, mined-out seam may fill with water. Under these circumstances a longwall in the lower seam could experience water intrusion problems if the fracture zone it creates extends into the upper seam workings.

The next issue is how to lay out the gate entries. In Great Britain, it is common practice to "columnize" gate entries as shown in figure 42*A* (*105*). This approach does keep the zone of stress concentration away from the active workings, but at the price of sterilizing large amounts of reserves. Columnization has additional disadvantages when multientry gates are used, because the middle entries and all the crosscuts must be driven within the high stress zone. For this reason, columnization is usually not recommended in the United States. A preferable alternative would be to develop gate entries in the destressed zone beneath the gob (fig. 42*B*), which should result in improved roof conditions. A potential disadvantage of this approach is that the center of the panel is subject to pillar load transfer, which may impact roof stability at the face.

Perhaps the most effective layout would employ only yield pillars in the first seam to be mined. Total yield pillar systems should crush out once they are isolated in the gob, resulting in a fairly uniform stress field. Experience gained in European longwalls indicates that elimination of rigid pillars minimizes multiple-seam interactions (*54*).

Where possible, longwall panels should be laid out so that the face does not cross a gob and solid-coal boundary in the mined-out seam. If such a boundary must be crossed, better conditions will result if the direction of mining is from the gob to the solid and if the boundary is crossed at an angle of 30° to 45° (*53*).

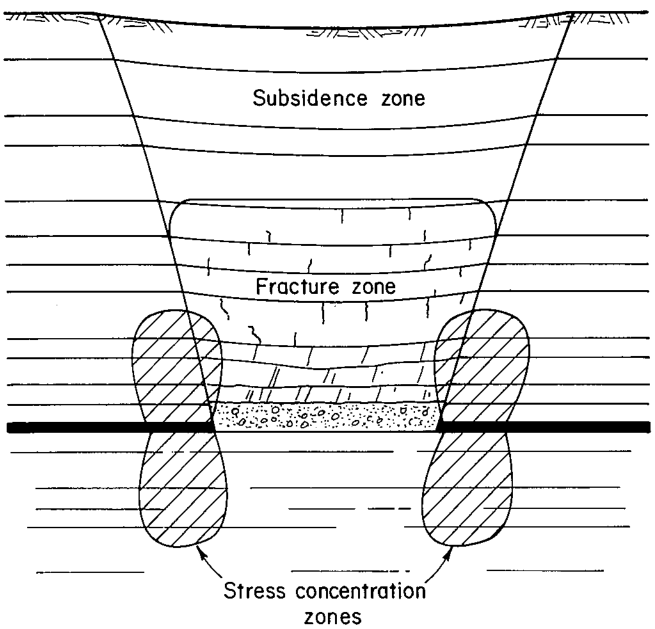

Figure 41.–Rock mass response to full-extraction mining.

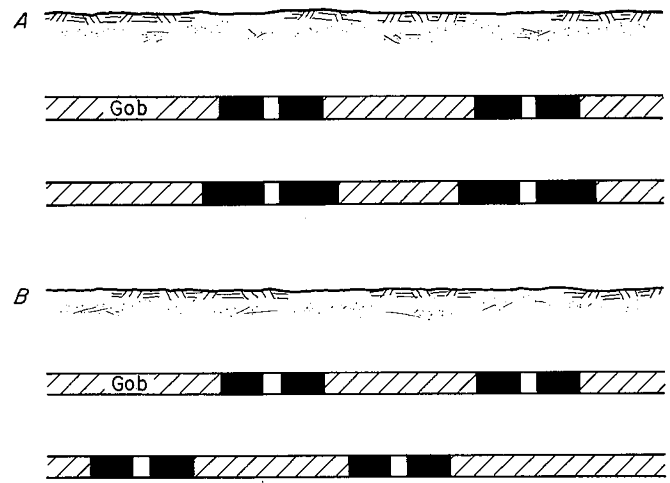

Figure 42.–Possible pillar layouts for multiple-seam mining. *A*, Columnization of chain pillars; *B*, location of chain pillars beneath gob areas.

SURFACE SUBSIDENCE

Surface subsidence is a major concern for much of the longwall industry. Chain pillars have an important influence on final subsidence profiles above longwall panels. Normally the ground does not subside fully above the chain pillars, resulting in "humps" between adjacent mined panels. The edges of these humps are zones of high surface horizontal strain, with the potential to cause considerable structural damage.

In spite of the obvious contribution of chain pillars to subsidence profiles, relatively little research has specifically addressed subsidence abatement through pillar design. One recent study compared the subsidence measured over four different chain pillar systems in the northern Appalachians (*59*). As the information presented in table 8 shows, more complete subsidence was measured over the chain pillars with the lowest ALPS stability factors. The results also indicate that more complete subsidence may also have reduced surface strains, as shown in figure 43. These results imply that oversized chain pillars may not be desirable where subsidence is a primary concern.

Yield pillars might also be expected to have an effect on surface subsidence profiles. A total yield pillar system might result in the most favorable profile, with almost uniform subsidence. Total yield pillar systems would also eliminate any possibility of long-term subsidence.

Table 8.–Subsidence over chain pillars

Mine	Overburden thickness, ft	Panel width (P), ft	Coal seam height (h), ft	Pillar centers, ft	ALPS SF	Subsidence over chain pillars, pct
1	510-660	605	5.5	100,100,80	2.63	0.0
2	680-950	625	6.0	100,100,60	1.36	9.5
3	745-910	630	6.0	90,90,90	1.29	16.5
4	660-710	1,000	5.5	80,80	1.12	17.0

SF Stability factor.

Source: Jeran and Adamek, (*59*, p. 67).

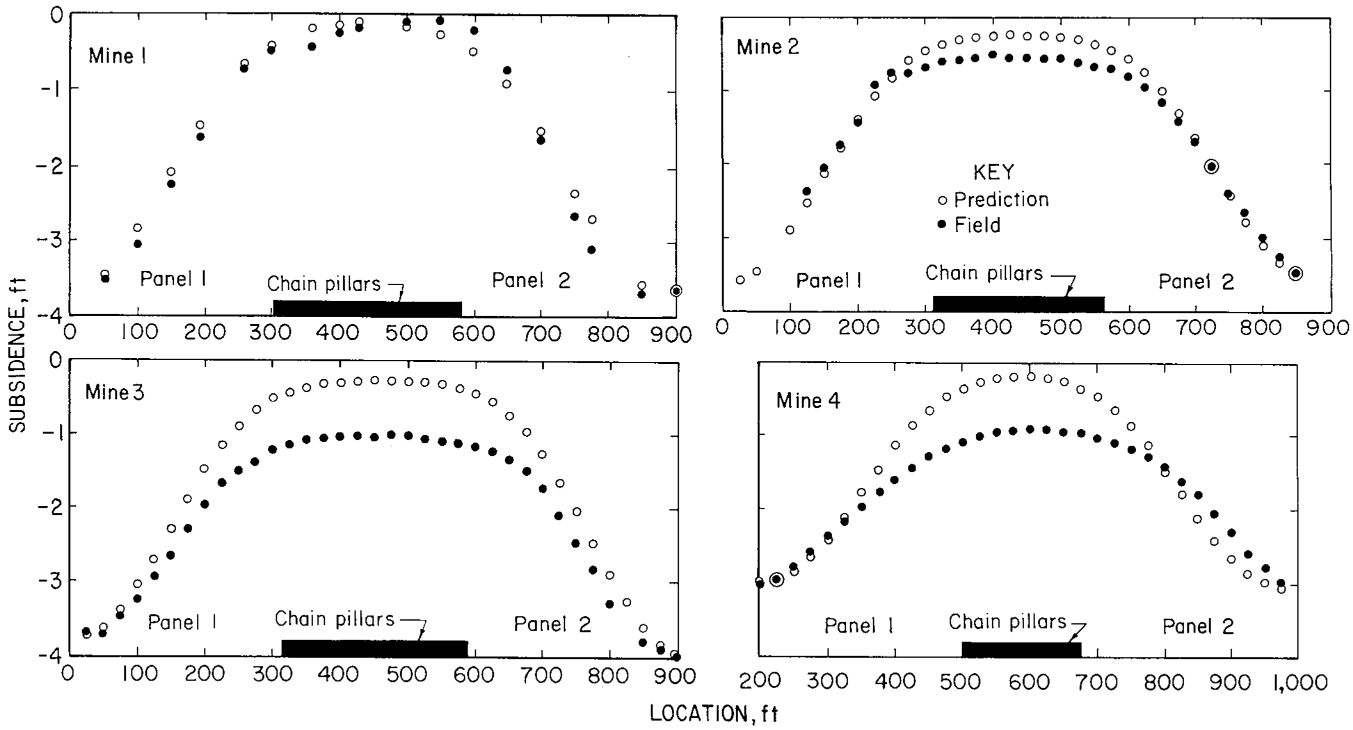

Figure 43.–Predicted and measured subsidence over chain pillars at four longwall mines (after Jeran and Adamek (*59*)).

STABILITY OF VENTILATION AIRWAYS

Ventilation requirements are a primary consideration affecting gate entry layout at many longwall mines. Deep, gassy mines in particular require large quantities of ventilating air both during development and on retreat. Stable, unobstructed gate entries are required to maintain ventilation.

The classic longwall ventilation scheme is the U-system (fig. 44*A*), in which intake air is brought up the headgate side, across the face, and returned via the tailgate entry (*31*). The U-system requires that stability in the tailgate be maintained outby the face. A gate entry design that meets current requirements regarding travelways on the tailgate side should also satisfy these ventilation needs.

Some longwalls have adopted the Y ventilation plan, in which intake air is also brought up the tailgate entry to the tailgate T-junction (fig. 44*B*). This system has a number of advantages from a ventilation standpoint, but it imposes an additional burden on the pillar design. Because the bulk of the return air must pass between the two goaves on its way to the bleeder entries, an open air passageway must be maintained inby the face on the tailgate side. Jim Walter Resources found that this ventilation requirement precluded widespread use of total yield pillar gate entry designs, because an open airway could not be maintained for long once the second face had passed (*81*). Similarly, Island Creek Coal Co.'s mines in Virginia recently increased their abutment pillar size, in part to improve face ventilation (*35*).

One ventilation requirement that impacts nearly all longwalls is the need to maintain bleeder entries around completed longwall panels. The longwall pillar design methods may be used to size bleeder pillars, considering a single side abutment as the design loading. A more conservative stability factor should be used where bleeder entries are expected to remain open for considerable periods of time, particularly when they must be inspected periodically.

Ventilation also impacts the number of entries that can be employed in gate entry systems. The ventilation regulations in 30 CFR require that at least three entries be driven when a conveyor belt is used, unless a specific variance is received from MSHA. A study conducted by Jim Walter Resources concluded that for its deep mines, four-entry longwall developments were superior to two- or three-entry systems because of ventilation requirements (*93*).

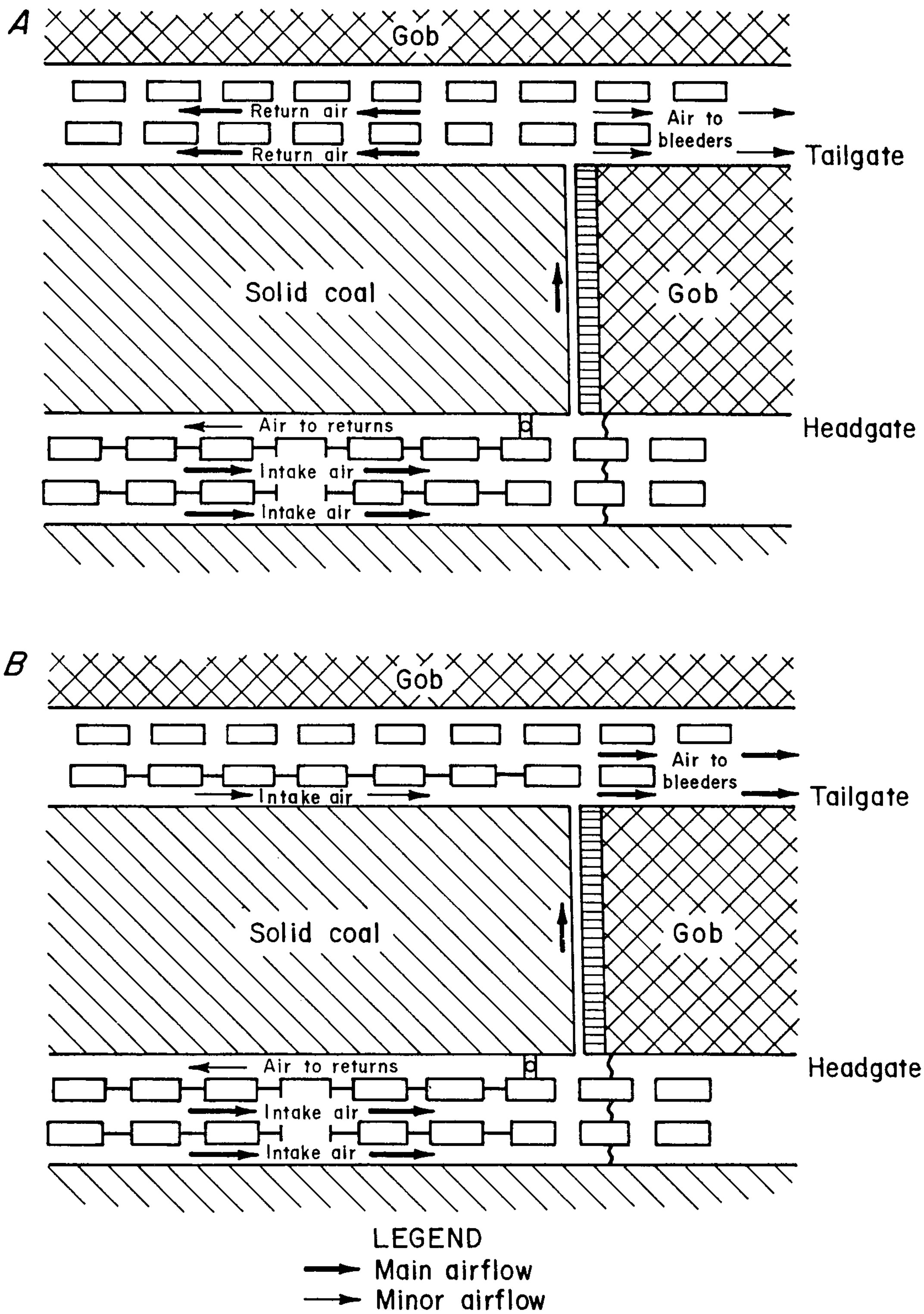

Figure 44.–Typical ventilation schemes for U.S. longwall panels (after Fuller (*31*)).

SUMMARY

Pillar design strategies for the special conditions described above may be summarized as follows:

Weak roof:

1. Increase stability factor of pillar design.
2. Reduce entry width.
3. Yield pillars may not be feasible.

Soft floor:

1. Increase stability factor of pillar design.
2. Flank abutment pillar with yield pillars.
3. Use yield pillars.

Excessive horizontal stress:

1. Adjust panel orientation.
2. Use truss bolts or high-strength combination bolts.
3. Use stress control methods.

Mountain bump potential:

1. Increase abutment pillar size.
2. Flank abutment pillar with yield pillars.

Multiple-seam interactions:

1. Work seams from top down.
2. Locate second seam gate entries under gob.
3. Use total yield pillar design in first seam.

Subsidence control:

1. Reduce stability factor of pillar design for more complete subsidence.
2. Use total yield pillar design.

CONCLUSIONS

Longwall is currently the safest and most productive method available for mining coal underground. For a longwall face to reach its full potential, stable gate entries must be maintained. Effective pillar design is often the single most important step a longwall operator can take to protect the gate entries.

This Bureau report has provided a number of tools that should be helpful in improving pillar design practice for longwall mining. The focus of the report has been on methods for conventional pillar design, where the pillars are sized to carry the abutment loads to which they will be subjected. The ALPS method was described in most detail, and three other methods were presented as well. In each case, the theories behind the method and the formulas necessary for its use were presented. Case history analysis was used to suggest the most appropriate input parameters and design criterion for each method. For example, it appears that ALPS usually works best when a stability factor in the range between 1.0 and 1.3 is used as the design criterion, and when the in situ coal strength is assumed to be 900 psi.

The report also described the current status of yield pillar design, in which the pillars are expected to transfer the abutment loads. Recent studies have shown that total yield pillar designs can be effective when they are combined with sufficient artificial support. Some field measurements were used to present preliminary guidelines for sizing yield pillars.

A number of other factors must often be considered in the longwall pillar design process, which are not directly addressed by the standard formulas. The report discussed ways in which pillar design might be adjusted, for example by increasing the stability factor for exceptionally weak roof conditions. Other factors, such as a need to reduce the potential for multiple-seam interactions, might point toward the selection of a yield pillar system over a conventional system.

Finally, pillar design is not the only variable in the longwall gate entry design process. Other important ground control parameters include the number of entries, entry span, and artificial support. Current Bureau research is investigating how these parameters may be incorporated into a complete design package for longwall gate entries.

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APPENDIX A.–STEP-BY-STEP GUIDELINES FOR USING THE ALPS METHOD

This section presents all of the information needed to use ALPS for practical longwall pillar design. The design equations used in ALPS are presented in a logical sequence, and a practical example of the use of ALPS is included.

The goal of ALPS is to size longwall pillar systems that are capable of carrying the abutment loads to which they will be subjected. ALPS consists of three basic elements:

- Estimating the load applied to the pillar system.
- Estimating the strength of the pillar system.
- Determining a design criterion (a stability factor).

Before using ALPS, it is necessary to collect the geometric and coal strength data used in the design equations:

- The depth of cover over the gate entries (H).
- The width of the panel, or face length (P).
- The entry width (w_e).
- The height of the coal seam (h).
- The unit weight of the overburden (γ, usually assumed to be 162 pcf).
- The in situ strength of the coal (S_1, usually assumed to be 900 psi).

Figure A-1 defines some of the geometric parameters listed above.

In regions of steep topography, the value of the depth of cover for use in ALPS can sometimes be difficult to determine. Using the maximum cover may be too conservative if it is present only over a small portion of the panel, but the average depth of cover might underestimate the load over the deeper sections. Some engineering judgment should be exercised, but in general an appropriate value of H is a high average expressed as

$$H = \left(\frac{H_{av} + H_{max}}{2}\right), \qquad \text{(A-1)}$$

where H_{av} and H_{max} are the average and maximum depths of cover over the panel, respectively.

ALPS also requires values for the parameters that are used to estimate the abutment load. Based on the research described in the main text, these parameters and their suggested values are

- The abutment angle (β) = 21°.
- The first front abutment factor (F_h) = 0.5.
- The second front abutment factor (F_t) = 0.7.

Finally, it is necessary to have estimates of the individual pillar widths (w), the pillar lengths (l_p), and the total width of the pillar system (w_t). The pillar widths are needed because both the development load and pillar strength equations are functions of w. If ALPS is being used to size pillars, it is necessary to perform several iterations, adjusting the pillar widths each time as required.

The length of the pillars is often based on ventilation and operations requirements. From a rock mechanics standpoint, the pillars should be as long as possible to increase the available load-bearing area. It is particularly important that the pillar length equal or exceed the pillar width. Pillar strength is determined by the smallest pillar dimension, and once the width exceeds the length of the pillars, very little additional load-bearing capacity can be obtained with further increases in pillar width.

The heart of the ALPS method is the estimation of the load applied to longwall pillars. The load estimation procedure begins with an estimate of the development load per foot of gate entry (L_d). Assuming that γ = 162 pcf, the tributary area expression for the development load may be written as

$$L_d = 162\,(H)\,(w_t)\,. \qquad \text{(A-2)}$$

The total pillar load is the sum of the development and the abutment loads. Two things are needed in order to

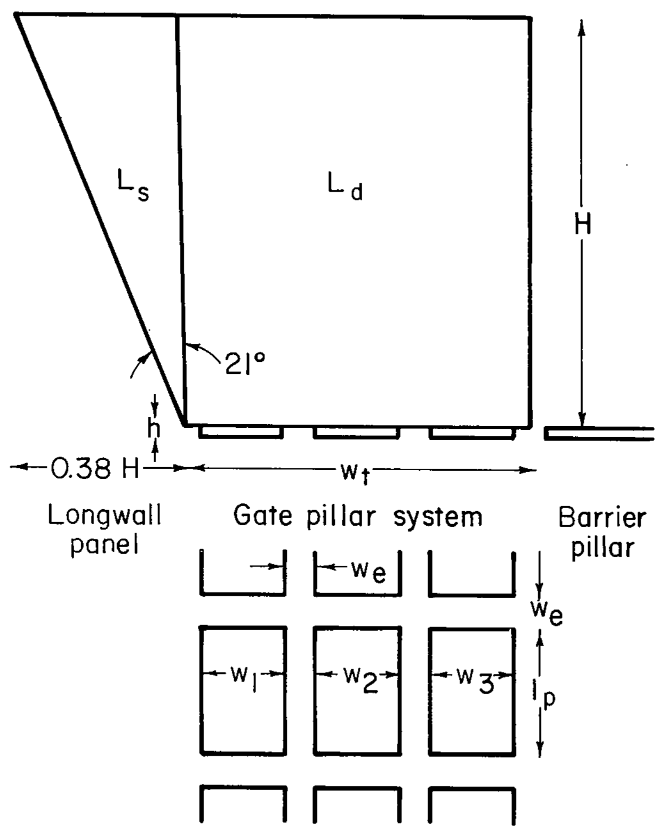

Figure A-1.–Definition of geometric parameters used in ALPS.

estimate the magnitude of the abutment load: the magnitude of the side abutment (L_s or L_{ss}) and the percent of the side abutment applied to the chain pillars (R). The first step in calculating the side abutment is to determine whether the panel width (P) exceeds the critical panel width. Assuming $\beta = 21°$, the critical panel width (P_{crit}) is

$$P_{crit} = (0.77)\ H\ . \qquad \text{(A-3)}$$

For critical and supercritical panels ($P > P_{crit}$), the magnitude of the side abutment per foot of gate entry (L_s) can be estimated as

$$L_s = (31)\ H^2\ . \qquad \text{(A-4)}$$

For subcritical panels ($P < P_{crit}$), the expression for the side abutment (L_{ss}) is

$$L_{ss} = 81\ (H)\ (P) - 53\ (P^2)\ . \qquad \text{(A-5)}$$

In equations A-4 and A-5 it is assumed that $\beta = 21°$ and $\gamma = 162$ pcf.

The percent of the abutment load applied to the chain pillars is the abutment fraction (R):

$$R = 1 - \left(\frac{D - w_t}{D}\right)^3, \qquad \text{(A-6)}$$

where D is the extent of the side abutment influence zone, which is equal to $9.3\ (H)^{0.5}$.

Now the maximum loading to which the pillar system is subjected (L) can be determined. The maximum loading depends on the services for which the pillar system will be used. Three possible loading conditions may be defined. The loading experienced by pillars at the T-junctions in the headgate, or in the tailgate during first panel mining, is called headgate loading. Headgate loading (L_H) consists of the development load plus the first front abutment, calculated assuming $F_h = 0.5$:

$$L_H = [L_d + (L_s)\ (0.5)\ (R)]\ . \qquad \text{(A-7)}$$

Pillars that are expected to protect bleeder entries will be subjected to the development load and the first full side abutment, or bleeder loading (L_B):

$$L_B = [L_d + (L_s)\ (R)]\ . \qquad \text{(A-8)}$$

The loads on barrier pillars may also be determined from equation A-8 by setting R = 1.

The most common design loading is tailgate loading (L_T), experienced during the mining of the second and subsequent panels. Tailgate loading consists of the development load, the first side abutment, and the second front abutment. Assuming $F_t = 0.7$, it is calculated as

$$L_T = [L_d + 1.7\ (L_s)]\ . \qquad \text{(A-9)}$$

Once the maximum pillar loads have been established, the next stage in the analysis is the estimation of the load-bearing capacity of the pillars. First, the strength of each individual pillar (S_p) is estimated using the Bieniawski formula (equation 11 in the text). If the in situ coal strength is assumed to be 900 psi, Bieniawski's formula may be written as

$$S_p = 576 + 324\ (w/h)\ . \qquad \text{(A-10)}$$

Then the load-bearing capacity of each pillar per foot of entry (B_p) is calculated as

$$B_p = \frac{S_p\ w\ l_p\ (144)}{(l_p + w_e)}\ . \qquad \text{(A-11)}$$

Then the load-bearing capacity of the pillar system per foot of gate entry (B) is calculated as the sum of the individual pillar resistances:

$$B = \Sigma\ B_p. \qquad \text{(A-12)}$$

Once both the load and the resistance of the pillars have been determined, the stability factor (SF) may be calculated as

$$SF = B/L. \qquad \text{(A-13)}$$

The final step in the analysis is the comparison of the stability factor determined in equation A-13 with a design criterion. If no previous longwall experience is available, a stability factor in the range of 1.0 to 1.3 should be used to size gate entry pillars. Where the roof and floor are expected to be especially competent, a stability factor towards the lower end of the range would be appropriate. If the roof is expected to be weak or the floor heave-prone, a stability factor towards the upper end would be more prudent. For barrier pillars that are expected to protect the main entries for a long period of time, higher stability factors (in the range of 1.5 to 2.0) should be used.

In many cases ALPS can be calibrated with actual field experience. Where several case histories are available from a given mine or mining area, a stability factor may be calculated for each pillar design and compared with the observed ground conditions. The stability factor that corresponds to acceptable conditions may then be used as the criterion for sizing pillars in future panels.

An important issue is whether it is better to use equal-sized pillars or combinations of large and small pillars in longwall gates. Although there is insufficient evidence to advocate one approach over the other, ALPS does predict that using a large pillar flanked on one or both sides by small pillars results in a more *efficient* design. The reason is that a single large pillar maximizes the load-bearing capacity for a given total pillar width, because pillar strength increases rapidly as the width-to-height ratio increases. For example, according to equation A-11, a single 100-ft-wide pillar has the same load-bearing capacity as two 68-ft-wide pillars. It appears that many mines could minimize the coal lost in chain pillars without compromising gate entry stability by shifting from equal- to unequal-sized longwall pillars.

If small pillars are used in combination with large ones in a three-entry system, the next question is whether to place the small pillars next to the tailgate or next to the headgate. Notable researchers have taken opposite sides of this question. Choi (*21*) made a case for placing the large pillar next to the headgate and the yield pillar next to the tail, while Peng and Chiang (*92*) argued for the reverse. Both designs have been successfully used in working longwalls, as have four-entry designs with a large pillar flanked by two small pillars (*16*). Again, it appears that the most important factor is not the arrangement of the pillars but whether the pillar system maintains an acceptable stability factor.

If a decision is made to use a combination of large and small pillars, the large pillars will provide most of the support. Their size will largely be determined by ALPS. The remaining question is what size of small pillar is best. As a rule of thumb, it is suggested that small pillars be sized to be weaker than either the roof or floor to reduce entry disturbance. Yield pillars may be even more effective, and the text provides preliminary guidelines for sizing them. Because yield pillar design is not yet a science, the performance of a yield pillar design should be evaluated in a less critical area before it is introduced into standard gate systems. The text shows some examples of yield pillar test areas.

Once an acceptable pillar layout has been obtained using ALPS, the next step in the design process is to check the result with the other available longwall pillar design formulas described in the text. Finally, the potential effects of the additional ground control factors that are discussed in the text should be considered.

The following problem illustrates how ALPS may be used to size longwall pillars.

Problem: A mine is developing its first two longwall panels in a block of coal reserves measuring approximately 2,000 by 4,000 ft. The seam is about 6 ft thick. The depth of cover over the proposed panels is approximately 1,000 ft. The face length is fixed by the available longwall equipment at 800 ft. The entries will be 18 ft wide, and three-entry gate systems will be used. MSHA approval has been obtained to drive the crosscuts on centers of 110 ft or the entry spacing, whichever is greater. It is desired to size the chain pillars for the three sets of gate entry systems, numbered G-1, G-2, and G-3 in figure A-2. If equal-sized pillars are used in each gate system, find the required pillar sizes.

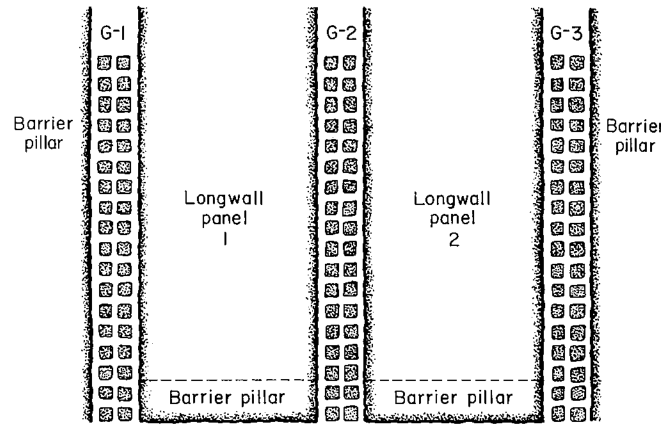

Figure A-2.–Panel layout for sample problem illustrating the ALPS method.

Solution: Each of the three gate systems will be subjected to different maximum service loadings. Therefore, in order to minimize the total amount of reserve sterilized in the chain pillars, the pillars will be sized for each gate system separately. A step-by-step solution for G-1 is given below.

1. Collect the required data:

$$
\begin{aligned}
H &= 1{,}000 \text{ ft.}\\
\gamma &= 162 \text{ pcf.}\\
P &= 800 \text{ ft.}\\
w_e &= 18 \text{ ft.}\\
l_p &= 92 \text{ ft (or the pillar width if } w > 92 \text{ ft).}\\
h &= 6 \text{ ft.}
\end{aligned}
$$

2. Provide initial estimates of the pillar width and the total width of the gate system:

$$
\begin{aligned}
w &= 72 \text{ ft.}\\
w_t &= 180 \text{ ft.}
\end{aligned}
$$

3. Estimate the development load using equation A-2:

$$
\begin{aligned}
L_d &= 162\ (1{,}000)\ (180)/2{,}000\\
&= 14{,}580 \text{ tons/ft.}
\end{aligned}
$$

4. Estimate the side abutment load. Because the panel is supercritical ($P > 0.77$ H), equation A-4 is used:

$$
\begin{aligned}
L_s &= 31\ (1{,}000)^2/2{,}000\\
&= 15{,}500 \text{ tons/ft.}
\end{aligned}
$$

5. Calculate the extent of the side abutment influence zone (D) and the abutment fraction (R), using equation A-6.

$$D = 9.3\ (1{,}000)^{0.5}$$

$$= 294.$$

$$R = 1 - \left(\frac{294 - 180}{294}\right)^3$$

$$= 0.94.$$

6. Estimate the maximum service loading (L). Since gate system G-1 will be used as a bleeder system, the maximum pillar loading is represented by equation A-8:

$$L_B = 14{,}580 + [(0.94)\ (15{,}500)]$$

$$= 30{,}080 \text{ tons/ft.}$$

7. Calculate the strength of the individual pillars in the system using equation A-10. As the pillars in G-1 are of equal size, only one calculation is necessary:

$$S_p = 576 + 324\ (72/6)$$

$$= 4{,}460 \text{ psi.}$$

8. Calculate the load-bearing capacity of the pillar system using equations A-11 and A-12:

$$B_p = (4{,}460)\ (72)\ (92)\ (144)/(92 + 18)\ (2{,}000)$$

$$= 19{,}340 \text{ tons/ft.}$$

$$B = (2)\ (19{,}340)$$

$$= 38{,}680 \text{ tons/ft.}$$

9. Calculate the stability factor (equation A-13):

$$SF = 38{,}680/30{,}080$$

$$= 1.29.$$

10. Compare the stability factor to a design criterion. In this example, no previous longwall experience is available, so a stability factor between 1.0 and 1.3 is suggested. The stability factor calculated in step 9 is very near 1.3, and should therefore be adequate. If a greater stability factor was desired, another iteration of steps 2 through 10 would be required. For example, in order to obtain SF = 1.5, 80-ft-wide pillars must be used.

The same procedure must be followed to size the pillars for G-2 and G-3, considering the different loading conditions in each case. The pillars in G-2 must protect the tailgate of the second panel and should be designed using the maximum load defined by equation A-9. If the entries in G-3 will not be used as bleeders for any length of time, then the headgate loading defined by equation A-7 can be used. Otherwise, equation A-8 would be more appropriate.

The advantage of using unequal-sized pillars may be illustrated using this example problem. If equal-sized pillars are used in G-2, each pillar must be 88 ft wide to maintain an SF of 1.3. The same SF could also be obtained using a 117-ft pillar in combination with a 20-ft pillar, thereby reducing the total pillar width by 23 pct, from 176 ft to 137 ft.

APPENDIX B.–ABBREVIATIONS AND ENGINEERING SYMBOLS USED IN THIS REPORT

A_t	total load-bearing area of the pillar system, ft^2
B	load-bearing capacity of the pillar system, lb/ft of gate entry
B_p	pillar load-bearing capacity, lb/ft of gate entry
C	shape constant, ft
c	gate road closure, ft
$C_1, C_2, \ldots, C_7$	constants, unitless
D	width of the abutment influence zone, ft
d	least dimension of laboratory specimen, in
d_{fs}	edge length of a full-scale coal cube, in
E_c	elastic modulus of coal, psi
E_f	elastic modulus of the floor, psi
E_i	elastic modulus of the immediate roof, psi
E_m	elastic modulus of the main roof, psi
F	a function of k in Wilson's formula, unitless
F_h	first (headgate) front abutment factor, unitless
F_t	second (tailgate) front abutment factor, unitless
H	depth of cover, ft
h	pillar height, ft
K	Gaddy strength factor, psi $in^{0.5}$
k	triaxial stress factor, unitless
L	design loading, lb/per ft of gate entry
L_A, L_B, L_{BP}, L_P	pillar loads, lb/ft of gate entry
L_B	bleeder loading lb/ft of gate entry
L_d	development load, lb/ft of gate entry
L_{fh}	first (headgate) front abutment, lb/ft of gate entry
L_{ft}	second (headgate) front abutment, lb/ft of gate entry
L_H	headgate loading, lb/ft of gate entry
l_p	pillar length (greatest pillar dimension), ft

l_r	rectangular pillar length, ft
LRS	limit of roadway stability, lb
L_s	side abutment for panels of critical and supercritical width, lb/ft of gate entry
L_{ss}	side abutment for panels of subcritical width, lb/ft of gate entry
L_T	tailgate loading, lb/ft of gate entry
n	number of entries in gate entry system
P	longwall panel width, ft
P_1	longwall panel length, ft
p'	uniaxial strength of fractured coal, psf
q	cover load or stress, psf
R	abutment fraction, unitless
RRF	rigid roof and floor, unitless
S_c	compressive strength of laboratory coal specimen, psi
SF	stability or safety factor, unitless
S_p	pillar strength, psi
S_1	in situ coal strength, psi
TRL	transferred remnant load, lb/ft of gate entry
UL	Ultimate Limit, lb
w	pillar width (least pillar dimension), ft
w_A	abutment pillar width predicted by Choi and McCain's equation, ft
w_e	entry width, ft
w_p	square pillar width, ft
w_r	rectangular pillar width, ft
w_t	width of pillar system, ft
w_y	yield pillar width, ft
x	distance from edge of longwall panel, ft
x_b	width of yield zone, ft
YRF	yielding roof and floor
α	size effect scaling factor, unitless
β	abutment angle, deg

γ	unit weight of the overburden, pcf
σ_A	average abutment stress, psf
σ_a	abutment stress, psi
σ_C	calculated pillar stress, psi
σ_m	measured pillar stress, psi
σ_s	side abutment stress, psi
$\hat{\sigma}$	peak abutment stress, psf
σ_P	total initial average pillar stress, psf
ϕ	angle of internal friction, deg