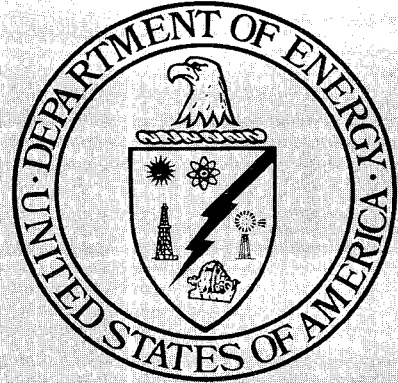


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IMPROVED DESIGN OF ROOM AND PILLAR COAL MINING

Final Technical Report

The Pennsylvania State University
University Park, Pennsylvania

June 1982

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U. S. Department of Energy
Assistant Secretary for Fossil Energy
Office of Coal Mining

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IMPROVED DESIGN OF ROOM AND PILLAR COAL MINING

FINAL TECHNICAL REPORT

FOR THE PERIOD

1 OCTOBER 1978 - 31 MARCH 1982

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Prepared for the

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To
Robert Stefanko (1926-1981)
who co-directed this project and
who always exemplified the best
in mining engineering research and education,
this report is dedicated.

ACKNOWLEDGEMENTS

The Co-directors of this project were Dr. Robert Stefanko and Dr. Z. T. Bieniawski, professors of mining engineering. Dr. George Luxbacher, research assistant, was responsible for the analyses of the coal mine survey data. Three graduate assistants were also involved: Robert M. Belesky (coal pillar stability and 'petite sismique' measurements), Farzan Rafia (assessment of mine roof spans by the Geomechanics Classification), David A. Newman (engineering geological mapping, internal sampling of floor strata and rock stress measurements). In addition, Dr. Jerzy Bauer, a visiting post-doctoral fellow, spent six months on this project conducting analytical studies of pillar strength and Dr. Danuta Krzyszton, a visiting research associate, spent five months investigating the theoretical aspects of the petite sismique technique. Mr. Jesse Core, adjunct professor and a former member of the President's Coal Commission, acted as an advisor.

ABSTRACT

The objective of this research grant was to improve upon the design of roof spans and coal pillars in a coal mining technique known as room-and-pillar mining.

Essentially, the project consisted of three aspects:

- i) determination of stable roof spans
- ii) determination of the strength of coal pillars
- iii) determination of safety factors for room-and-pillar coal mining conditions in the United States

The study included a critical review of the available pillar design formulas as well as the design methods for selecting stable roof spans.

Three novel approaches were utilized during this research:

- i) the "petite sismique" technique was assessed for possible determination of coal pillar deformability; this was the first use of this technique in the United States since its development in France.
- ii) the "Geomechanics Classification" was extended for determination of safe roof spans in room-and-pillar coal mining.
- iii) a national survey of the current design practice as well as of the stable and failed coal pillars and roof spans was performed with the aim of determining factors of safety in room-and-pillar coal mining.

Research investigations included field studies, laboratory testing and analytical computer simulations.

The final outcome of the project is a proposal for design code for room-and-pillar coal mining in the United States.

In the course of this research, seven publications were prepared and three M.S. theses were completed.

For practical applications of this research, the interested reader may turn to Chapters 4 and 5.

TABLE OF CONTENTS

	<u>Page</u>
Acknowledgements	ii
Abstract	iii
List of Figures	vi
List of Tables	vi
I. INTRODUCTION	1
1.1 Statement of the Problem.	2
1.2 Scope of Study and Novel Approaches	2
1.3 Publications from Research Performed	2
II. STABILITY OF ROOF SPANS IN COAL MINES	5
2.1 Engineering Geological Characterization of Coal Bearing Strata	6
2.1.1 In-mine observations and mapping	8
2.1.2 Geotechnical core drilling and logging	9
2.1.3 Integral sampling	12
2.1.4 Laboratory material testing	15
2.2 Assessment of Roof Spans by Rock Mass Classifications	18
2.2.1 Roof strata classifications	19
2.2.2 Modifications to the Geomechanics Classification	26
2.2.3 Collection of roof-fall data	30
2.3 Prediction of Stable Roof Spans	31
III. STRENGTH AND DEFORMABILITY OF COAL PILLARS	41
3.1 Literature Review	41
3.1.1 Pillar load considerations	42
3.1.2 Pillar strength formulae	45
3.1.3 Pillar deformability	59
3.2 Field Study of Pillar Loading	62
3.2.1 Site description	63
3.2.2 Results	64
3.3 Numerical Analysis of Pillar Load	64
3.3.1 Procedure	65
3.3.2 Results	67

	<u>Page</u>
3.4 Evaluation of the Petite Sismique Technique	69
3.5 Recommended Pillar Strength Formula	72
IV. SURVEY OF MINING PARAMETERS FOR ROOM-AND-PILLAR COAL EXTRACTION	85
V. A DESIGN CODE FOR ROOM AND PILLAR MINING	107
5.1 Basic Concepts	107
5.2 Selection of Roof Spans	108
5.2.1 Geomechanics Classification.	108
5.2.2 Applications	113
5.3 Pillar Sizing	115
5.3.1 Pillar load	115
5.3.2 Pillar strength.	117
5.4 Factors of Safety	121
5.4.1 Assumptions for pillar load determination. . . .	121
5.4.2 Assumptions for pillar strength determinations .	121
5.5 Design Procedure.	127
5.5.1 Planning new mines	127
5.5.2 Optimizing mining in progress	134
5.6 Practical Examples of Mine Design	134
VI. CONCLUSION AND RECOMMENDATIONS	147
6.1 Conclusions	147
6.2 Recommendations for Future Research	148
APPENDICES	149

LIST OF FIGURES

		<u>Page</u>
Figure 1	Geotechnical Core Log	11
2	Integral Sampling Technique	14
3	A Comparison of Output Data from the Geomechanics Classification and the Q-System	24
4	Relationship between RQD and Discontinuity Spacing.	29
5	Stand-up Time versus Roof Span for Coal Mine Roofs.	32
6	Critical-size Phenomenon Exhibited by Different Rock Types	47
7	Specimen Size Effects in Coal (Singh, 1980)	48
8	Compressive Strength as a Function of Cube Size for Pittsburgh Coal (Hustrulid, 1975)	49
9	Comparison of Protodyakonov's Equation with Experimental Data	50
10	Effect of Width-to-height Ratio on Strength of Sandstone	52
11	Correlation between Static Modulus of Deformation and Shear Wave Frequency from "Petite Sismique"	60
12	Comparison of Results from "Petite Sismique" and Plate Bearing Tests	62
13	Locations of Instrumentation in a Coal Pillar Studied	63
14	MINSIM Representation of Panel Geometry	66
15	Initial Results from SAP IV and MINSIM analyses	67
16	Numerical and In Situ Results of Relative Pillar Loads	68
17	Results of Petite Sismique Trials in Coal	71
18	Pittsburgh Seam: Strength vs w/h Ratio of Pillars.	76
19	Pittsburgh Seam: Strength vs Pillar Width for h = 10 ft	77
20	Comparison of Pillar Strength by Empirical Formulas and a Theoretical Strength Formula.	78
21-32	Histograms of Data from PSU Survey of Coal Mining Dimensions.	88-99
33-39	Regression Analyses of Correlations between the Various Parameters from PSU survey.	100-106

LIST OF TABLES

Table 1	Research Plan and Personnel	3
2	Fracture Index and Rock Mass Quality.	15
3	Results of Elastic Velocity Determinations on Coal and Limestone	18
4	Elastic Pillar-Load Formula by Coates	44
5	Range of Room-and-Pillar Parameters Used in U. S. Coal Mining (Survey of 174 Cases)	87

FIGURES AND TABLES USED FOR THE DESIGN CODE

(CHAPTER 5)

		<u>Page</u>
Table 1	Geomechanics Classification	109
2	Effect of Discontinuity Orientations.	109
3	Example of Classification Procedure	111
4	Classification of Roof Conditions	114
Figure 1	Output of Stand-up Time and Roof Span	112
2	Tributary Area Concept for Pillar Loading	116
3	Comparison of Pillar Strength Formulas w.r.t. w/h Ratio	120
4	Histogram of Safety Factors - PSU Strength Formula.	124
5	Histogram of Safety Factors: Obert-Duvall Strength Formula.	125
6	Histogram of Safety Factors: Holland Strength Formula	126
7	Pillar Strength in Pittsburgh Seam vs w/h Ratio as Predicted by Three Formulas	129
8	Pittsburgh Seam: PSU Formula, Depth 500 ft, Span 18 ft.	130
9	Pittsburgh Seam: Holland and Obert-Duvall Formulas, Depth 500 ft, Span 18 ft.	131
10	Pittsburgh Seam: PSU Formula, Depth 1000 ft, Span 18 ft.	132
11	Pittsburgh Seam: Holland Formula, Depth 1000 ft, Span 18 ft.	133
12	Pittsburgh Seam: Pillar Strength vs Pillar Width for Different Pillar Heights and Different Depth, Span 20 ft PSU Strength Formula	137
Chart I	Design Code: Pillar Width vs Entry Width for Different Extraction Ratios	138
Chart II	Design Code: Depth vs Width-to-height Ratio for Different Extraction Ratios Applicable to the Pittsburgh Seam and Safety Factor $F = 1.5$	139

CHAPTER 1

INTRODUCTION

The efficient and safe design of room and pillar panels in coal mines remains as one of the most pressing problems facing the mining industry in the United States. Although some attention has been paid recently to the in situ strength of coal pillars, no satisfactory pillar design method has as yet been formulated. In addition, insufficient attention has been devoted to the design of roof spans in room-and-pillar layouts. Present-day design is a trial-and-error procedure based mainly on experience, and while this approach might have been acceptable in the past, this will not be the case in the future. As coal mining takes place at greater depths, and this energy resource becomes more critical nationally, an improved design philosophy for room-and-pillar coal workings is necessary. This is particularly so because this method of coal extraction is most commonly used in the United States in underground coal mining. Although there is considerable interest in other methods of coal extraction such as longwall and shortwall mining techniques, as well as newer, more novel systems, undoubtedly the use of pillar support in development entries will continue to be of prime importance. Development entries, serving as service entries for such vital services as ventilation power distribution, escapeways and haulage, represent the umbilical cord of a coal mine and must be maintained in a safe and adequate manner. Pillars provide the primary support in any mining system, and thus the proper design of spans and pillars for these main arteries is paramount.

The size and shape of pillars will markedly effect production. By utilizing the optimum sizing and spacing of pillars, a more productive mining sequence will become possible. However, the extraction ratio in U. S. coal mines has traditionally been just slightly in excess of 50 percent. More recently, some states have been passing subsidence legislation requiring the use of pillar supports to protect surface structures from subsidence. Thus the extraction ratio will be reduced further unless better pillar design techniques are developed.

Furthermore, with increasing depths, spans have been reduced and pillar sizes increased. While this has resulted in reasonably good ground stability in the past, as mines are now going deeper than 2,000 feet, the limits of room-and-pillar mining are being reached. This will require modification of the mining systems currently used. Such modifications can only come with a better knowledge of pillar and span design.

The objective of this research was to improve upon the design of pillars and roof spans in the coal mining technique known as room-and-pillar mining. A critical review of the available mine pillar and roof span design approaches developed in the United States and abroad has been conducted. However, in order to apply any design formulas to full size coal pillars, a method had to be found for

determining the characteristics of the in situ coal. For this purpose, a novel method known as the "petite sismique" technique was selected. This was the first use of this technique in the United States.

Simultaneously with the pillar studies, research was conducted to determine the safe roof spans in room and pillar coal mining. Use was made here of another novel technique known as the "Geomechanics Classification of rock masses"--a technique successfully applied to the design of tunnels and other underground excavations.

These investigations were supplemented by a field survey of stable and failed coal pillars and roof spans in the United States to determine factors of safety in room-and-pillar coal mining. The final aim of the project was to propose a design code for room-and-pillar coal mining.

1.1 Statement of the Problem

No comprehensive design code is available for room-and-pillar coal mining in the United States. As this method of mining accounts for over 90 percent of all underground coal mining in the USA, there is a pressing need for a practical design code.

1.2 Scope of Study and Novel Approaches

The research plan and the personnel employed during the three and one-half years of research are listed in Table 1.

Objectives

1. Improve design of roof spans
2. Improve design of coal pillars
3. Use high risk innovative approaches: e.g., integral sampling, petite sismique, rock classifications

Tasks

1. Critical review of roof and pillar design formulas
2. Use of Geomechanics Classification (modified for coal mining) for assessment of roof quality and span selection (support selection excluded)
3. Application of "petite sismique" technique to coal mining.
4. Survey of room-and-pillar dimensions in the USA for determination of factors of safety
5. Proposing a design code for room-and-pillar mining

1.3 Publications from Research Performed

In the course of the research conducted on this project, the following publications were prepared:

Table 1. Research Plan and Personnel

<u>Name and Task</u>	<u>1st year: 1979</u>	<u>2nd year: 1980</u>	<u>3rd year: 1981/82</u>
Project Directors: Z. T. Bieniawski and Robert Stefanko (died July 25, 1981)	Project formulation and direction.	Propose procedures for roof span selection and pillar strength determination.	Finalize design code and test for validity by case histories.
Pillar strength: R. Belesky	Review pillar design formulas. Conduct field studies in Utah. Assess 'petite sismique' in laboratory. Review published literature.	Assess 'petite sismique' in mines (limestone and coal). Analyze Utah field data.	Compare field data with numerical modeling of stress distribution in coal pillars.
Roof stability:	Apply rock mass classifications to coal mining. Modify Geomechanics Classification.	Finalize classification changes. Recommend relationship of stand-up time vs span for coal.	- -
Site characterization and stress measurement experiments: D. Newman	Develop geotechnical input data collection for rock mass classifications (engineering geological mapping and core logging procedures).	Conduct sonic tests in laboratory. Conduct petrographic analyses. Develop integral sampling procedures for coal mine floors.	Field trials of rock deformability. Rock stress measurement experiments in the laboratory.
Safety factors:	Prepare a survey of stable and failed coal pillars and roofs in the U.S.	Analyze data and responses.	Recommend factors of safety.
Analytical studies: Bauer/Krzyszton (post-doctoral fellows)	- -	Theoretical 'limit design' formulation (Bauer)	Theoretical basis for petite sismique (Krzyszton).

- Bieniawski, Z. T., F. Rafia and D. A. Newman "Ground control investigations for assessment of roof conditions in coal mines," Proc. 21st U. S. Symp. on Rock Mech., University of Missouri, Rolla, 1980, pp. 691-700.
- Bieniawski, Z. T. "Current possibilities for rock mass classifications as design aids in mining," SME-AIME Fall Meeting, Minneapolis, Paper No. 80-349, AIME, 1980.
- Bieniawski, Z. T. "An overview of ground support considerations in room-and-pillar coal mining," Proc. Polish-American Conf. on Ground Control, Southern Illinois University, Carbondale, August 1980.
- Bieniawski, Z. T. "The 'Petite Sismique' technique - a review of current developments," Proc. 2nd Conf. on Acoustic Emission/Microseismic Activity, Trans Tech Publications, Clausthal, 1980, pp. 305-318.
- Bieniawski, Z. T. "Estimating Safe Roof Spans in Coal Mines," Proc. 11th Annual Inst. on Coal Mine Health, Safety and Research, Va. Polytechnic Inst., Blacksburg, 1980, pp. 181-187.
- Bieniawski, Z. T. "Improved design of coal pillars for U. S. mining conditions," Proc. Conf. on Ground Control in Mining, West Virginia Univ., Morgantown, July 1981, pp. 13-22.
- Belesky, R. M., Z. T. Bieniawski, and R. Greenfield "Latest research investigations into the 'Petite Sismique' technique," Proc. 3rd Conf. on Acoustic Emission/Microseismic Activity, Trans Tech Publications, Clausthal, October 1981.

Theses:

- Rafia, F. "An Assessment of Room-and-Pillar Coal Mine Roof Conditions by Means of Engineering Rock Mass Classifications," M.S. Thesis, The Pennsylvania State University, May 1980, 122 pp.
- Newman, D. A. "Engineering Geological Characterization of Coal Bearing Strata," M.S. Thesis, The Pennsylvania State University, November 1980, 158 pp.
- Belesky, R. M. "An In Situ Assessment of Deformability and Strength Coal Pillars," M.S. Thesis, The Pennsylvania State University, May 1981, 275 pp.

CHAPTER 2

STABILITY OF ROOF SPANS IN COAL MINES

The increased demand for energy during the past decade, compounded with difficulties in stimulating increased production from present energy sources, has focused attention on the more effective utilization of existing coal reserves. Since the development of a mining operation is capital intensive and the trend toward increased mechanization of mining creates less flexible mining methods, unknown geologic conditions heighten the financial and safety risks.

Safety and economic considerations are intimately related in underground coal mining; 75 percent of all coal mining fatalities occur underground (Wier, and Wobbler, 1976) and roof falls accounted for 40-50 percent of the fatalities in 1977 and 1978 (Marovelli, 1979). Therefore a detailed understanding of the geological conditions, particularly the roof, rib and floor strata, is a prerequisite to all phases of mine planning.

The isolation of geologic factors associated with roof instability and the prediction of roof conditions in advance of mining have been the subject of extensive research (Ealy et al., 1979, Campbell et al., 1975, Hylbert 1978, Dunham et al., 1977, Rinkenberger, 1979). The methods employed in these investigations range from remote sensing and lineament analysis to detailed in-mine geological mapping. Each technique contributes to the understanding of the geological conditions but cannot be directly integrated into the design process. Engineering geology quantifies geological description and provides the engineering parameters which are then used to predict the behavior of the rock mass.

Presently, the application of geology in relation to coal mining is insufficient (Ferm, 1976). Yet, underground civil engineering projects such as tunnels, underground chambers, and hydroelectric schemes depend upon and utilize engineering geological data for design. These data serve as input for rock mass classification systems which provide output in terms of unsupported spans or stand-up times, and which recommend support for permanent stability. The concern here is whether engineering geological methods, similar to those used in large underground civil engineering projects, can be adapted for underground coal mines.

The objectives of this aspect of the project are as follows:

- 1) To characterize coal-bearing strata in an engineering geological fashion by considering the roof, rib, and floor as the main structural units.
 - a) The modification of the peripheral technique of tunnel mapping to coal mines, including the development of a data collection format and procedure.

- b) The development of a geotechnical core log, a logging procedure, and core handling recommendations designed for coal mine strata.
- c) The design of integral sampling equipment to investigate coal mine floor conditions.
- 2. To evaluate the rock materials encountered in terms of their sonic velocity characteristics in relation to the seismic velocity in rock masses.
- 3. To analyze petrographic thin sections to ascertain the mineral composition and other relevant features of the coal bearing strata.
- 4. To apply the data collected above to the Geomechanics Classification System for assessing coal strata quality.
- 5. To initiate a data base system.

2.1. State-of-the-Art: Engineering Geological Characterization of Coal Bearing Strata

The characterization of the subsurface geological environment is essential to mineral exploration and to the estimation of ground support requirements. Throughout history, methods for collecting, portraying, and integrating of geological information for mine planning and design have been associated with mineral exploration and extraction.

Traditionally the role of geology in coal exploration, mine planning and development has been small (Ferm et al., 1976). This is attributed to the oversimplification of the geological concepts associated with the planning of coal mines. These concepts are as follows:

- 1) Coal occurs in beds or layers and is overlain or underlain by claystones, shales, sandstones, or limestones in varying proportions.
- 2) Each coal bed is named and certain quality characteristics (thickness, BTU, ash, and sulfur content) can often be associated with particular beds.
- 3) The coal beds and immediate strata are often folded into broad anticlines and synclines, and in some places, displaced or faulted (Ferm et al., 1976).

Since the quality and thickness of the coal deposits varies laterally, these guidelines are insufficient. The occurrence of these deviations was believed to be unpredictable. The only feasible solution for ground control problems produced by changes in the coal seam and immediate strata relied upon past experience and engineering judgment. Yet, the mechanization, economics, and safety requirements connected with modern coal mining demand complete knowledge of the geological environment to assure a profitable operation. Therefore, numerous methods have been developed to define the regional geology of a particular area.

Remote Sensing - Lineament Analyses

While flying from Oklahoma to Kansas in 1928, J. L. Rich (Rich, 1928) observed distinct jointing patterns expressed on the Dewey limestone. He traced the regional jointing through the alteration of vegetation growing in the fractures. Since this initial discovery, considerable attention has been focused on the use of aerial photography to delineate patterns of regional jointing. Mollard, 1957, conducted detailed studies relating the surface expression of fractures visible in aerial photographs to subsurface structures. Lattman, 1958, attempted to standardize terminology used in aerial studies by defining fracture traces as features less than one mile in length and lineaments as those greater than one mile.

Hough, 1960, conducted extensive research in the Morgantown, West Virginia Quadrangle. He found the orientation of regional discontinuities observed in aerial photographs was similar to the orientation of the local structure, drainage pattern, and to the coal cleat. This was used in ascertaining the direction of the original stress and any subsequent rotation of the stress field. He theorized that the drainage pattern aligned itself with the fractured, less resistant zones associated with the lineaments. Overbey et al., 1973, in a study on the prediction of roof instability in advance of mining discuss the effected of topographic unloading due to stream erosion. Elder et al., 1974, used radar imagery to determine the structural geology in an area of Buchanan County, West Virginia. They found it to be a valuable aid in the planning of mine sites and locating potential oil and gas drilling sites.

The launching of Skylab and the LANDSAT satellites provided a new source of aerial photography. Wier and Wobber, 1976, Campbell et al., 1975 and Dunham et al., 1976 discuss techniques of lineament analysis using black-and-white, color, and infrared satellite photography. An excellent discussion of fracture mapping and lineament analysis using satellite and aerial photography is presented by Rinkenberger, 1977. He cites fracture orientation, density, topographic unloading and orientation of mine workings as being highly correlated with underground instability. The approach to remote sensing used by the Mine Safety and Health Administration is outlined and includes the effect of different spectral bands in color imagery.

Hazard Analysis

Recognizing that fractures and lineaments alone do not characterize the geologic environment, several researchers have combined lineament analyses with other significant geological features. Overbey et al.,

1973, Dunham et al., 1976, and Ealey et al., 1979, incorporated numerous geologic factors into hazard analysis maps. This technique involves the plotting of topography, drainage patterns, lineaments, faults, changes in roof lithology, sandstone channels, roof falls, floor heaves, the extraction ratio, and areas of potentially high water head on the overlays of a mine property. The location and interaction of potential geological hazards in relation to existing and planned mine workings are clearly presented in the overlays. The hazard analysis map is used to predict the occurrence of potentially unstable areas. A similar technique was devised (Hylbert, 1978) in which various roof rock lithologies were classified according to their relative stability. The results of borehole logs were used to delineate potentially unstable areas.

Paleodepositional Environment

The study of paleodepositional environments is a relatively new facet in coal geology, developed in the last twenty years. It synthesizes the characteristics of coal-bearing strata with observations of modern lake and swamp environments. Investigations in the Appalachian basin have demonstrated that one of the critical determinants of coal seam character is the depositional environment of the coal (Horne et al., 1978, Williams and Ferm, 1964, Houseknecht, 1978). The depositional surface on which a coal swamp develops largely controls its extent and thickness, while the overlying sediment deposited on the peat influences the quality of the coal seam (Ferm et al., 1976, Williams, 1959).

Lineament analysis, hazard analysis and paleodepositional environment are regional techniques which delineate areas of potential instability prior to mining on the basis of fracture frequency, orientation, and other significant geological features. However, none of the regional techniques quantifies the geologic environment into parameters applicable to engineering design. This can only be obtained through detailed engineering geologic investigations, including mapping, diamond core drilling, and the physical testing of the coal bearing strata.

2.1.1 In Mine-Observations and Mapping

In the last decade, numerous techniques have been developed to characterize the geological environment of an underground coal mine. These techniques encompass the assessment of the regional geology through lineament and hazard analysis (Weir and Wobber, 1976, Campbell et al., 1975, and Rinkenberger, 1977), the investigation of the subsurface environment using diamond core drilling (Geol. Soc. of London, 1970, Cameron, 1971, and Kalia, 1976) and underground geological mapping (McCulloch et al., 1975 and Bolstad et al., 1974).

Engineering Geological Mapping

The primary objective of an underground engineering geological mapping program is to portray accurately the three-dimensional qualities of the entries, crosscuts, and mine geology on paper. Of particular importance are the orientations of discontinuities (bedding planes, faults, and joints) in relation to the excavation. Since the stresses surrounding an underground opening do not exceed the strength of the rock material, the behavior of the rock mass is governed by the location, orientation, and condition of the discontinuities. A mapping method was developed for use in underground coal mines (Bieniawski, et al., 1980). It is based on the peripheral method of tunnel mapping and enables large areas to be mapped without sacrificing accuracy or detail. The input parameters were chosen to quantify the geological information and the engineering characteristics of the rock mass. The parameters are directly applicable to rock mass classification schemes enabling an estimation of the stand-up time, roof span, and support requirements. This serves as an important aid to mine planning and design.

Engineering geological mapping was conducted in a coal mine and in an underground limestone mine, referred to as Mines I and II. At Mine I the mapping was conducted around a pillar, instrumented for seismic testing (Belesky, 1980). A Clar compass provided the orientation of discontinuities in the form of dip direction and dip. There were few observed discontinuities, and with the exception of slight spalling due to weathered shale, the roof was in excellent condition. The location of the source plate and geophones was recorded as the mapping proceeded around the pillar. The roof span and intersection length were also recorded.

The mapping at Mine II (limestone) was initiated at the outby corner of the pillar and was continued around the test area. Due to the height of the pillar (45-50 feet), a 200 ft cloth tape was placed on the rib at eye level, approximately five feet high. The location, orientation, persistence, and condition of each discontinuity were recorded on an input data sheet. At intervals of roughly 30 feet, entry span measurements were taken. Red spray paint was used to mark areas of particular interest and these were photographed later. Emphasis was placed on an accurate representation of the in-mine conditions; consequently, discontinuities without sufficient exposure in the entry were ignored. The Clar compass was highly effective in mapping as both dip direction and dip measurements could be taken simultaneously. In addition, a mirror permitted measurements above eye level.

2.1.2 Geotechnical Core Drilling and Logging

Diamond core drilling is an integral facet of any engineering geological investigation in an underground coal mine. The core specimens enable the engineering geologist to estimate the rock mass quality through detailed examination of the orientation, spacing, and condition of the discontinuities. This is supplemented by a series of rock mechanics tests to assess the strength, moduli, and behavior of the rock material.

The geotechnical core log provides a standardized format to record the geological and engineering characteristics of the rock core in addition to the results of any field tests. The core log must be systematic to record the vast amount of information available from the core. Due to the rapid deterioration of core samples, the log should enable all relevant investigations to be conducted in the field and be designed so the data are recorded as the core is extruded from the core barrel. The geological and engineering parameters must illustrate and quantify only those features of the rock mass that influence the engineering design.

Geotechnical Core Log Developed at Penn State

After evaluating the existing core logs none was found to be completely satisfactory for coal mining applications. A geotechnical log, shown in Figure 1, was developed for use during this study.

The top line of the core log is completed prior to drilling. The location is preferably defined by underground survey stations, facilitating the transfer of data to plan maps. If this is not possible, it must be located by entry and pillar number. The column designated "run and length" is completed as the core barrel is withdrawn from the borehole. The core must be gently extruded from the barrel and under no circumstances should it be removed by hammering the side of the barrel. Once the core is out of the barrel, it is gently washed to remove dirt and rock chips. This may be omitted in the case of loosely consolidated sediments, fault gouge, or clay veins. Several color photographs should be taken while the wet core is lying in the box. These photographs will serve as a record of the core and are included with the final log. Percent core recovery and RQD are the initial quantities calculated once the core is in the box. Quantitative data are recorded in a histogram format. The histograms are arranged so that optimal conditions (high RQD, fracture frequency, etc.) are depicted by a completely blackened column. Therefore the blacker its particular column, the more favorable the given rock mass parameter. The core in the box is read like a book, left to right and top to bottom. The strata depth should increase to the right in a row and to the front in the box.

Core recovery is the length of core taken from the core barrel divided by the length of the run and is most consistently expressed as a percentage. Rock Quality Designation (RQD) is calculated as the sum of core pieces greater than 4 inches divided by the length of the run and again expressed as a percentage.

<u>RQD Value</u>	<u>Classification</u>
100 - 90%	very good
90 - 75%	good
75 - 50%	fair
50 - 25%	poor
> 25%	very poor

If the core recovery is less than 100 percent, a wood block equal in length to the lost core should be placed at the beginning of the run. An exception is made in cases where the lost core can be attributed to a specific segment of the core run. The fracture spacing is the distance, recorded in millimeters, between adjacent fracture. If any infilling is present, its type (breccia, clay, etc.) and thickness should be noted in the description of strata and on the symbolic log. Where slickensides are observed the direction of the slickenlines should be recorded after the standard discontinuity surface description. The degree of weathering is divided into five categories and is entered on the log in histogram form using the first letter to denote the degree of weathering.

The symbolic log is a graphic representation of the contents of the core box, recording lithologic change, discontinuities, fossils, and mineralization.

2.1.3 Integral Sampling

The engineering geological characterization of coal-bearing strata is incomplete without a detailed investigation of the mine floor. Some coal mine floors are composed of claystone, mudstone, and shales which are inherently weak and subject to rapid disintegration in the presence of drill water. Realizing the necessity of recovering core from the mine floor, several techniques of enhancing core recovery were studied. The integral sampling method proposed by Rocha, 1971, was selected for the following reasons.

- 1) It has been proved successful in civil engineering projects and was able to recover fault gouge, a weak, highly friable material.
- 2) A minimum of equipment is required and the method is easily adapted for use in underground coal mines.
- 3) Integral sampling enables the complete recovery of an oriented core, facilitating structural analysis through detailed study of the discontinuities and any infilling.

Historically, the integral sampling method (ISM) was developed by Manuel Rocha, 1973, at the Laboratorio Nacional de Engenharia Civil in Lisbon, Portugal. Rocha conducted extensive laboratory testing using artificially constructed rock masses. Rock layers, one to twenty centimeters thick, were alternated with silt and clay material, zero to ten centimeters thick to simulate natural infillings in a fractured rock

mass. Drilling was perpendicular to the discontinuities, duplicating the most adverse conditions. In the method used by Rocha, a small hole is drilled into the rock mass, and a bar is oriented and grouted in place. After the bar has set, the surrounding rock is overcored, thus producing an integral sample. Variations in hole diameter, length, grout, bar diameter, type and core barrel were tested. Rocha found that even the very weak, water soluble layers were recovered using integral sampling.

Extensive field tests were carried out at the Albarellas, Aguiera, and Volta Grande dam sites during construction to ascertain engineering geological information concerning the discontinuities. At Albarellas and Aguiera the rock masses consisted of an intensely fractured granite and a schist-greywacke complex, respectively. Integral sampling is highly effective in obtaining 100 percent core recovery in very weak strata. Several faults were intersected and all the gouge material was recovered.

Bieniawski directed a study (Orr et al., 1976) at the Council for Scientific and Industrial Research (CSIR) in South Africa. They designed and built an improved version of the equipment used by Rocha. The apparatus was field-tested in July 1975 and provided excellent results.

Integral sampling was introduced in the United States by Mattson, 1979, who conducted extensive sampling during the extension of the Boston Subway. Excellent core recovery was obtained in a highly fractured argillite. Integral sampling provided the sole means of assessing other strata since conventional drilling provided insufficient core recovery.

Since the integral sampling method has never been used in an underground coal mine, a complete ISM facility was designed and built. Applying ISM in this situation required significant modification due to the height limitations imposed by underground coal mines. The equipment was designed for recovering the immediate 15 feet of floor strata. The procedure, similar to that described by Rocha, is depicted below in Figure 2.

The procedure is characterized by the following steps:

- 1) An EX (1-1/2" O.D.) pilot hole is drilled in the floor to a depth of 3 feet.
- 2) A grooved section of PVC pipe is placed in the hole with sufficient FASLOC "T" (1-1/4" x 12") epoxy resin cartridges to fill the annular space.
- 3) The resin cartridges are broken as the PVC pipe is spun by the drill. The drill steel is removed and the PVC pipe is allowed to harden in the hole.
- 4) The orientation is recorded by matching the PVC pipe and calculating the azimuth using a Clar compass.

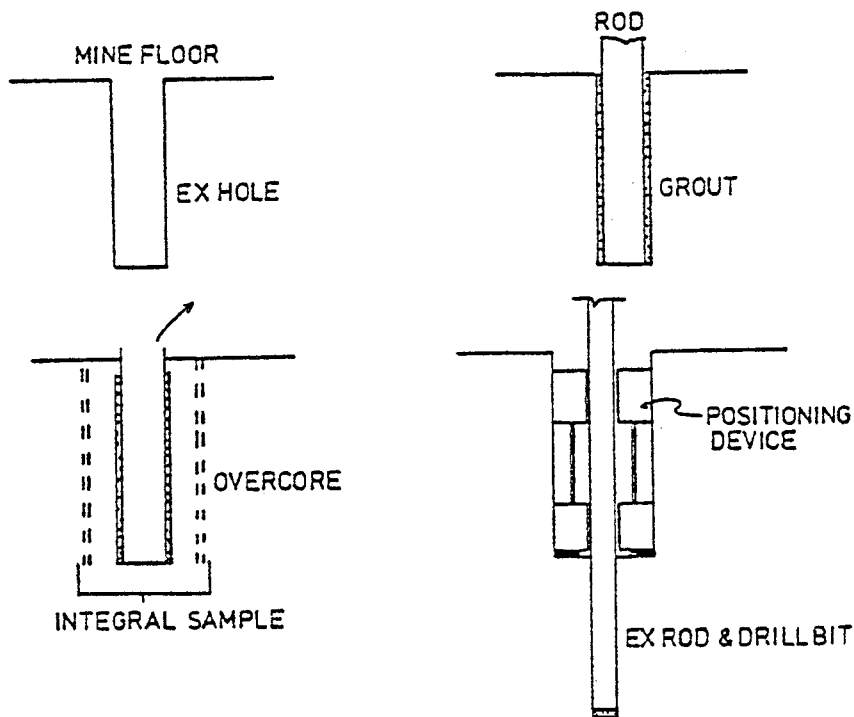


Figure 2. Integral Sampling Technique

- 5) The PVC pipe is overcored using a single tube 5-1/2" core barrel and the integral sample is withdrawn from the hole.
- 6) The EX pilot hole guide is inserted into a hole to insure the pilot hole will be concentric with the overcore.
- 7) The procedures described in steps 1 through 6 are repeated and another integral sample is obtained. This sequence continues until the required depth is attained.

To assess the ISM facility prior to conducting underground trials, a series of integral samples were taken at a strip mine. Since ISM has never been applied to coal bearing strata, the purpose of this initial series of experiments was to test the newly designed facility in typical floor strata. A strip mine owned by the C and K Coal Company located near Blandburg, Pennsylvania on Route 865 was selected as the test site.

The field trials of the integral sampling facility failed due to the inability of the drill to penetrate the floor strata. The machine, the only one available at the Mineral Engineering Department, was too light and could not provide the necessary thrust to drill the EX pilot hole. Since no other drill was available, it was decided to test a

sample of the floor strata in the rock mechanics laboratory. A suitable specimen was obtained from the mine and the integral sampling technique was tested in the laboratory using the above procedure.

The results of the laboratory testing clearly demonstrated the feasibility of integral sampling for obtaining full recovery in coal mine floor strata. Given a robust drill, the ISM used earlier can be easily implemented in the field, since the type of drill was the sole difference in the laboratory investigations. ISM is a useful tool for characterizing coal mine floor strata since it insures the complete recovery of this weak, often fractured material.

2.1.4 Laboratory Material Testing

The behavior of a rock mass is predominantly controlled by the orientation and condition of its discontinuities, not by the properties of the rock material. The need has long existed for a test that would focus on the discontinuities within the rock mass in addition to including the rock material. The fracture index is defined as the square of the ratio of the seismic (field) velocity to the sonic (laboratory) velocity. The elastic wave velocities of the rock mass are recorded and core samples of the rock mass are prepared and taken to the laboratory where the sonic velocities are determined. The field velocity values are influenced by such factors as stress, water content, and discontinuities. When an elastic wave traverses a discontinuity it loses energy and velocity due to incomplete coupling across the aperture. The saturation and stress conditions are simulated in the laboratory, thereby, isolating the effect of the discontinuities. Therefore the closer the field value is to the laboratory value, the higher the rock mass quality. Table 2 illustrates this relationship.

Table 2. Fracture Index and Rock Mass Quality (after Coon and Merritt, 1970).

Fracture Index (V_F/V_L) Rock Mass Quality	
0.2 - 0.4	poor
0.4 - 0.6	fair
0.6 - 0.8	good
0.8 - 1.0	very good

Experimental Equipment

The pulse method was selected for use in this series of experiments on the basis of the following aspects.

- 1) The simplicity of the equipment involved in the velocity determinations.
- 2) Accuracy in determining the elastic wave velocities.
- 3) Ease of sample preparation.

The pulse generator was a General Radio model 1398-A which produced a 300 Hz signal, 80 volts in amplitude. The decay time of each pulse was significantly reduced by matching the impedance of the crystal in the driver transducer. The crystal was assumed to be characterized by a resistor and a capacitor in parallel. An external circuit consisting of a one Meg. ohm resistor and capacitance box in parallel was designed so the impedances of the signal and crystal could be matched. The signal from the receiver is amplified by a PAR 113 Pre-Amp and filtered using a Rockland Analog filter model 1100. The low band pass was 70 kHz and the high band pass 10 kHz. The filter removes electrical noise associated with the operation of the MTS servo-hydraulic testing facility. A Tektronic model T912 storage oscilloscope displayed both the trigger signal and the elastic wave travel time. A CB-5 Polaroid camera pack was used in photographing the oscilloscope display.

Experimental Procedure

The pulse method is based upon the velocities of the compressional and shear waves that pass through the specimen. Since the oscilloscope displays the total travel time, the time required to traverse the transducers must be subtracted to obtain the travel time through the specimen. This was accomplished using magnesium calibration slug A.S.T.M. Alloy (#QQ 31B) whose length, mechanical properties, and approximate sonic velocities are known. Three slugs, 4.500 in, 3.000 in, and 1.501 inches in length were used.

The compressional and shear wave travel times for each slug were plotted on a graph recording travel time vs. specimen length. The relationship was linear as expected and the travel time through the transducers was taken as the y intercept of the line.

The calculated values were based on the known velocities for compressional and shear waves in the magnesium slug, 2.27×10^5 in/sec and 1.22×10^5 in/sec, respectively. It required 2 μ sec for a pulse to traverse the compressional transducers and 3 μ sec for the shear transducer. The elastic constants of the slugs were calculated, using the equations below and compared with the known values.

$$G = \rho V_s^2 \quad (1)$$

$$\mu = \frac{(V_p^2 - 2V_s^2)}{2(V_p^2 - V_s^2)} \quad (2)$$

$$E = 2G (1+\mu) \quad (3)$$

$$E = \text{dynamic modulus of elasticity (lbf/in}^2\text{)}$$

G = shear modulus (lbf/in²)

μ = Poisson's Ratio

σ = density

V_p = velocity of compressional wave (in/sec)

V_s = velocity of shear wave (in/sec)

Initially, the specimen and transducer were coupled with phenyl salicylate. A drop of molten phenyl salicylate was placed on the specimen and the transducer was immediately positioned on top of the specimen. The bond provided an excellent transfer of energy in a unloaded condition. When the specimens were loaded in the MTS facility, the integrity of the bond was limited by the compressive strength of phenyl salicylate. As a result, high vacuum grease was substituted as a coupling medium when loading the specimens. It proved effective in transferring energy across the interface, but required its own calibration to insure accurate results.

Blocks of Indiana limestone 10 in x 10 in x 7 were ordered from a quarry in Bedford, Indiana. The blocks were cored perpendicular to the bedding plane using the laboratory drill and an NX diamond coring bit. The cores were cut and polished to 0.001 inch to insure a flat surface for the transducers and parallelism between the two faces. In accordance with I.S.R.M. specifications, a length to diameter ratio of 2.5 was used. Strain gages (SR-4) were placed on each core to measure axial and transverse strains. Three sets of rosettes (each rosette is composed of two strain gages oriented 90 degrees apart) were placed 120 degrees on each specimen.

The specimens then were placed in the MTS facility and the compressional and shear wave velocities were determined at uniaxial pressures of 0, 1000, and 2000 lb/in². This procedure was repeated after soaking the cores in water for 48 hours.

Discussion of Results

This series of sonic velocity experiments provided the velocities of the shear and compressional waves under varying conditions of stress, water saturation and specimen size.

Field investigations were conducted at Mines I (coal) and II (limestone) for the purpose of comparing rock mass quality assessments using the Geomechanics Classification and the Fracture Index. Seismic (field) velocities were determined by instrumenting a pillar with geophones and a 12 channel seismograph (Belesky, 1981). The elastic waves were propagated through the pillar by striking a steel source plate with a sledge hammer. Table 3 summarizes the average sonic and seismic velocities.

Table 3. Results of Elastic Velocity Determinations on Coal and Limestone

		<u>Mine I</u>	<u>Mine II</u>	
V_p	(in/sec)	1.053×10^5	2.354×10^5	SONIC VELOCITY DATA
V_s	(in/sec)	6.31×10^4	1.48×10^5	
V_p	(in/sec)	4.967×10^4	1.594×10^5	SEISMIC VELOCITY DATA
V_s	(in/sec)	2.980×10^4	1.004×10^5	

Using the compressional wave velocities the Fracture Indices follow:

	<u>Fracture Index</u>	<u>Rock Mass Quality</u>
Mine I (coal)	0.47	fair
Mine II (limestone)	0.68	good

The area surrounding each pillar was also classified according to the Geomechanics Classification System.

	<u>RMR</u>	<u>Rock Mass Quality</u>
Mine I (coal)	55	fair
Mine II (limestone)	84	very good

2.2 Assessment of Roof Spans by Rock Mass Classifications

The approaches employed to date for mine roof stability assessment can be categorized as follows:

- Analytical Methods
- Geological Methods
- Observational Methods
- Empirical Methods

Analytical approaches generally utilize the fundamental concepts of strength of materials, continuum mechanics, structural analysis, and soil mechanics.

As indicated by Adler and Sun (1976), the immediate roof has been analyzed employing beam and plate theories as early as 1905. The concepts of cantilever beam, simply supported beam, and clamped or fixed-end beam were considered as analytical models for the behavior of

immediate roof. In cases where length of roof is less than twice its width, a three-dimensional problem is encountered which can be analyzed by more complex plate theory. The concept of "voussoir arch" was introduced to explain the stability of mine roofs acting as beams containing significant cracks. The "pressure arch" theory was proposed as early as 1911. The British National Coal Board has proposed empirical formulas to calculate the maximum width of the pressure arch. The "dome theory" was also evolved from wide-spread observations. The dimensions of a dome-shaped space around underground cavities were found to fit an empirical formula. Wright (1973) has reviewed the application of beam action and arching of the roof in a roof control program.

In recent years, numerical methods have been widely used in solving the problem of underground structure stability. Finite element method, boundary element method, finite difference method, analog simulations and other numerical modeling techniques have been employed in numerical approaches. The application of numerical methods in stability assessment of coal mines has been steadily growing but has not reached a stage where the methods can be considered as design approaches of their own, because of the difficulties in providing the input data.

Geological Approaches, basically, seek the cause-effect relationship between the geological features and the structural failure. It is important to know the characteristics of these features which range from small fossils or crystal inclusions to large regional structure features such as folds and stream washouts (Peng, 1978). The geological irregularities or anomalies in the coal seam overburden, known as causes of roof falls in some types of failure, can be divided into underground and surface features.

Observational Approaches rely on monitoring of movement and detecting instability in the mines. Although considered an independent method, observational approach is the only way of checking the results and predictions of the other methods. In fact neither theory nor model studies provide the degree of reality that is required for understanding the stability of structures in rock, without the in situ measurements.

2.2.1 Roof Strata Classifications

Engineering classifications of rock masses are widely recognized as an integral facet in the designing of underground openings in rock (Barton, 1976, Bieniawski, 1979, Hoek and Brown, 1980). The primary objective of a rock mass classification scheme is to provide a quantitative assessment of rock mass quality based on well-defined engineering geological parameters. The parameters are easily determined in the field and are empirically weighted by their observed relationship to the stability of an underground opening. An effective classification scheme enables the engineer to subdivide the rock mass into classes of similar behavior. The classes are established on the basis of case histories which embody a wide variety of geological structure, lithology, ground support and engineering application. Rock mass classification schemes allow the engineer to relate experience gained in similar rock mass

conditions at other sites and thus estimate the stability and the support requirements needed at a specific underground excavation.

Another objective of a rock mass classification is to facilitate the planning and design of underground openings in rock. This task is accomplished in three ways. First, the input parameters ensure the compilation of pertinent information concerning the geologic structure and mechanical properties of the rock mass during exploration. This is essential to a thorough understanding of the rock mass. Second, the geological parameters are quantified and combined with the mechanical properties so they can be directly integrated into an engineering design.

The input parameters include:

- Uniaxial compressive strength,
- RQD, (Rock Quality Designation) of borehole core
- Joint spacing, condition, and orientation,
- Groundwater conditions.

The input parameters are obtained through examining exploration core and engineering geological mapping. The output of a classification scheme is in terms of span versus stand-up time and is accompanied by ground support guidelines for each rock mass class. Third, the classification provides a common basis for clear communication between the engineer, geologist, contractor, and owner throughout the planning, design, and excavation of the underground opening.

Historical Development of Rock Mass Classification

Rock mass classification schemes were initially applied to near surface civil engineering tunnels. Since the early tunnels were designed by structural engineers, emphasis was placed upon the estimation of rock loads. Later, as the geotechnical aspects of tunnel design were accentuated, classification evolved to include support recommendations for either wood or steel. This development culminated in the modern format which integrates the span, stand-up time, and provides support guidelines. The guidelines utilize modern ground support techniques including rockbolts, steel mesh, steel arches, and shotcrete. Since extensive reviews detailing the theory, methodology, and application of specific classifications are available (Bieniawski, 1973, 1977, Barton, 1974, Steiner and Einstein, 1980, Hoek and Brown, 1980) in addition to the original references, the significant aspects of each scheme are presented here in their relation to the evolution of rock mass classifications.

Early Attempts to Estimate Rock Loads on Tunnels

Ritter (1879) proposed equations to quantify rock loads in circular and flat roofed tunnels. However, his method considers only gravity forces in the roof and wedge failure in the sidewalls. Ritter discusses the limitations of his method and emphasizes the need to monitor tunnel

behavior during construction. This statement is significant since it is essential to even the most sophisticated classification scheme.

Bierbaumer (1913) developed a method for rock load determination based on observations of timber support behavior. Laboratory tests were conducted to determine the load-deformation behavior of timber support. In the field, rock loads were determined through back calculation of the observed support deformation. Bierbaumer established the first rock mass classification scheme by defining five ground support classes. The classes were designated by rock load and geological conditions. Support guidelines were provided for primary timber support and for the final liner of masonry and dimensional stone. Bierbaumer's classification scheme provides the basis for subsequent rock mass classifications and he clearly achieves a balance between theory and empiricism.

Kommerell (1940) proposed a method of determining rock load based on the settlement of the tunnel crown and subsequent loosening of the overlying strata. Kommerell's contribution is the quantification of squeezing ground and the estimation of sidewall support requirements. This foreshadowed the later work of Terzaghi (1946).

Modern Rock Mass Classification Schemes

Terzaghi (1946) devised a rock mass classification based on rock load which marked the beginning of modern rock mass classification schemes. He outlined nine classes of rock mass behavior and related the rock load to: i) intensity of jointing; ii) tunnel dimensions; iii) water pressure; iv) capacity for self support on the basis of arching experiments in sand (Terzaghi, 1936), and v) overbreak during construction. Terzaghi's classes differentiate rock mass behavior on the basis of loosening, squeezing, and swelling. The Terzaghi method is supplemented by extensive steel set support recommendations proposed by Proctor and White (1946).

Stini (1950) proposed a method of estimating rock loads in a "standard" tunnel, four to five meters in width. The method is similar to that of Terzaghi since it relates rock mass behavior to lithology. Stini emphasized the importance of discontinuity orientation in relation to the direction of tunneling. The estimated rock load for the "standard" tunnel is corrected for a specific case through adjusting the load value by the percent for each meter difference in width between the actual and "standard" tunnel dimensions. However, since only timber support is considered, the method is limited in application.

Lauffer (1958) introduced the concepts of unsupported span and stand-up time. Through the two concepts, the time-dependent instability of tunnels was addressed in a classification scheme. Eight classes of rock mass behavior were correlated with unsupported span and stand-up time. Lauffer (1960) provided support recommendations for each class, utilizing timber, steel sets, wire mesh, rock bolts, and shotcrete. Lauffer isolated four factors which influence tunnel stability:

- 1) the orientation of geological structure,
- 2) shape of the tunnel cross-section,
- 3) type of excavation,
- 4) support procedure.

Although Lauffer's method lacks specific input data and is primarily based on one case history, the method had a profound influence on the direction of subsequent rock mass classification schemes. The relation between unsupported span and stand-up time served as the basis for later classifications proposed by Wickham, Tiedemann, and Skinner (1972), Bieniawski (1973), and Barton, Lien, and Lunde (1974). The Lauffer scheme utilizes five methods of rock reinforcement in the support guidelines. This feature has been incorporated in later classification schemes.

Deere (1963) proposed the Rock Quality Designation (RQD) to provide a quantitative index of rock mass quality. The RQD index is based upon the length of core recovered as intact pieces greater than 100 mm divided by the total length of the drilling run. The index is expressed as a percentage and is correlated with five classes of rock mass quality. Cording, Hendron, and Deere (1972) formulated a relationship between the modified Terzaghi rock load and the RQD index. Cording and Deere (1972) investigated wedge stability in tunnel sidewalls correlating the rock load factor with an equivalent angle for wedge failure. In this method, the effects of joint spacing, orientation, strength of the intact rock, and groundwater are not taken into account.

The concepts initiated by Lauffer were elaborated upon by Wickham, Tiedemann, and Skinner (1972) who proposed the Rock Structure Rating (RSR) concept. The RSR method assesses tunnel stability from three aspects:

- 1) lithology and geological structure,
- 2) spacing and orientation of joints,
- 3) the effect of groundwater inflow.

A correction for machine bored tunnels is provided reflecting new tunnel excavation methods. The RSR concept is highly significant to the evolution of classification schemes because the input parameters are weighted according to their influence on tunnel stability.

Bieniawski (1973) proposed the Geomechanics Classification which assesses rock mass quality on the basis of six field determined parameters.

- 1) Uniaxial compressive strength of rock material
- 2) RQD
- 3) Discontinuity spacing
- 4) Condition of the discontinuities
- 5) Orientation of the discontinuities in relation to the tunnel direction
- 6) Groundwater conditions

The parameters are weighted by their observed influence on tunnel stability. The input data requirements achieve a balance between the discontinuity characteristics and the mechanical properties of the intact rock. The output is in terms of unsupported span versus stand-up time, and the scheme provides support guidelines incorporating steel sets, wire mesh, shotcrete, and rock bolt support. The Geomechanics Classification contributed the following aspects to the evolution of classification schemes.

- 1) Incorporation of rock material properties
- 2) Reliance on field measurements
- 3) Output based on case histories encompassing a wide range of rock mass conditions
- 4) Versatility in its application to tunnels, slopes, and foundations.

The Geomechanics Classification has been utilized in tunnels, underground hard rock mines (Laubscher and Taylor, 1976, Ferguson, 1979) and underground coal mines (Djahangiri, 1977, Rafia, 1980, Ghose, 1981). An important aspect in regard to the accuracy of the classification is the continued revision of the rock mass rating (RMR) and parameter weighting (Bieniawski, 1974, 1976, 1979) on the basis of additional case histories. This refinement is essential to the improvement of empirical methods.

Barton et al. (1974) devised a rock mass classification scheme known as the Q-System. The method is similar to the Geomechanics Classification in format, but it emphasizes different rock mass characteristics. The six input parameters selected to describe the rock mass follow:

- 1) RQD
- 2) J_n = joint set number
- 3) J_r = joint roughness number
- 4) J_a = joint alteration number
- 5) J_w = joint water reduction factor
- 6) SRF = stress reduction factor.

The Q-value is computed by the following equation:

$$Q = (RQD/J_n) \cdot (J_r/J_a) \cdot (J_w/SRF).$$

The Q-System considers discontinuity shear strength and the in-situ stress field. This reflects the intended application of the Q-System in the commonly higher quality massive, igneous rock that is predominant in Scandinavia. This difference in application is illustrated in Figure 3, where the Geomechanics Classification appears as the more conservative of the two methods. The higher quality igneous rock in Scandinavia

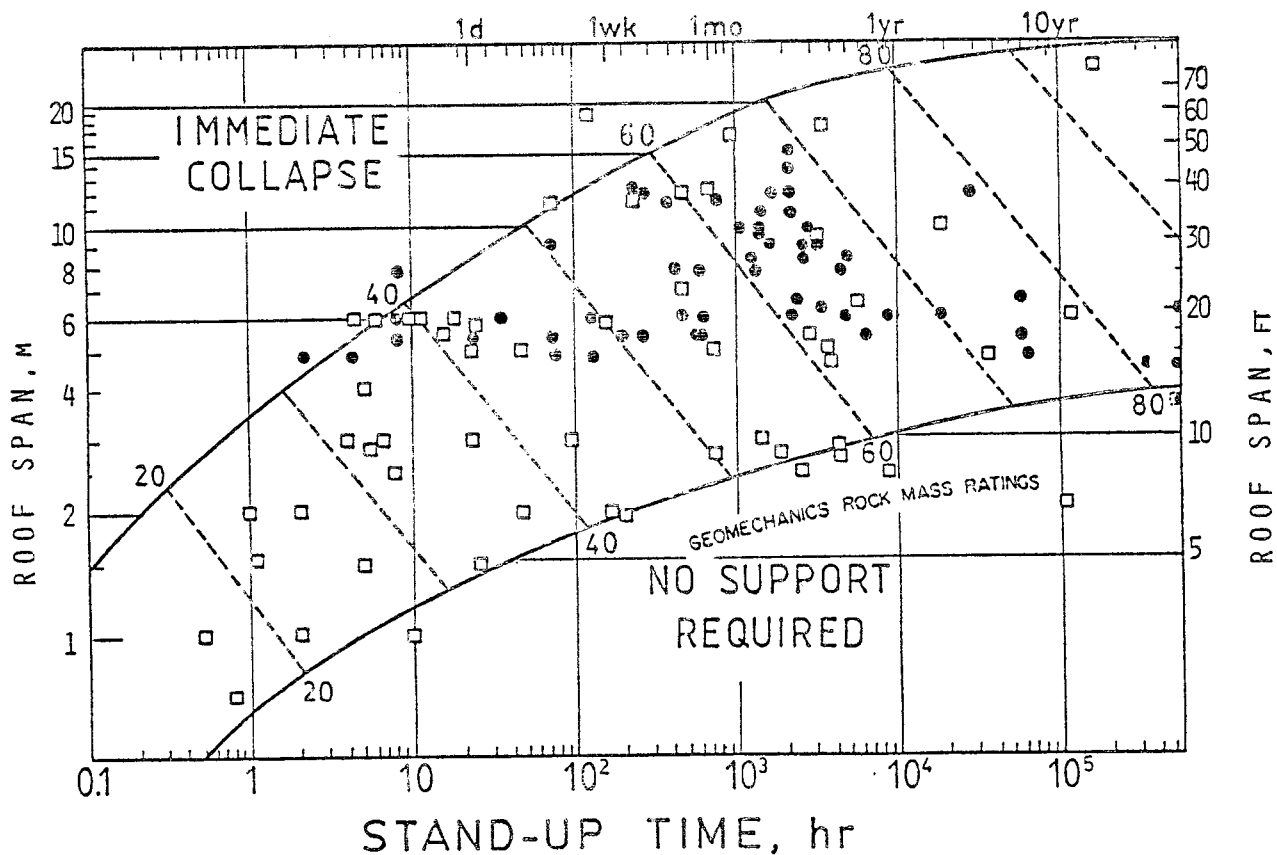


Figure 3. A Comparison of Output Data from the Geomechanics Classification and the Q-System (Bieniawski, 1979).

provides a longer stand-up time for a given span than the sedimentary rock upon which the Geomechanics Classification is based.

Louis (1974) proposed a method which utilizes four input parameters: i) joint spacing, ii) compressive strength of the rock, iii) overburden stress, and iv) tunnel diameter. The joint spacing is described both by RQD and the median fracture spacing. The excavation method is determined from a plot of RQD versus compressive strength. Five ground classes are specified from a graph in which normalized fracture spacing (mean spacing/diameter) is plotted against the normalized compressive strength (uniaxial compressive strength/3 x the overburden stress). Support guidelines are included for each ground class. Water pressure, permeability, and swelling pressure are also

considered in the determination of the ground class. Discontinuity strength and orientation are not considered by this classification scheme.

A similar classification was developed by Franklin (1975-1976) in which four parameters are examined: uniaxial compressive strength, block size (fracture spacing), opening dimensions, and the overburden stress. On the basis of these parameters, a diagram is presented to evaluate both tunnel stability and potential failure mechanisms. An important feature of Franklin's method is a graphical procedure using a stereonet used to assess local instability related to discontinuity orientation and shear resistance.

Application of Rock Mass Classifications to Underground Mines

Hard Rock Mines

Rock mass classification was used extensively for stability assessment in Rhodesian asbestos mining. Since block caving is the predominant asbestos mining method, rock mass classification serves a dual purpose. First, it evaluates the cavability of the orebody and second, it assesses the stability and support requirements for long-term structures such as ore draw points and main haulageways. Furthermore, the Geomechanics Classification was selected since it could be directly integrated into normal mine geological investigations (Laubscher and Taylor, 1976).

For purposes of asbestos mining percentage adjustments are provided for the effects of weathering, in-situ stresses, change in stress due to mining, orientation of discontinuities, and the effects of blasting.

Laubscher, 1977, describes appropriate support guidelines using rockbolts, wire mesh, and shotcrete for long-term structures such as chambers and pillars. Ferguson, 1979, outlines the role of the modified classification in the design process for asbestos mines.

The benefits of rock mass classification to underground hard rock mining are demonstrated clearly by the above authors. Classification is routinely used as a facet of engineering geological investigation. Currently a classification scheme specifically designed for hard rock mining in the United States is being developed by Engineers International Inc. under a grant from the U.S. Bureau of Mines.

Underground Coal Mines

Qualitative Classifications

Recently several rock mass classification schemes have been proposed for use in underground coal mines. These classifications are separated into two categories--qualitative and quantitative schemes. Hylbert, 1978, proposed a classification of the former variety. Geologic structure and lithology are correlated with mine roof stability. Hylbert identified four classes of roof behavior at the

Highsplint mine in eastern Kentucky. The data was collected by detailed underground geological mapping and depicted on an outline of the mine workings. By supplementing this data with geological information from exploration boreholes, Hylbert is able to predict roof stability prior to mining. Support recommendations are provided for each class. The recommendations are based upon the support procedures which had been effective for a particular class in the past. Although Hylbert demonstrates the validity of his method, it is far too limited for general application. The method is mine specific and would require extensive geological investigation to establish a similar classification in other mines.

Patrick and Aughenbaugh, 1979, proposed a scheme for classifying roof falls according to their physical geometry and extent. Sixteen classes are described and an attempt is made to correlate the classes with specific locations in the mine. For example, the authors note that dome-shaped roof falls occur almost exclusively in four-way intersections. Although the magnitude of the horizontal stress field was not measured directly, the presence of tensile fractures in the roof is used as a criteria for assessing a tensile stress field. The presence of tensile fractures is plotted against the relative frequency of roof falls.

Quantitative Classifications

The integration of quantitative rock mass classification schemes into underground coal mine design has only recently been attempted. Djahanguiri, 1977, applied the Geomechanics Classification for the design of a thick seam longwall mine in Wyoming. Urug and Szwilski, 1980, devised a rock mass classification, the Roof Quality Index (RQI), specifically designed for longwall mining. The RQI is used to evaluate the feasibility of longwall mining through the assessment of the roof caving characteristics and the loading conditions at the coal face.

Three classification schemes, the Geomechanics, the Q-System, and RSR were employed in a rock mechanics study of coal mine roof conditions in a section mined by the automated extraction system (AES)(Bieniawski, et al., 1980). The study demonstrated the effectiveness of the Geomechanics Classification in assessing roof conditions and presented potential modifications for application to underground coal mining. Rafia, 1980, proposed a series of modifications that address the weatherability of coal strata, the relations between the parameters of RQD and fracture spacing, and the effects of a high horizontal stress field. Ghose and Rajer, 1981, applied the Geomechanics Classification to Indian coal mines citing its potential for the evaluation of roof stability.

2.2.2 Modification of the Geomechanics Classification for Coal Mining Applications

Engineering rock mass classification have always been regarded as developing techniques. In order to improve rock mass classification

applicability for coal mines, certain modification to the Geomechanics Classification will be considered. From the three classification systems applied, the Geomechanics Classification was selected for this purpose because this classification system has already made important contributions to mining in a wider range of applications.

In this study, three changes in the rating values are suggested for coal mine applications. The slake-durability index has been considered as one parameter, contributing to the classification of rocks in the coal mines. Secondly, since the RQD and spacing of the discontinuities in a rock mass are interrelated, the corresponding ratings have been combined to form one rating value instead of two separate ones. In addition, the field and induced stresses have been regarded as important factors in appraisal of roof stability in coal mines. Therefore, the effect of stress has been taken into consideration.

Modification for Weatherability

The slake-durability index has been developed for providing a quantitative measure of the relative breakdown from drying and wetting of argillaceous rocks. Durability tests predict the effects of weathering agents with short-term engineering significance. Of these, climatic slaking is undoubtedly the most widespread and therefore the most important. A slake-durability test predicts deterioration due to climatic wetting and drying.

Among the parameters of the Geomechanics Classification, three of them, namely RQD, strength of rock material and condition of joints are affected by the weathering process (Laubscher and Taylor, 1976).

The RQD can be decreased by an increase in fractures as the rock deteriorates and the volume increases due to humidity effect. The strength of rock material will decrease, slightly, in better quality rocks such as hard shale, and highly in weaker rocks such as claystone. The main influence of the deterioration due to wetting and drying process is on the condition of the discontinuities by alterations, either of the joint discontinuity, wall rock, and/or the joint filling.

In order to take into consideration the strength of the roof rocks of the coal mines against the wetting and drying process it is suggested to use the standard slake durability index (Id_2) as a multiplier for the corresponding rating values found for RQD, strength of rock material, and condition of major discontinuities.

RQD and Spacing of Discontinuities

The Rock Quality Designation (RQD) is defined as the proportion of borehole core consisting of intact lengths equal to or longer than 4 inches (10 cm). To obtain the RQD value of a given rock it is not always possible to conduct a core drilling program. To determine the RQD of rock surfaces a tape is suspended along the surface and an assessment of the true fracture spacing is obtained. This then may be regarded as directly analogous to a borehole core.

Priest and Hudson (1976) presented a theoretical approach to discontinuity spacing and RQD based on the statistical distributions of spacing values that could occur along scanlines. The discontinuity spacings have been considered with reference to the distances between points where discontinuities intersect a straight line through the rock mass. They assumed the discontinuity intersection points could be evenly spaced, clustered, random or some combination of these. The most probable type of discontinuity spacing distribution was proved to be the negative exponential distribution. They verified the validity of this assumption from the available experimental data. They found a linear relationship between the minimum possible RQD value (RQD_{min}) and the discontinuity frequency as follows:

$$RQD_{min} = 100 (1 - 0.1\lambda + 0.1/L)$$

where:

L = scanline length

λ = mean number of discontinuities/meter

If the scanline is long enough (comparable to 0.1m):

$$RQD_{min} = 100 (1 - 0.1\lambda)$$

They also found the following relationship between the theoretical maximum RQD, (RQD^*), and discontinuities frequency (λ), when the discontinuities spacing follow a negative exponential distribution:

$$RQD^* = 100 e^{-0.1\lambda} (0.1\lambda + 1)$$

On the other hand, the Geomechanics Classification of rock masses, regards six parameters of importance, including both RQD and discontinuity spacing. Since these two parameters are related to each other, one can put them together and assign a rating value for the combined effect.

Figure 4 was developed by changing the values in accordance with the new rating system of the Geomechanics Classification (Bieniawski, 1979A). The dotted lines in Figures 4 are the average data with a single value. The usefulness of these lines is that a proper rating (given in squares) can be allocated to RQD and/or discontinuity spacing when only one of these two parameters of importance is known.

Modification Due to State of Stress

As was mentioned earlier, the Geomechanics (RMR) rock mass classification does not count for the in-situ state of stress. Field and induced stresses can have an influence on the condition of joints by maintaining the joint under compression, by increasing the possibility of a shear movement or by allowing the joints to open. Laubscher and Taylor (1976) suggested an increase of basic rating up to 120% in the

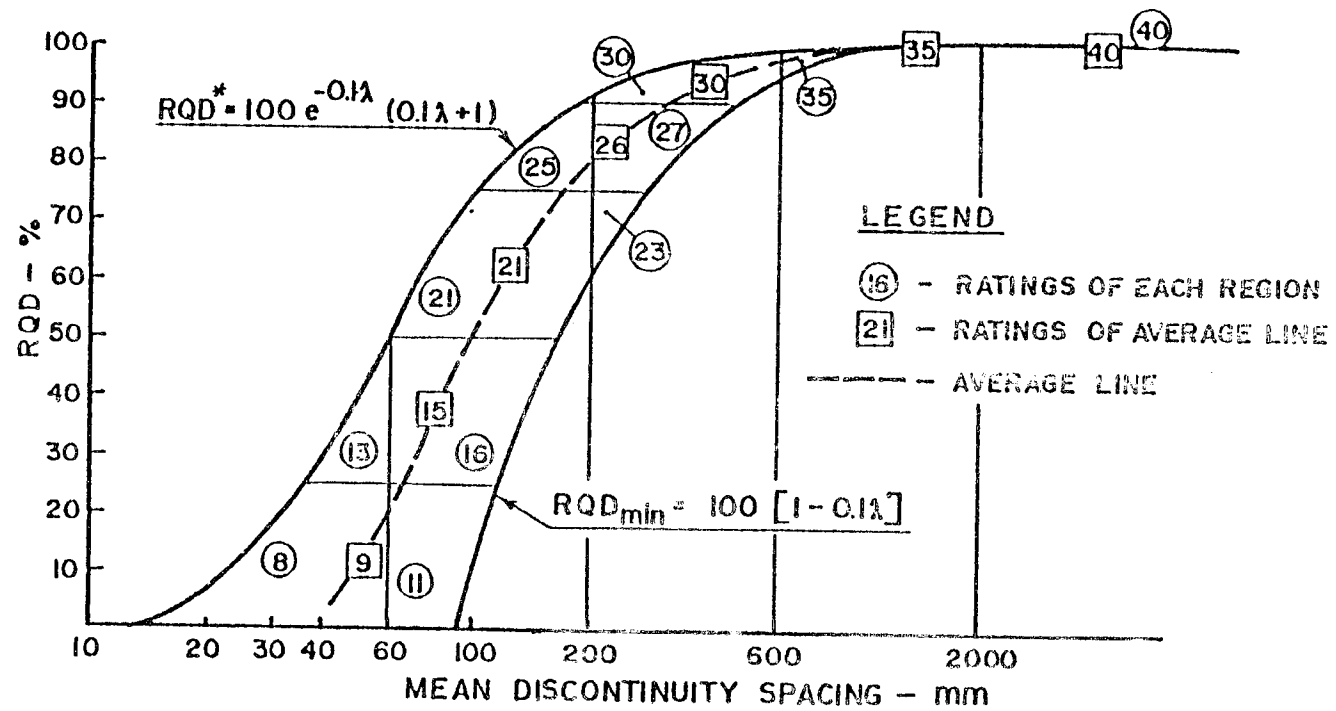


Figure 4. Relationship between RQD and discontinuity spacing.

most favorable state of stress. The rating should be decreased to 90% if the possibility of shear movement is increased. If the stress causes the joint to open the rating is decreased to 76%. Therefore, according to Laubscher a total adjustment of 76 to 120% due to state of stress is possible. Although this adjustment is mainly applicable to large excavations where two of the dimensions are significantly different from each other, it will apply to small openings in exceptional cases. However, to prove the validity of the proposed adjustments for underground coal mine openings, it will be required to conduct a detailed study in conjunction with field tests.

Considerations of Litho-Structure of Immediate Roof

If the strata immediately above the coal seam comprise a massive strong rock, no problem will be encountered when classifying the roof rock. A problem arises when the immediate roof consists of a variety of layers with various thicknesses and characteristics. There are also areas of disturbances due to presence of geological anomalies ranging from small fossils or crystal inclusions to large regional structural features such as folds and stream washouts (sandstone channels).

When applying the Geomechanics Classification to the immediate roof, the best method is to divide the immediate roof into structural units (mostly by lithology), to apply the classification for each structure, and finally, to find the weighted average rating considering the thickness of the units. The rating value found by this method, is then is allocated to roof quality.

2.2.3 Collection of Roof-Fall Data

One of the problems in applying rock mass classifications to coal mining is the lack of data on the roof span versus stand-up time relationship. Consequently, a study was undertaken the main objectives of which were as follows:

- to collect the data from roof falls featuring the roof span and corresponding stand up time for establishing a base of case histories,
- to apply the Geomechanics Classifications to test its accuracy.

For the purpose of this investigation, a "roof fall data sheet" was developed.

Dimensions of the roof falls studied ranged from small (about one foot) to very large (400 feet), with different shapes such as arch, dome, dome with plane top, and rectangular (cutter type). Out of 47 recognized roof fall cases, the dates of fall occurrence for 43 cases were available.

Span Versus Stand-up Time Considerations

The study was primarily directed to assess the roof falls data concerning the span of the openings versus their stand-up time. The data were carefully assessed. The time between the date of excavation and the date of roof failure was defined as the stand-up time of the roof. The stand-up time data were obtained for 30 cases. These cases were plotted in Figure 5, as an addition to the graph of span versus stand-up time, proposed by Bieniawski (1973).

2.3 Prediction of Stable Roof Spans

For the purposes of estimating stable roof spans in room-and-pillar coal mines, the Geomechanics Classification has been developed to a stage that it can be applied with much confidence in U.S. mines. Not only some 60 coal mine case histories were collected during this research to substantiate the relationship given in Figure 5, but research currently conducted in the United States and Canada is directed to extend this approach even further (roof support guidelines) and will result in many more case histories, both in coal mining and in metal mining.

Detailed procedures for employing the Geomechanics Classification as well as practical examples of its use are given in Chapter 5, Section 5.2.

The Geomechanics Classification could be improved in the future use if the following three aspects were addressed:

- (i) reducing the number of classification parameters, thus simplifying the classification procedures;
- (ii) including the effects of field stresses as an additional adjustment to the rock mass ratings (RMR);
- (iii) correlating roof support requirements with roof rock classes (RMR values).

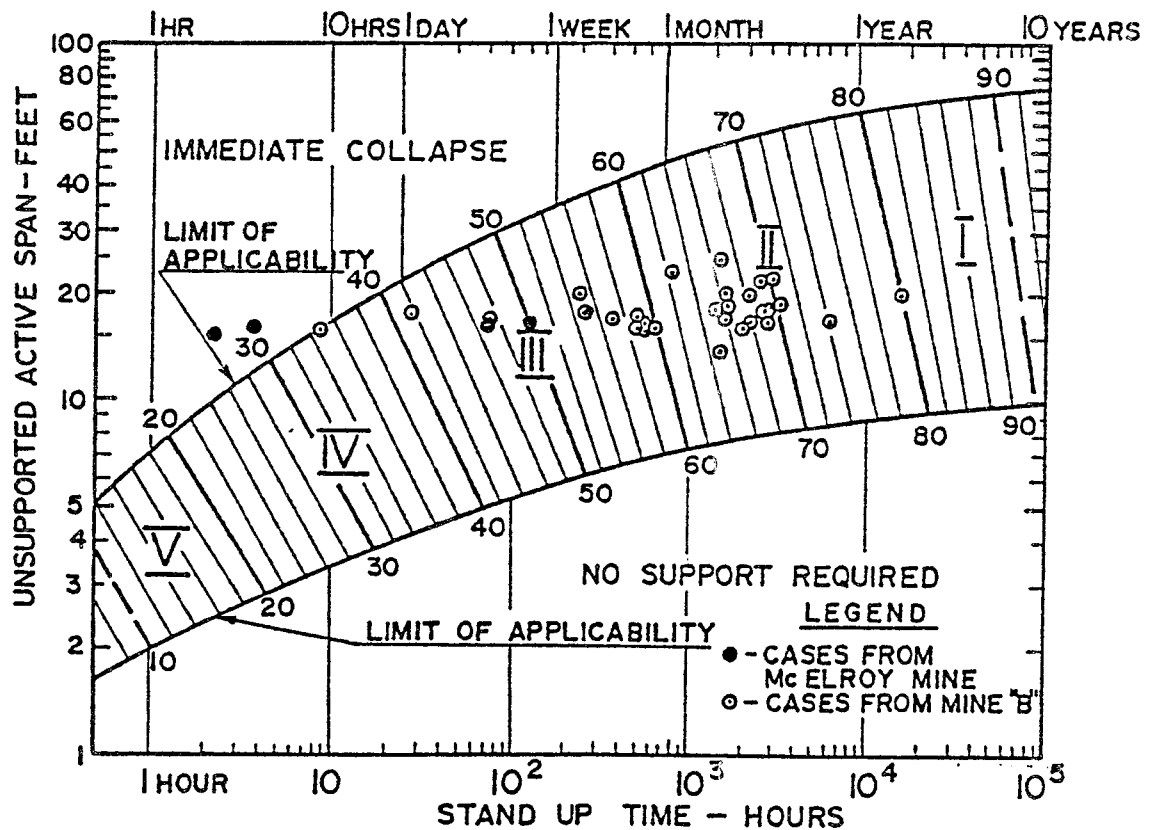


Figure 5. Stand-up time versus roof span for coal mine roofs.

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CHAPTER 3

STRENGTH AND DEFORMABILITY OF COAL PILLARS

Mine-pillar design depends on providing an adequate factor of safety for structural stability, defined as:

$$S.F. = \frac{\text{Predicted Strength}}{\text{Predicted Stress}}$$

Consequently, both pillar-load and pillar-strength assessments are primary areas of consideration.

3.1 Literature Review

Two types of design philosophies exist for assessing pillar stability (Bieniawski, 1980). The first is the "ultimate-strength approach" which states that pillar failure occurs when the load reaches the ultimate strength of the pillar. Two commonly made assumptions in this approach are:

- 1) an average pillar stress or load exists across the pillar;
- 2) a uniform, ultimate strength is derived by scaling uniaxial strength of laboratory specimens.

The second is the "progressive-failure approach" which states that pillar failure initiates at some critical point and may propagate throughout the pillar. Overall stability is maintained despite the local failure. In this approach, two assumptions are made that more closely represent actual mine conditions:

- 1) a non-uniform pillar load exists across the pillar,
- 2) a non-uniform pillar-strength profile resulting from pillar-core confinement is assumed.

In the ultimate-strength approach, a global or average safety factor is obtained for the entire pillar, whereas, in the progressive-failure approach, local safety factors are the prime aim of the calculations.

The global safety factor is based on an average pillar strength and load; consequently, it is constant across the entire pillar. In contrast, the local safety factor relies on the pillar strength and load at a given point. Since pillar strength and load vary across an actual pillar, a local safety factor better represents reality. However, the progressive-failure approach and, consequently, the local safety factor concept have not been implemented in the United States. Title 30 of the Code of Federal Regulations (CFR), Paragraph 75-200, states that ribs

not be on the verge of incipient failure. Herein lies the reason for non-acceptance of the progressive-failure approach as the philosophy is based on incipient failure along the pillar rib. Thus, despite an adequate global safety factor, the inadequate local safety factor is in violation of the mining law. Consequently, the ultimate-strength approach is the currently accepted pillar-design philosophy because it is acceptable to mining law.

3.1.1 Pillar-Load Considerations

Two major pillar-load theories are available in the literature. They are the tributary-area theory and an elastic-deflection theory.

Tributary-Area Theory

In the United States, pillar load or average pillar stress is accepted as conforming to the tributary-area theory (Duvall, 1977; Peng, 1978; Bieniawski, 1980). This theory expresses pillar load for rectangular pillars in the following manner:

$$S_p = \frac{1.1 H (w+B) (L+B)}{w \times L} \quad (3.1)$$

where

S_p = pillar load or average pillar stress (in psi),

H = overburden depth (in feet)

w = pillar width (in feet)

B = entry width (in feet)

L = pillar length (in feet).

The term $1.1 H$ can be replaced by the virgin vertical pressure, S_v , for a more general expression. The term S_v is due solely to the overburden weight above the seam, γH , where γ is the unit weight of the overburden. Since the average unit weight of sedimentary rock is 160 lb/ft^3 , the pressure can be considered to increase at 160 lb/ft^2 per foot or 1.1 lb/in^2 per foot of depth.

This theory incorporates the following assumptions:

- 1) The seam is subjected only to vertical pressure that is constant over the mined area. However, stress transfer occurs where stiff abutments exist in underground workings. Thus, this vertical pressure may be relieved partially.
- 2) Each pillar supports the rock column above the pillar plus

an equal share of the rock above the entry. This implies that the working geometry is uniform, the mined area is greater than the overburden depth, and the structural elements are rigid and homogeneous.

- 3) The load is uniformly distributed across each pillar. However, research has shown that stress is distributed non-uniformly across a pillar (Wang, et al., 1976). Thus, spalling ribs are not indicative of complete pillar failure as this theory would imply when combined with an ultimate pillar-strength value.

Without question, the assumptions made in the formulation of this theory lead to its conservative estimate of pillar load. Therefore, it represents the upper limit of the average pillar stress (Bieniawski, 1980). The simplicity and conservatism of this theory lead to its present popularity.

Elastic Theory

Mining researchers (Berry and Sales, 1961; Salamon, 1964; Coates, 1965; Salamon, 1967; Crouch, 1972; Starfield and Wawersik, 1972; Oravecz, 1977) support the concept of a pillar-load theory based on considerations of the theory of elasticity. The most important consideration is that of seam compression between a relatively stiff roof and floor. Coates (1965) states that the tributary-area theory overestimates pillar load. His complicated equation is listed in Table 4 for incremental loading due to mining. The value of $\Delta\sigma_{pl}$ is then entered into $\sigma_{pl}/S_o = \Delta\sigma_{pl}/S_o + 1$ to obtain the total pillar stress. S_o represents the virgin vertical stress acting on the seam. Coates' elastic solution results in average pillar loadings that are 40 percent lower than the tributary-area-theory values. Elastic deformations of the structural elements produce this stress reduction.

Oravecz (1977) also studied pillar loading with the assumption of elastic behavior. The study used an electrical-resistance analog model to simulate seam convergence and obtain pillar-load distributions. The results of three field-convergence comparisons with the model predictions revealed good correlation of pillar-load magnitudes and distributions with an appropriate in-situ elastic-modulus selection. These field tests consisted of convergence measurements using a specially designed, surface-mounted extensometer that extended down to the seam. This system enabled Oravecz (1977) to measure very small displacements which often exist in coal mining. Oravecz outlined modeling assumptions which affect the applicability of this theory. These assumptions are:

- 1) the analog solution is based on the existence of a homogeneous medium;
- 2) the mine-panel geometry must be uniform;

Table 4. Coates' Elastic Pillar-Load Formula

$$\sigma_{pl}/S_o = \Delta\sigma_{pl}/S_o + 1$$

where σ_{pl} = total pillar load

S_o = virgin vertical stress

and $\Delta\sigma_{pl}$ = pillar load due to mining which is described as:

$$\Delta\sigma_{pl} = \frac{2R - k h (1-w) (\sqrt{1-x^2} + h) - w_0(khn)}{hn + \pi (1-R) (1+1/N) (1+h/\sqrt{1-x^2}) / 2 + 2R b' (1-w) \pi}$$

where, for plane strain:

$$M = E/(1-\nu^2)$$

$$b' = b/L$$

$$w = \nu/(1-\nu)$$

$$x = x^*/L^*$$

$$k = \sigma_h/\sigma_v$$

$$h = h^*/L^*$$

$$n = M/M_p$$

(p subscript denotes pillar rock value)

E = Young's Modulus of the roof rock (psi)

ν = Poisson's Ratio

σ_h = Horizontal stress acting on the seam (psi)

σ_v = Vertical stress acting on the seam (psi)

b = Width of the pillar (feet)

L = Breadth of the mining zone (feet)

x^* = Displacement in x direction (feet)

h^* = Pillar height (feet)

L^* = Pillar length (half-length) (feet)

N = Number of pillars

R = Radial distance from center (feet)

- 3) the depth of cover must be uniform;
- 4) the displacements increase linearly in the vicinity of the seam.

The first three assumptions correlate to those made in the tributary-area theory. However, rigid elements and, more importantly, restrictions on stress distributions across the pillar and panel are eliminated and replaced with more realistic in-situ characteristics.

Oravecz concluded that the analog technique described adequately the stress displacement distribution across a pillar and supported the elastic-theory approach of Coates rather than the tributary-area-theory approach. However, despite this agreement between theory and field measurements, the tributary-area-theory is used in practical mining applications due to its simplicity and conservative results.

In summary, the commonly used tributary-area theory overestimated pillar loads determined from field investigations as indicated by Oravecz (1977) and Coates (1965). Conversely, the elastic-theory approach which accounts for deflections correlated well with the observed pillar loads. However, the rigorous solution and extensive analog-type procedure required to determine pillar load by this technique hampers the mining industry from utilizing it in this form. The development of a computer code, MINSIM (Crouch and Fairhurst, 1973), allows for the application of the theory in a convenient manner. Thus, the need exists for comparison of the computer version of this technique with observed in-situ pillar loads and the tributary-area technique. If a favorable comparison results between MINSIM and the in-situ loads, then the elastic theory may become an accepted method for evaluating pillar load.

3.1.2 Pillar-Strength Formulae

Prior to a discussion of pillar-strength formulae, a discussion of laboratory-strength characteristics is warranted. Most of the currently used pillar-strength formulae utilize a unit-strength value derived from the uniaxial compressive strength of laboratory specimens. However, laboratory strength is highly variable with specimen size and shape. Therefore, selection of an appropriate strength value requires extreme care. Cognizant of this variability, a discussion on specimen strength is presented with particular emphasis on the critical-size concept.

Critical Size

Early research was concerned with small-specimen strength dependence on specimen size (Millard, et al., 1955; Evans, et al., 1961; Daniels and Moore, 1907; Gaddy, 1956). Greenwald, et al. (1939), also investigated size effects but did so on large specimens taken from the mine. From this data, Gaddy (1956) commented that his specimen

strength-size equation "fits" the combined data from his and Greenwald's experiments in situ.

Holland (1962) stated that Gaddy's equation applied only up to specimen sizes of six to eight feet. Salamon and Munroe (1967) compared numerous pillar-strength-formula data bases and noted that formulae based on larger specimens result in less significant size effect than formulae based on small specimens. However, Bieniawski (1968a,c) first formally proposed the concept of critical size as a rock-mass property. He defined the critical size as that specimen size at which a continued increase in specimen width causes no significant decrease in strength. Other authors (Nose, 1964; Jahns, 1966; Gimm, et al., 1966; and Lama, 1971) confirmed that this critical-size phenomenon exists in various rock types as shown in Figure 6.

For South African coal, Bieniawski (1968c) concluded that 1.5 m (5 ft) cubic specimens constitute the critical-size value. Pariseau, in 1977, reported that the critical size for western coal is 0.9 m (3 ft). Hustrulid (1975) pointed out that a critical size of 3 ft would be generally applicable to coal for practical engineering purposes. Wang, et al. (1976) utilized the critical size given by Bieniawski in his research program. Singh (1980) compiled the data shown in Figure 7 relating specimen size to specimen strength from many investigations which presents a strong argument for critical size. Similarly, Peng (1978) compiled strength-size effect data from Bieniawski (1968a) and Greenwald et al. (1941) which yields the results shown in Figure 8. Both curves indicate a strong critical-size property for coal.

Protodyakonov's Formula

Protodyakonov (1964) developed a formula which considers the coal specimen as being broken up by discontinuities of an average spacing, b . The formula is stated as:

$$\frac{\sigma_d}{\sigma_m} = \frac{d/b + m}{d/b + 1} \quad (3.2)$$

where

σ_d = ultimate strength of the cylindrical specimen of diameter d ,

σ_m = ultimate strength of the rock mass, theoretically as $d \rightarrow \infty$,

b = average spacing of discontinuities,

$m = \sigma_o / \sigma_m$; where σ_o is the ultimate strength of the rock as $d \rightarrow 0$

The factor m is quite variable but recent research on coal indicated that a range of 5 to 7 is adequate for compression. Bieniawski (1980) plotted his specimen strength data along with Protodyakonov's relation and revealed an excellent correlation as shown in Figure 9.

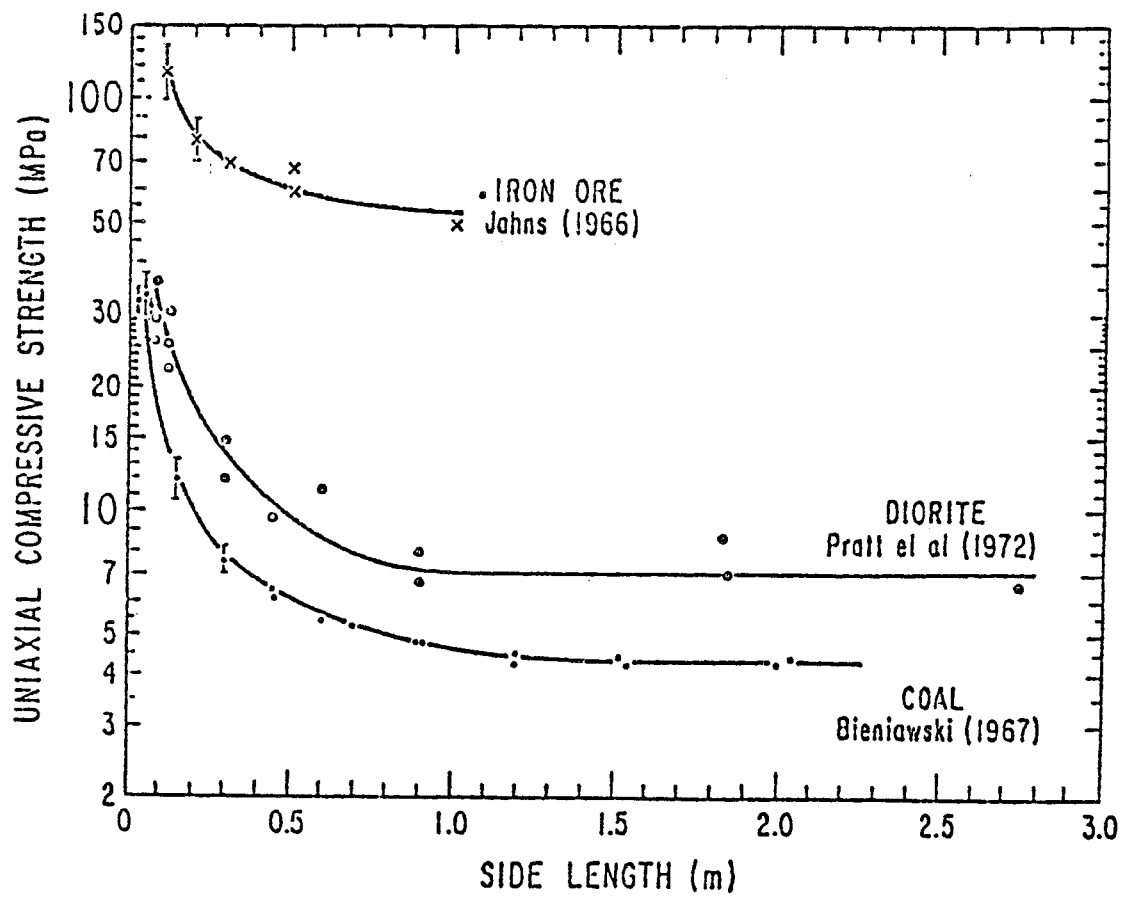


Figure 6. Critical-size phenomenon exhibited by different rock types.

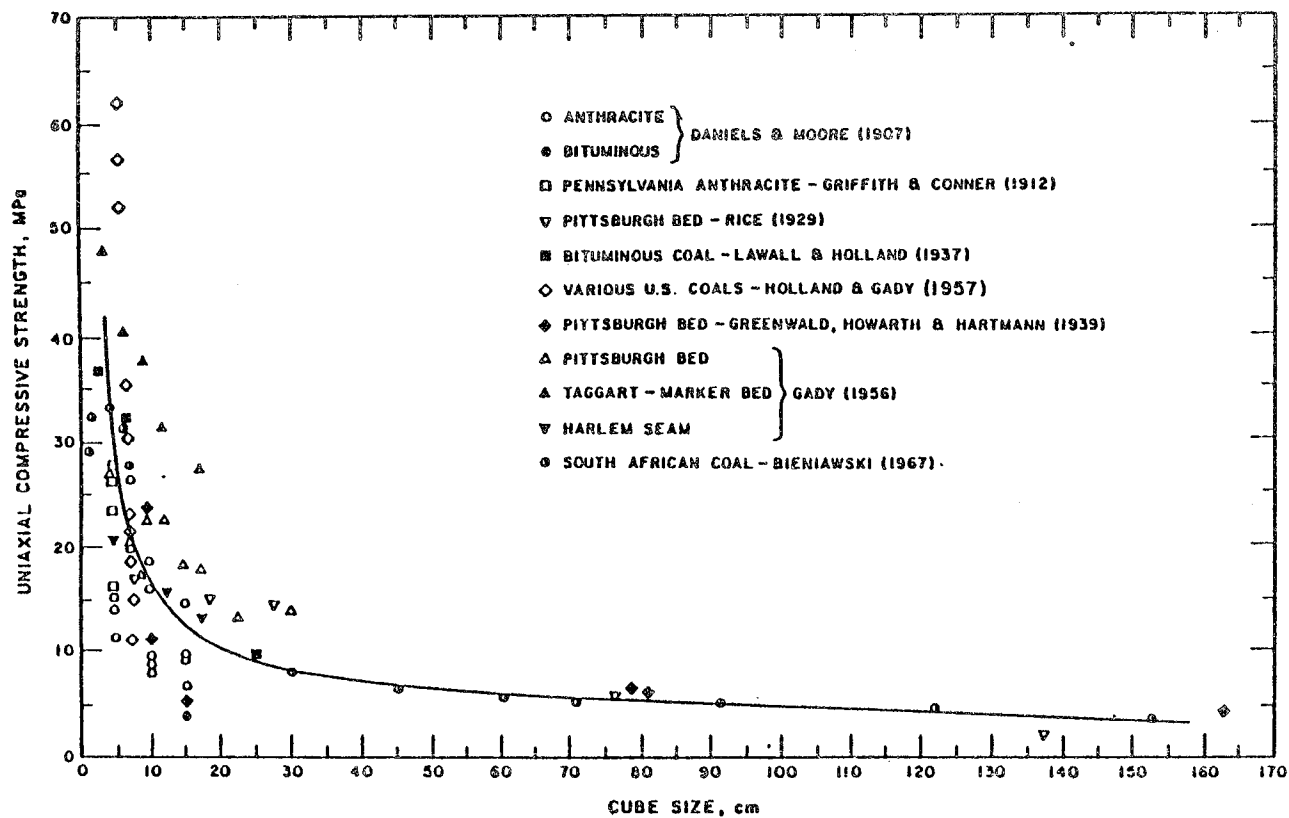


Figure 7. Specimen size effects in coal (Singh, 1980).

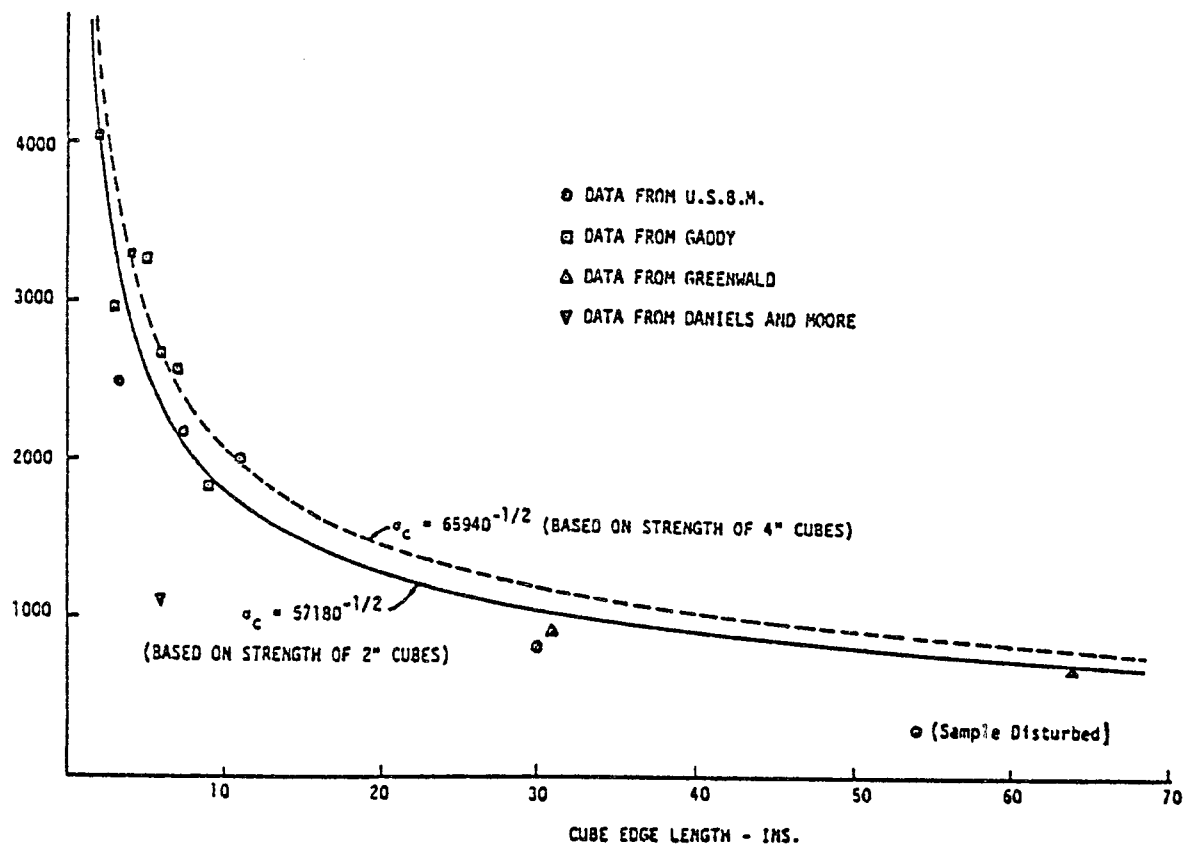


Figure 8. Compressive strength as a function of cube size for Pittsburgh coal (Hustrulid, 1975).

THE EFFECT OF SPECIMEN SIZE ON STRENGTH OF COAL

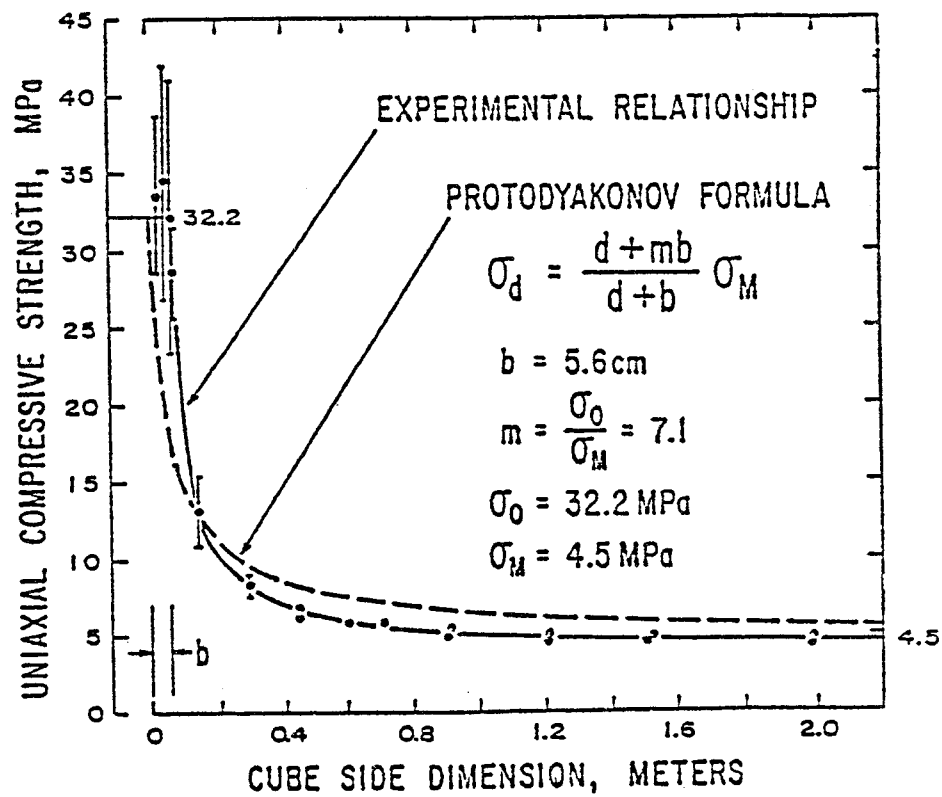


Figure 9. Comparison of Protodyakonov's equation with experimental data.

Shape Effects

Specimen strength behavior is described by two effects, size and shape. The first, size effect, was evident in Figures 6, 7, and 8. In addition, however, laboratory strength is also dependent on shape effect as reported by Greenwald, et al. (1941), Gaddy (1956), Bieniawski (1968a,b), and Holland (1973). This effect is described by the equation:

$$\sigma_p = \sigma_1 (w^\beta / h^\alpha)$$

and by

$$\sigma_p = \sigma_1 (A + B w/h)$$

where

σ_p = uniaxial compressive strength of specimens having

w = specimen width and h = specimen height;

σ_1 = strength of cubical specimens ($w = h$)

β , α , A , B = constants which vary with investigators and coal seams

This shape effect is valid from laboratory size specimens through full-size pillars at which point the pillars may become indestructable at high w/h ratios as shown by Figure 10 for sandstone specimens (Bieniawski, 1968b).

Since smaller specimens contain fewer defects, they exhibit greater strength. In addition, laboratory-specimen preparation and transportation affect the true population from which strength could be obtained. In essence, these handling procedures destroy the weaker portions of the rock mass. In-situ tests eliminate or reduce both size and environmental effects on rock-mass strength and, consequently provide more representative results.

Bunting/Obert & Duvall/Wang's Formula for Pillar Strength

Obert and Duvall (1967) derived from laboratory tests on hardrock and elasticity considerations the same relationship as did Bunting in 1911. This formula is given as:

$$\sigma_p = \sigma_1 (0.778 + 0.222 (w/h)) \quad (3.3)$$

where

σ_p = pillar strength (in psi)

w = pillar width (feet)

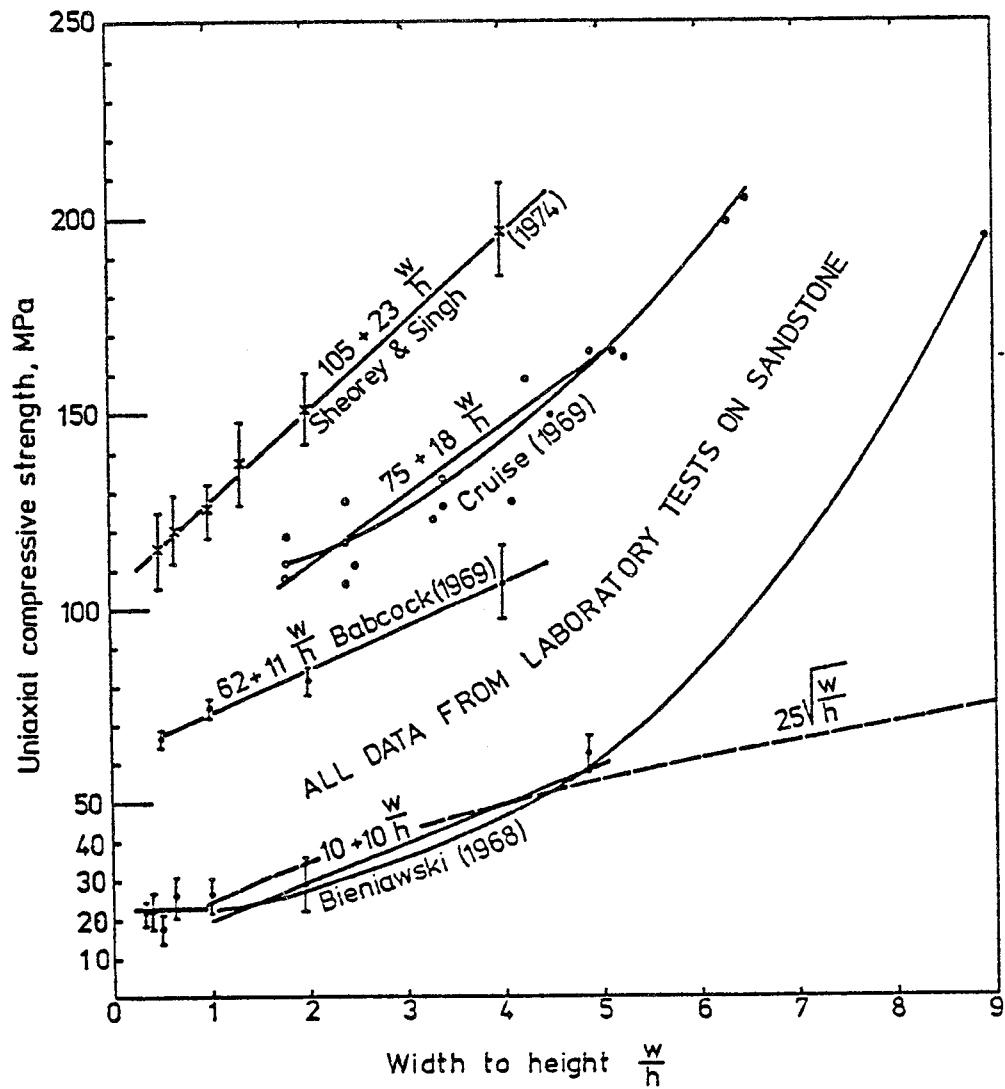


Figure 10. Effect of width-to-height ratio on strength of sandstone.

h = pillar height (feet)

σ_1 = uniaxial compressive strength of a cubical specimen
($w/h = 1$)

According to Obert and Duvall, the equation is valid for w/h ratios of 0.25 to 4.0 assuming gravity-loading conditions. Through back calculations from mining case histories and utilization of laboratory rock properties, safety factors of 2 to 4 were derived for short-and-long-term pillar stability, respectively. Essentially, this safety factor accounts for strength scaling from laboratory (or rock-material) strength to in-situ (or rock-mass) strength.

In 1975, Wang, Skelly, and Wolgamott conducted in-situ tests on two coal pillars located in West Virginia (Wang, et al., 1976). The tests consisted of reducing pillar dimensions until failure occurred and then determining the pillar strength. The authors proposed the same formula as above and expressed it as:

$$\sigma_p / \sigma_1 = 0.78 + 0.22 (w/h) \quad (3.4)$$

where σ_1 = the ultimate strength of a cubical specimen of critical size or greater.

The other variables remain as defined in equation (3.3). Nevertheless, distinct differences exist between this equation and equation (3.3). First, equation (3.4) was applied to coal strata. Second, the term σ_1 was introduced which acknowledged the existence of a critical-size phenomenon. Third, the equation was stated as being valid for w/h ratios up to 8. This extension was based on the results of the field tests conducted.

Holland-Gaddy Formula

Gaddy (1956) proposed a relationship between coal specimen strength and size of the form,

$$\sigma_c = kd^{-1/2} \quad (3.4)$$

where

σ_c = the strength of a cubical specimen

d = the edge dimension of the cube

and k = the factor that relates σ_c and d .

Holland (1964) extended Gaddy's work and discussed stress-concentration effects, rock discontinuity effects, and roof-to-floor friction effects on pillar strength. In 1964, he proposed the formula:

$$\sigma_p = \sigma_1 \sqrt{w/h} \quad (3.5)$$

In 1973, however, he proposed a different expression which became known as the Holland-Gaddy formula:

$$\sigma_p = \frac{k \sqrt{w}}{h} \quad (3.6)$$

where

σ_p = pillar strength (in psi)

w = pillar width (in inches)

h = pillar height (in inches)

k = factor from equation (3.4).

The Holland equations are widely used pillar design expressions in for underground coal mining in the United States.

Holland also recommended a safety factor between 1.8 and 2.2 for the design of coal pillars due to the pillar-load uncertainties included in the formula derivation, with the recommended value of 2.0. The width-to-height ratio, for which the Holland formula is valid, ranges from 2 to 8. Holland (1962) stated that w/h ratios of 9 to 10 can be used, but the result will underestimate the in-situ pillar strength due to significant confinement effects which his formula derivation did not consider.

Salamon and Munro's Formula

Salamon and Munro (1967) conducted a statistical study on 125 full-scale pillar case histories which included 98 stable and 27 failed pillar cases. Failure was confined to those collapsed areas which were larger in diameter than the depth below surface. These areas failed after a few weeks to 32 years from the actual date of mining. Stable workings were those standing for at least 18 months prior to the analysis and were assumed to stand forever. The validity of this latter assumption could not be verified since insufficient data was available for time-effect analysis. As a result of this study, they proposed the following empirical formula for ultimate pillar strength of square pillars in South Africa:

$$\sigma_p = kh^\alpha \frac{w^{0.46}}{h^{0.66}} \quad (3.7)$$

where

σ_p = pillar strength (in psi)

k = a constant representing the strength of one foot cubical specimen of coal

h = the pillar height (in feet)

w = the pillar width (in feet)

and α , β = statistically derived coefficients. This form of equation was selected because of earlier pillar strength equations of similar form (Greenwald, et al., 1939; and Gaddy, 1956). For their data, α was equal to -0.66, and β was equal to 0.46.

This statistical formula is applicable for square pillars and South African conditions and represents averaged coal-pillar strength in the coal mines analyzed. For use in the United States, the value of k must be determined for U. S. mining conditions as applicable for each coal seam.

In Situ Tests Formula

Bieniawski (1969) (1977) and Bieniawski and van Heerden (1975) conducted large-scale in-situ tests on 66 coal specimens of width-to-height ratios from 0.5 to 3.4. The following formula for pillar strength was proposed:

$$\sigma_p / \sigma_1 = 0.64 + 0.36 (w/h) \quad (3.8)$$

where

σ_p = the pillar strength

w = the pillar width

h = the pillar height

σ_1 = the ultimate strength of a cubical specimen of critical size or greater.

The formula is valid for w/h ratios up to 5. Beyond this value, confinement effects due to specimen geometry will tend to accelerate the rate of strength increase. Previous investigations (Bieniawski, 1968b) showed that a rapid strength increase occurred in sandstone specimens of w/h ratios greater than 5. Although the formula was developed in South African coal mines, Holland (1973) suggested that a safety factor of two is adequate for U. S. applications of this formula.

Wilson's Concept

Wilson's pillar-strength concept (Wilson, 1972; Wilson, 1977) was based on laboratory specimen behavior and elastic-plastic considerations. He visualized the pillar as containing two distinct zones: 1) a yield zone and 2) a pillar core. The first zone was the area of highly stressed coal along the pillar edge which had passed the yield point of the material. The latter zone consisted of the coal that has not reached the yield point and consequently behaved elastically.

Wilson (1977) utilized the axial strength behavior exhibited by rock specimens when subjected to various degrees of confinement in the formulation of his hypothesis. This behavior is described by:

$$\sigma_1 = \sigma_c + k\sigma_3$$

for the elastic zone, where

σ_1 = axial stress (in psi)

σ_c = unconfined compressive strength (in psi)

σ_3 = confining pressure (in psi)

k = triaxial stress factor relating σ_1 to σ_3 .

For broken rock, the unconfined strength was assumed to be negligible and the relationship reduced to $\sigma_1 = k\sigma_3$. This fundamental relationship, applied to elastic and plastic boundary conditions, resulted in the prediction of yield-zone widths and subsequent "minimum" yield-pillar sizes. The yield zone is expressed for two cases as follows:

- 1) weak stratum between strong roof and floor:

$$\sigma_y = k\gamma H$$

$$x_b = \frac{h}{F_k} \ln \frac{\gamma H}{p} \text{ where } F_k = \frac{k-1}{\sqrt{k}} + \frac{(k-1)^2}{k} \tan^{-1} \sqrt{k} \quad (3.9)$$

- 2) weak stratum between weak roof and floor:

$$\sigma_y = k\gamma H \quad (3.10)$$

$$x_b = \frac{h}{2} \left[\frac{\gamma H}{p} \right]^{1/k-1}$$

where

σ_y = peak abutment pressure (psf)

γ = unit weight of the overburden (pcf)

H = overburden depth (feet)

h = seam height (feet)

p = support resistance (see Wilson, 1977)

x_b = width of the yield zone (feet).

As an approximation, Wilson stated that $x_b = 0.0015 h$, where x_b is in feet.

The preceding assumptions may or may not be applicable to United States coal-measure rock. Therefore, laboratory data should be examined for verification of the validity of Wilson's assumptions to United States coals (Bieniawski, 1980). Furthermore, investigations of stress distributions across pillars are required to confirm the presence of a solid core and yield zone. The approach accounts for increased pillar strength due to confinement effects through the progressive-failure type of stability analysis. Thus, a non-uniform strength profile is incorporated in Wilson's hypothesis and, consequently, separates it from the other formulae presented.

Herget and Unrug's Approach

Herget and Unrug (1974) employed triaxial test data on broken and intact siderite specimens of various sizes (0.375 to 9.5 inches) for assessing pillar strength. The resulting Mohr's envelopes were plotted and clearly indicated a size effect. The strength prediction via a log-volume/log-strength relation was very low. However, the intact- and broken-specimen uniaxial-compressive strengths converged to the same asymptotic level from infinity and zero, respectively. Thus, this asymptotic value was used to predict pillar strength. The shortcoming is possibly in the attempt to extend laboratory-specimen volume effects to full-scale pillars directly.

Pariseau's Contributions

Pariseau conducted a study (Pariseau, 1975; Pariseau, 1977) directed toward assessing in-situ pillar strength. The research included laboratory, field, and analytical work on Utah coal. Pariseau concluded that numerical methods using limit-stress analysis are reliable design tools for pillar design. He added that data input remains a problem and that laboratory values were sufficient for the present stage of development. Pariseau (1975) also concluded that mine pillar design has stagnated at the elementary statics level; yet, these numerical tools are available for performing detailed stress analysis. This finite-element method, incorporated into a limit-design approach, allowed for the prediction of the progression of failure from local spalling to pillar collapse (Pariseau and Sorenson, 1979). Both local and global safety factors were promoted as indicators of pillar stability. Upper and lower bounds of pillar stability were obtained using elastic perfectly-plastic and elastic-brittle failure criterion, respectively. Analytical results showed that initial rib spalls enhanced pillar strength through core confinement. Therefore, this approach could be categorized as a progressive-failure approach.

Panek's Approach

Panek proposed an approach based on similitude modeling of the in-situ pillar. His equation as given in (1980) had the following form:

$$\frac{\sigma_P}{E} = K_o \left(\frac{d_1}{w} \right)^{C_1} \left(\frac{w}{h} \right)^{C_2} \left(\frac{z}{w} \right)^{C_3} \left(\frac{E_r}{E_s} \right)^{C_4} \left(\frac{E_f}{E_s} \right)^{C_5} \left(\frac{\mu_{s/r}}{z} \right)^{C_6} \left(\frac{\mu_{s/f}}{z} \right)^{C_7} \\ \times \left(\frac{1}{v_r} \right)^{C_8} \left(\frac{1}{v_f} \right)^{C_9} \left(\frac{1}{v_s} \right)^{C_{10}} \left(\frac{d_1}{d_2} \right)^{C_{11}} \left(\frac{d_1}{d_3} \right)^{C_{12}} \quad (3.11)$$

where

l, w, h = pillar dimensions, length, width, and height

d_1, d_2, d_3 = discontinuity spacing in each direction

v_s, v_r, v_f = Poisson's ratio for the seam, roof, and floor

$\mu_{s/r}, \mu_{s/f}$ = coefficient of friction at roof and floor contact of the seam

E_s, E_r, E_f = Young's Modulus of seam, roof and floor

σ_P = pillar strength

K_o, C_i = constants.

The method incorporates size effect, shape effect, and mechanical properties of the roof, seam, and floor. Herein lies the problem as these properties are difficult, if not impossible to obtain with sufficient accuracy.

Barrier-Pillar Formulae

Ashley (1930) proposed the following formula for barrier-pillar width:

$$W_{bp} = 20 + 4h + 0.1 H \quad (3.12)$$

where

W_{bp} = width of barrier pillar (feet)

h = seam height (feet)

H = overburden depth (feet)

British mines utilized the rule of thumb of one-tenth the overburden depth plus 45 feet:

$$W_{bp} = H/10 + 45$$

where

W_{bp} and H are as previously defined.

Pressure-Arch Theory

A concept of a 'pressure arch' is sometimes used for pillar-sizing. This concept is not limited to a single entry but entails the study of the entire panel width. It relies on the formation of yielding pillars--pillars that deform in a controlled fashion--and the subsequent transferral of overburden stress to the abutments (Stefanko, 1978). Yield-pillar-size guidelines are dependent on seam depth, seam thickness, and entry width.

However, yield-pillar sizing guidelines are only rules of thumb.

In summary, the pillar-strength formulae for support pillars generally used in the United States take the form of: $\sigma_p/\sigma_1 = A+B (w/h)$ or $\sigma_p/\sigma_1 = K \sqrt{w/h}$.

It should be noted that these formulae are valid for a limited range of w/h ratios and they require input from scaled laboratory values of strength (σ_1) which lead to, in part, the safety factor applied.

Most investigators (Holland, 1962; Wilson, 1972; Hazen, et al., 1975; Bieniawski and van Heerden, 1975; Wang, et al., 1976; Pariseau, 1977) agree that confinement effects enhance pillar strength at large width-to-height (w/h) ratios.

3.1.3 Pillar Deformability: Petite Sismique

Petite sismique is a geophysical technique featuring a seismic refraction survey, conducted over a relatively small area (2 to 10 m). This technique focuses on the determination of shear-wave parameters, especially the frequency, for the subsequent empirical correlation with the in-situ static deformation modulus. Schneider (1967, 1969) proposed the technique and introduced three separate findings.

- 1) An identification profile of the rock mass can be established that is based on shear-wave velocity and attenuation trends.
- 2) A relationship exists between the wave length of the shear-wave and the ratio of field seismic-to-static moduli.
- 3) A correlation is apparent between the transverse shear-wave frequency and static modulus of deformation as shown in Figure 11. Roussel (1968) offered a theoretical explanation for this correlation based on a rheological rock-mass model.

Therefore, Schneider went beyond the determination of seismic rock properties and proposed an empirical relationship between static properties and shear-wave parameters. This correlation implies tremendous potential if it proves valid because petite sismique would yield the rock-mass properties directly.

Houghton and Coetzer (1977) attempted to parallel this work in South Africa. Their survey involved distances of up to 18 m. A signal-enhancement seismograph that stores and sums signals on a record while possibly reducing random noise was found to aid in shear-wave identification. In addition, directional geophones were used which took

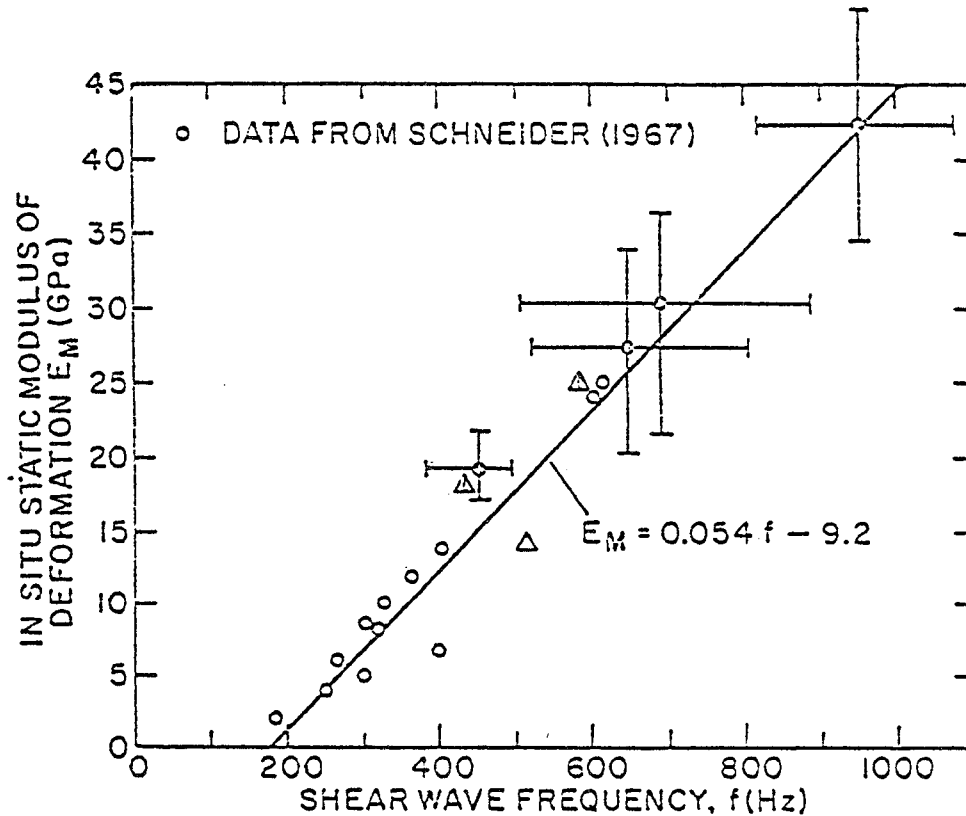


Figure 11. Correlation between static modulus of deformation from plate-bearing tests and shear-wave frequency from "petite sismique" (Bieniawski, 1978).

advantage of the shear-wave polarity characteristics. However, considering the distances used in the study and the physical records, Rayleigh-wave properties might have been measured instead of shear-wave properties.

Bieniawski (1979a, 1979b) summarized the previous work on petite sismique. He emphasized the need to use a signal-enhancement seismograph and directional geophones to obtain good shear-wave arrivals. His recommendations included the development of an improved shearwave generating source and data-recording

data-recording procedure, concurrent engineering geological mapping of the test site, and more field trials to verify the proposed correlations. Petite sismique is seen as a powerful tool since it potentially provides a large-scale and sound means of assessing the "rock mass" deformational properties.

Shear-Wave Dependence

The petite-sismique technique has not been used extensively due to the difficulties in obtaining clear shear waves on seismic records (Bieniawski, 1979a, 1979b). The quickest and easiest technique to propagate shear waves is to use a surface source along with surface-mounted receivers (Bieniawski, 1979b; White, et al., 1956). Shear waves have polarizing characteristics that subdivide them into radial (SV) and transverse (SH) components. Theoretically, SH waves can be generated and detected without compressional (P) wave interference--contrary to SV-wave propagation (Mooney, 1974). Therefore, properly oriented sources and receivers should purvey clear SH-wave arrivals. Furthermore, receiver type influences the success with which shear waves are detected (Mooney, 1974). Moving-coil geophones yielded the best results over the frequency range of 1 to 1000 Hz.

Past petite-sismique investigators (Schneider, 1967; Schneider, 1969; Bieniawski, 1979a) applied the previously mentioned techniques to enhance shear-wave detection and propagation. Once shear waves are identified, the frequency is determined and correlated subsequently to the static deformation modulus. The next step is to evaluate the rock-mass strength.

Couedtic and Barron's Formula

Couedtic and Barron (1975) statistically correlated the in-situ modulus of deformation from plate-load tests with uniaxial rock-mass compressive strength. This relationship was given as:

$$\log Q_u = 0.123 + 0.358 \log E_M$$

where

Q_u = uniaxial compressive strength (MPa)

E_M = deformation modulus (MPa).

Because of the limited number of tests performed, the authors did not place great significance on the actual values presented; what was more important was that it was possible to assess uniaxial compressive strength from plate-load tests.

Plate-load modulus of deformation values were found to follow the trend of those determined with petite sismique (Bieniawski, 1979), as shown in Figure 12.

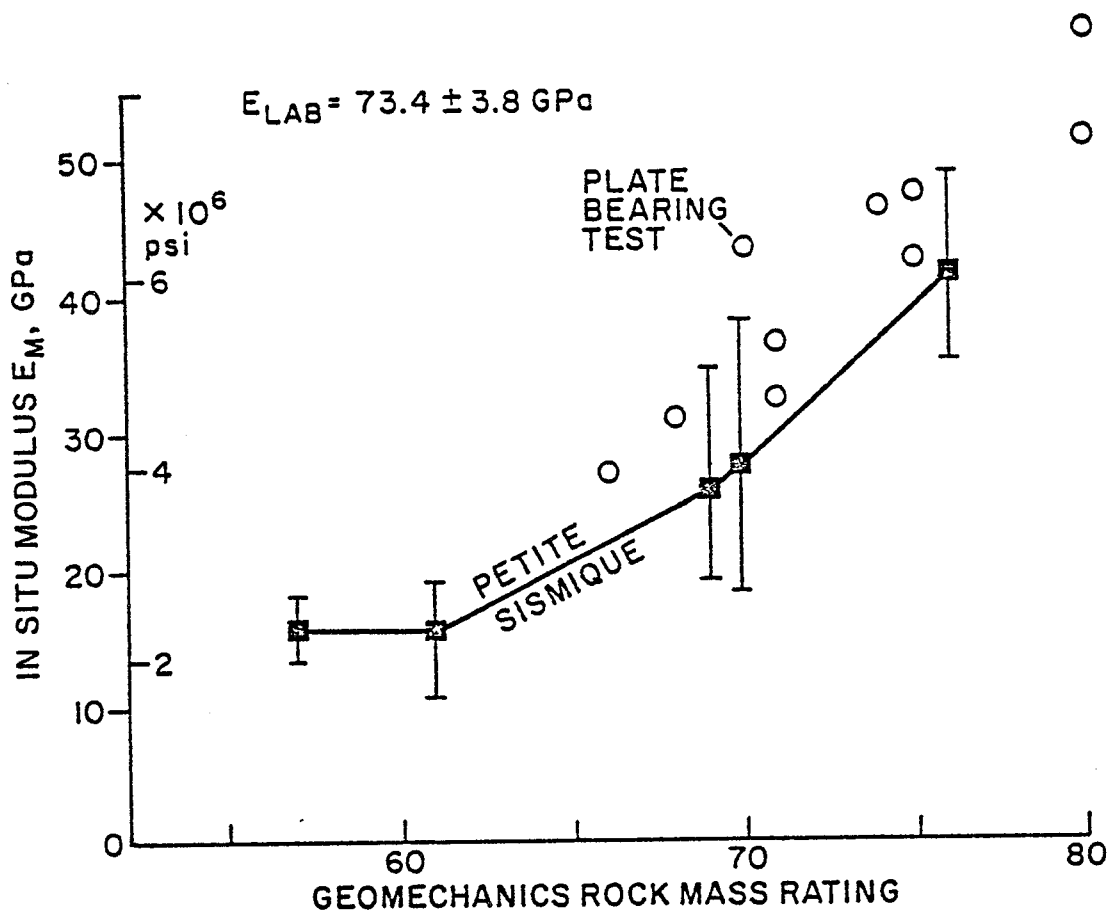


Figure 12. Comparison of the results from "Petite Sismique" and plate bearing tests (Bieniawski, 1979).

3.2 Field Study of Pillar Loading

To assess the pillar-load theories available for pillar design, a field study aimed at deriving in-situ load data was needed. Such data could provide a basis for comparison of the tributary-area theory and elastic-deflection theory. Hence, two pillars in a Utah coal mine were instrumented with vibrating-wire stressmeters to construct stress profiles within the pillar. The stress-profile magnitudes were compared with the tributary-area theory and numerical-modeling results representing the elastic-deflection theory.

3.2.1 Site Description

The selected site, code-named Mine Bud, is located in Utah. The depth of cover is 393 feet (123 m) at the instrument location which is the deepest cover in the mine. The site was selected because of ease of access and the need for data on western coal mines.

The panel geometry in section One East at the time of instrumentation consisted of a squared-off face area in front of the panel pillars. The panel pillars are cut on 80-by-100 ft centers with 18 feet (5.5 m) entries typically. Average mining height is nine ft (2.75 m) and the seam dips at six percent in the area of instrumentation.

IRAD Gage Vibrating-Wire Stressmeters (VWSM) were selected for pillar-load monitoring because of their low cost, ease of installation, and robustness. In this study, the soft-rock model was used since it is more suitable for coal. Recent field studies (Wang, et al., 1976; Pariseau, 1977) also incorporated VWSM for pillar-load detection with successful results. A convergence rod equipped with a dial gauge was used to measure closure between the roof and floor convergence pins. Similarly, four-point rod extensometers from IRAD Gage were used to monitor pillar dilation at various distances into the pillar. Figure 13 depicts the typical instrumentation scheme within a pillar which consists of VWSM, convergence pins, and four-point extensometers.

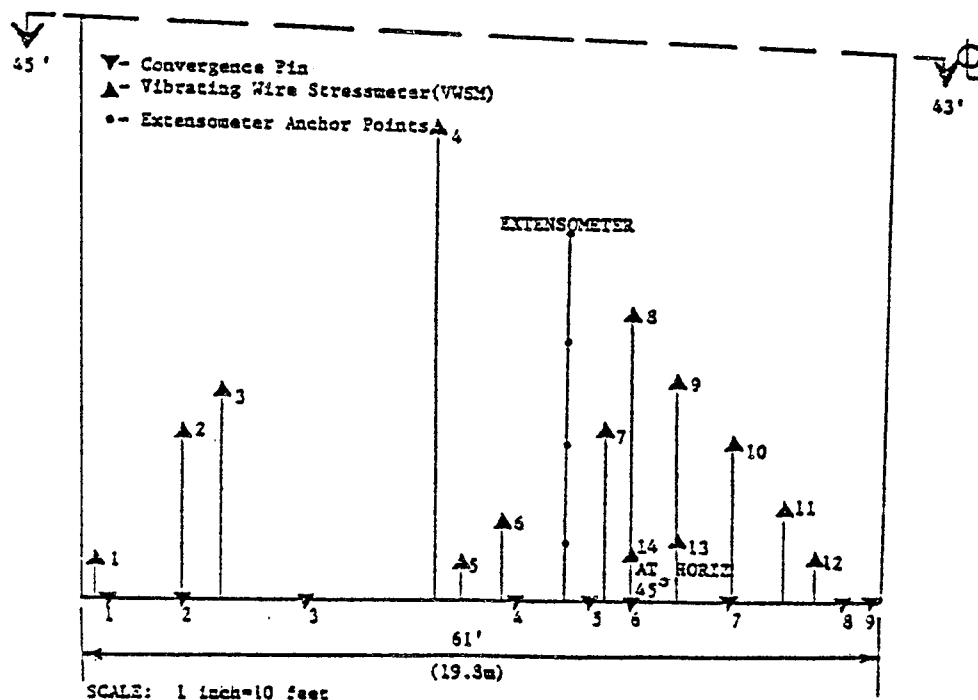


Figure 13. Locations of instrumentation in a coal pillar studied

The placement of the stressmeters was such that the stress profile with depth into the pillar and the deformation modulus along the rib could be determined. Consequently, sufficient data points are available for the construction of stress profiles along the pillar diagonals. Additionally, sufficient stress and deformation points are available in two areas for determination of the modulus of deformation.

3.2.2 Results

The results of the field measurements may be separated into two categories: short term and long term. The short-term instrument readings occurred during test-block mining. Long-term instrument reading occurred occasionally beyond this stage of mining and continued until the test pillars were split ten months later. The short-term results were needed for fulfillment of the stated objectives. The long-term results were needed to gain an insight into long-term pillar stability. However, many of the stressmeter electrical connections were pulled out and those that remained might have been affected adversely by tampering. Consequently, quantitative long-term VWSM results are not presented due to the paucity of the data base and the questionable validity of the data. A notable qualitative result was that long-term VWSM readings showed no significant stress change until retreat mining occurred. Therefore, long-term stability was likely.

The pillar-load trends compiled from the individual stressmeter results indicated that, to within three feet of the pillar rib, no yield zone developed. Therefore, elastic behavior occurred to the pillar rib. The existence of this elastic condition was strengthened by the absence of rib spalls. The lack of yield zone apparently disproves Wilson's (1972) hypothesis regarding the formation of a yield zone along the pillar edge. However, Wilson (1977) determined that no yield zone would develop at depths less than 660 ft (200 m) based on the hypothesis proposed. Since the field site was under 400 ft (120 m), this condition in Wilson's hypothesis actually is verified.

Based on the VWSM and convergence measurements taken, a deformation-modulus value was calculated for the coal mass. This value, 0.08×10^6 psi (0.58 GPa), is approximately one-sixth the laboratory-derived value of 0.48×10^6 psi (3.31 GPa). Use of the Goodman Jack would have enhanced these results since it yields the deformation modulus directly. However, the Goodman Jack was not used successfully due to the inability to drill an adequate borehole. Later tests in the laboratory showed this device to be an accurate method of deformability assessment.

3.3 Numerical Analysis of Pillar Load

Numerical techniques were used to assess the pillar-load theory that best represents the field-measured pillar loads. Although the tributary-area theory is used extensively in industry, Coates (1972) and Oravec (1977) showed that a theory based on elasticity represented

in-situ load well. The results obtained by Coates and Oravec stimulated interest in using MINSIM (Mining Simulation), a boundary-element technique, for assessing pillar loading. This numerical technique conforms to the alternative elastic theory and is convenient to use. In addition, SAP IV, a finite-element method computer program was applied to the field-study pillars since the technique has been shown to be reliable (Bathe, et al., 1973). These numerical techniques are used to evaluate both total and relative pillar-load values with mining advance.

The abutment load across the face area is required initially. Then the numerical results can be compared with the field-derived stresses to ascertain which pillar-load theory applied in situ.

3.3.1 Procedure

The first objective was to assess the abutment stresses prior to instrumentation. This objective was met by using both SAP-IV and MINSIM models. The SAP-IV model consisted of a two-dimensional, plane-strain, three-layer, 92-node mesh that represents a continuous flat face.

Since one computer run of MINSIM includes all the mining stages encountered, the stresses determined for the flat-face stage as shown in Figure 14 were extracted to represent the abutment stress field. In Figure 14, zero (0) values indicate unmined areas. The lines are added for clarity. Each number on the grid represents 6.6 feet (2.0 m) for this model. The darkened triangles show the location of the field instrumentation.

Figure 15 contains the SAP-IV and MINSIM results and shows the lower stress levels predicted by MINSIM. These lower stresses were a result of the panel abutments and stress relief due to a stiff, massive roof strata. The SAP-IV and MINSIM results are in good agreement for this geometry. Therefore, the use of MINSIM for abutment-stress loading is justifiable based on the comparison with SAP-IV results.

The second objective, that of MINSIM comparisons with in-situ measured pillar loading, consisted of MINSIM calculations solely. A sample comparison between MINSIM and the field results is provided in Figures 16a and 16b for selected stages of mining. Although stress levels were calculated at more stages of mining, some stages were eliminated from the figures for clarity. These figures present the relative and total stress changes predicted numerically and experimentally for the indicated stage of panel development. The straight-line connections between VWSM locations emphasize the discrete nature of these readings. The curved lines on the MINSIM plots indicate the continuous nature of these results. The overall trend with the VWSM data is similar to the elastic profile offered by the MINSIM analysis.

DIRECTION OF MINING

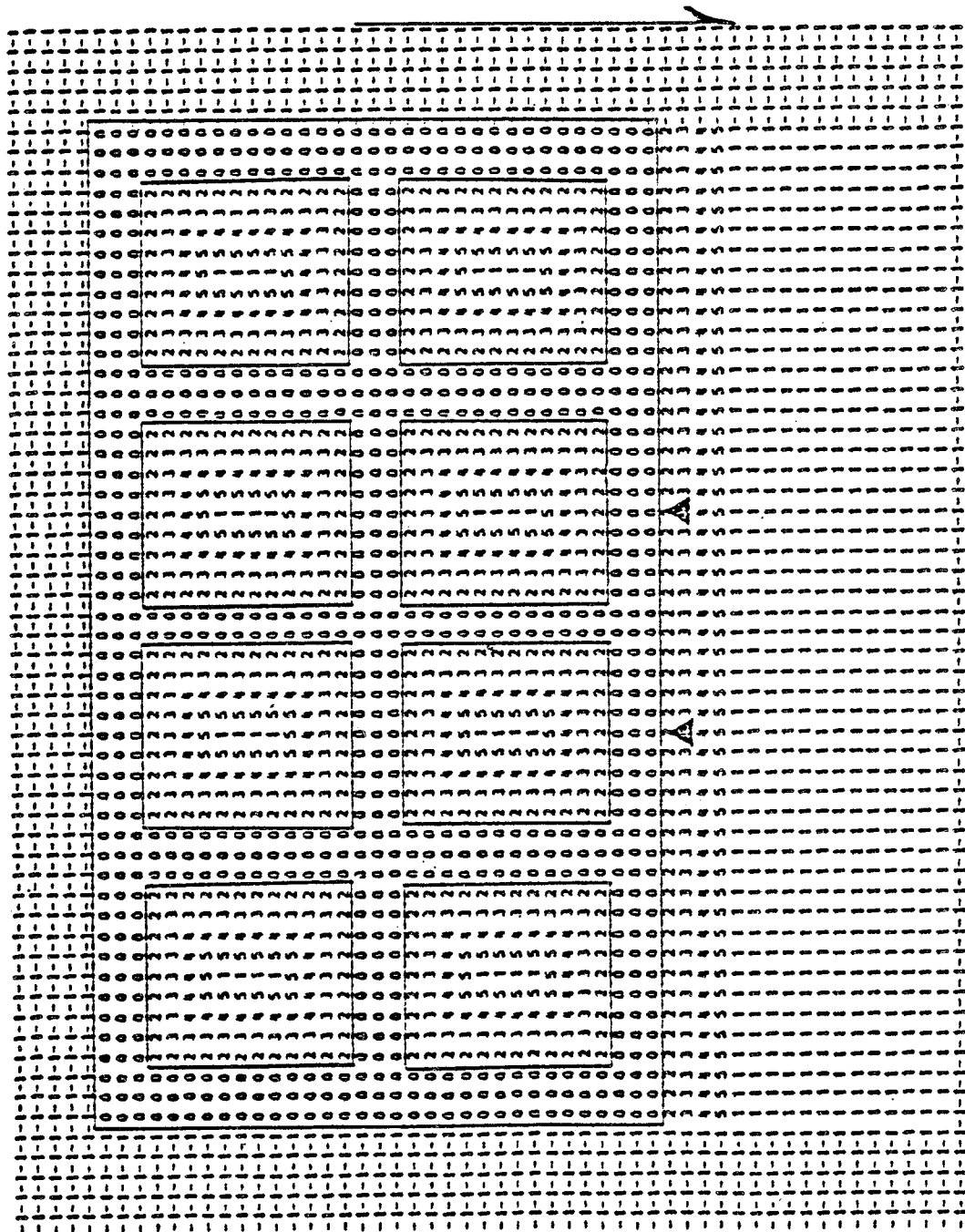


Figure 14. MINSIM representation of panel geometry.

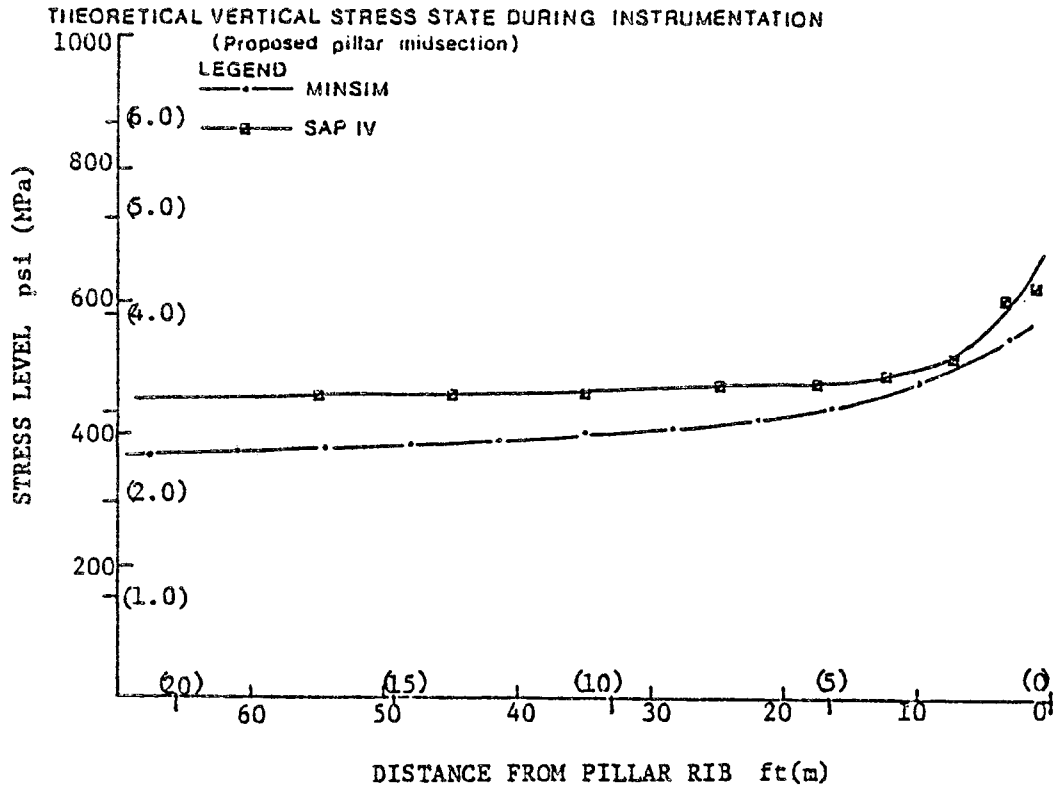


Figure 15. Initial results from SAP-IV and MINSIM analyses.

3.3.2 Results

The correlation as represented in Figures 16a and 16b show that MINSIM is a viable means of in-situ pillar-load assessment. However, the universal application of this technique, MINSIM, is not proven by these results from one field site.

In this study, existing numerical techniques were used as tools for strengthening the conclusions from the experimental results. However, several lessons were learned during this research. First, input data is an extremely sensitive aspect governing the ultimate result of numerical calculations. To accurately represent the field conditions, field-derived properties should be used.

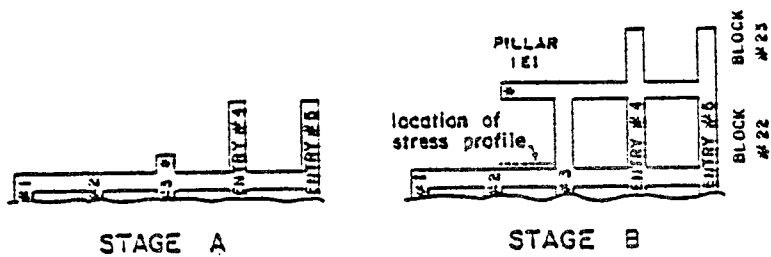
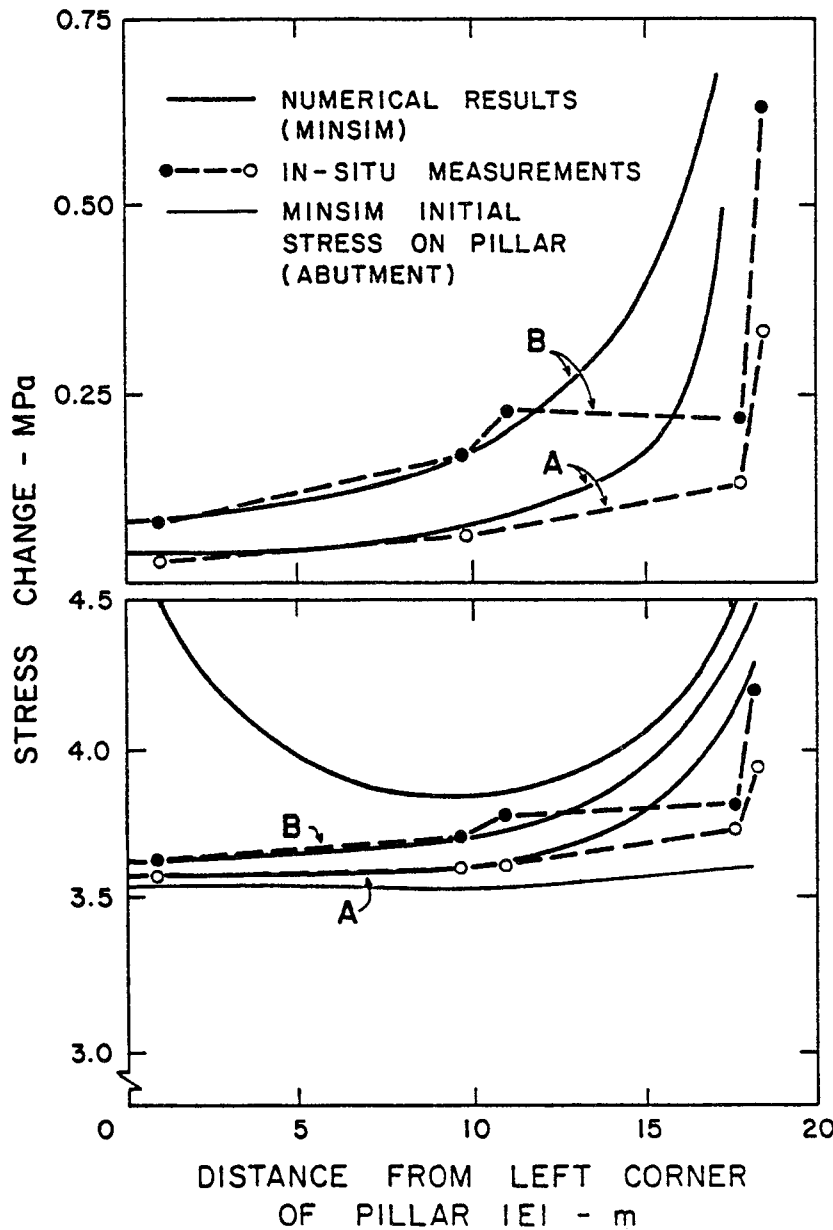


Illustration of mining advance for stages 'A' and 'B' as noted in Figures 16a & b.

* MINING OF BLOCK PROGRESSED TO 6.6 m IN ENTRY #3

* MINING OF BLOCK PROGRESSED TO CROSS-CUT 2-3



(a)

(b)

Figure 16. Numerical and in-situ results of: a) relative and b) total pillar loads across Pillar 1E1 near the initially exposed rib for mining states 'A' and 'B' as defined above.

Both laboratory and field-measured material properties were input into numerical calculations. The problem was that only coal properties were available from field measurements. Therefore, roof and floor laboratory data were reduced arbitrarily to reflect in-situ conditions. The massive nature of these strata warranted moduli reductions of 25 percent as compared to the assessed 80 percent modulus reduction for the coal seam. The resulting high roof-to-seam modulus ratio caused computational difficulties in both MINSIM and SAP-IV. Had each modulus been reduced equally, e.g., 25 or so percent, the resulting stress levels across the pillar would have been identical. This highlights that in numerical analyses, for a given roof-to-seam and seam-to-floor modulus ratio, regardless of the moduli magnitudes, the calculated seam stresses will be consistent. However, the seam and strata deformations will change with the elastic-modulus variations. This frustrating attempt to use representative in-situ rock-property values culminated in relying on laboratory moduli values in numerical and analytical (VWSM) calculations. This use of laboratory input is what Pariseau (1977) suggested to use due to the present ambiguity surrounding laboratory to field-property differences. Thus, the stress analysis presented earlier relies on the premise that the laboratory and in-situ modulus ratios between the roof, seam, and floor remain constant.

The results show that the elastic theory as incorporated in the boundary-element technique, MINSIM, represents in-situ pillar load well. Conversely, the tributary-area theory and SAP-IV analyses overestimate total pillar load in this case by nearly 37 percent. The shortcoming of not including panel abutments in their formulation causes this overestimate in the tributary-area and SAP-IV analyses. However, the tributary-area still has a role in providing "conservative" estimates of pillar load, as it contains a built-in safety factor of about 1.4.

3.4 Evaluation of the Petite Sismique Technique

Field trials provided a basis for assessing the feasibility of the petite sismique technique (Belesky, 1981). The trials were conducted in three phases.

- Phase I preliminary trials in a model mine-ventilation tunnel at The Pennsylvania State University with the sole intent of equipment and procedural familiarization,
- Phase II in-situ trials in an underground limestone mine to assess the feasibility of using petite sismique in the field,
- Phase III in-situ trials in two underground coal mines to assess the applicability and reliability of petite sismique for deformability evaluation of coal strata.

A description of the procedures and results from Phase I and Phase II tests is contained in Belesky (1981) and Belesky, et al. (1981). Of most importance are the results from the Phase III tests conducted in coal.

The introduction of 100-Hz geophones instead of the 28- and 14-Hz geophones used previously, along with the expanded-scale physical records and increased amplifier gains enhance the potential for successfully determining shear-wave frequency in these Phase III trials. These trials were undertaken in two different bituminous-coal seams, the Lower Kittanning and Upper Freeport, located within Pennsylvania. The immediate objective of identifying the best source-to-receiver orientation was met in this phase of the project. Based on velocity determinations (due to the large data base), the optimal strike direction on the source bar in surface-refraction surveys was downward, perpendicular to, and in the plane of the impact. This results agrees with Jolly's (1956) findings on SH-wave radiation patterns in which the maximum SH-wave energy occurred in a direction perpendicular to, and in the plane of, the impact direction.

The less competent Lower Kittanning seam yielded lower velocity values than the more competent Upper Freeport seam indicating that coal seams can be distinguished from the seismic properties. The 100-Hz geophones allowed frequency measurements to be made since previous resonance problems decreased in severity. The frequencies measured were applied subsequently to the correlation proposed by Schneider. However, these frequency values did not confirm Schneider's correlation as shown in Figure 17.

In spite of the lack of correlation the petite sismique trials were relatively simple to conduct compared to other available rock-mechanic methods. At this stage of development, surface-refraction surveys proved to be the best method for observing shear waves.

Discussion of Petite-Sismique Results

The objective of developing a correlation to substantiate Schneider's results between the observed shear-wave frequency and the static deformation modulus is the major goal of this research. The observed frequency will depend on two factors: 1) the source-time function and 2) the transfer function between the source and geophone.

Greenfield (1978) computed the total seismic response for a blow using a theoretical waveform-modeling procedure (WMP). This WMP was verified by comparison to observe seismic waveforms. Greenfield's study showed that a major factor controlling the observed frequency was the time, T_s , that the striking implement was in contact with the struck surface.^s It can be expected that T_s becomes shorter and the frequency emitted from the source becomes higher as the rigidity of the rock increases. However, T_s will depend also on the type of contact between the striking implement and struck surface.

The source used in the present study -- a blow on an implanted steel bar -- did produce different dwell times in the different rock media. However, Schneider (1967) struck the rock directly with a hammer, whereas the present study utilized a blow on an implanted steel

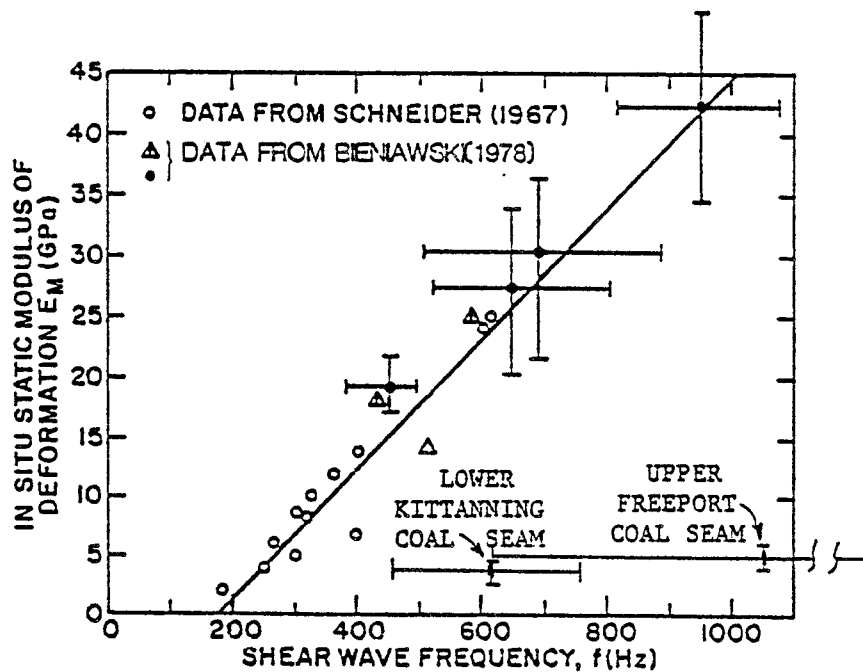


Figure 17. Results of petite sismique trials in coal

bar. Consequently, the higher-than-expected frequency observed for coal is probably a result of a smaller T_s value than Schneider's source would have produced. This result does not imply that a correlation between the deformation modulus and frequency is nonexistent. However, T and subsequently the observed frequency depend on the geometry of the implantation of the bar and of the impacting body as the elastic properties of the medium.

Thus the development of an empirical relationship between the elastic moduli and the signal frequency will require the development of a reproducible implementation procedure for the source and force applied to the source. One such device is the Schmidt hammer. Hardy (1974) showed the reproducibility attainable using the Schmidt hammer as a seismic source.

The transfer function, the second factor affecting the seismic response, includes elastic-wave effects of both geometric spreading and wave-type excitation as well as the effect of wave attenuation. Advanced record-analysis techniques should allow for frequency determination to be done in a fashion to account for transfer-function effects.

Finally, resonance attributed to the geophones created major difficulties in record interpretation. Although the manufacturer's response curves showed a flat response beyond 80 Hz for the 14-Hz Sensor receiver, these geophones produced a resonance at approximately 240 Hz for all forms of excitation. Use of high-frequency, 100-Hz geophones

resulted in significant reductions in receiver resonance. A further suggestion from Geometrics/Nimbus (Totman, 1981) is to use hi-fidelity geophones if resonance is critical. Similarities between first-arrival frequencies recorded on "high frequency" and standard geophones strengthen the argument that these frequencies are dependent on the source and wave-propagation principles.

In conclusion, the immediate aim, that of producing clear shear waves for subsequent use in petite-sismique evaluations, was partially satisfied in this research. Receiver resonance posed shear-wave interpretation problems throughout the investigation. Due to the frequency ranges under investigation (200 to 1500 Hz), careful selection of receivers is imperative to the success of eliminating these mechanical resonances. However, using the frequency results believed to represent shear-wave frequency, the data does not support Schneider's correlation as shown in Figure 17. This discrepancy may be attributed to differences in the source characteristics between the present work and that of Schneider. Since source characteristics and geometry influence wave parameters greatly, the future goal should be to standardize the entire seismic system, especially the source. The trend in the data justifies further investigation and suggests potential applicability of the petite-sismique technique.

Seismic analyses as discussed herein are seen as feasible means of assessing in-situ properties. However, much work is required before useful correlations with rock properties evolve. The following recommendations are aimed at improving the potential feasibility and reliability of petite sismique. First, a reproducible, standardized source such as the Schmidt's hammer or an electromechanical device is essential for possible future correlations. Research is continuing on this aspect at Penn State. Second, flat-response, hi-fidelity geophones are needed to eliminate mechanical resonances. Third, magnetic recording and high recording speeds are needed to analyze the frequency results better. Fourth, a detailed laboratory investigation is called for which would be directed at optimizing geophone mount, source type, blow orientation, and geophone type in addition to selecting optimal record gain and filtering.

3.5 Recommended Pillar Strength Formulae

From the review given in the previous sections, it is apparent that, although many pillar strength formulae have been proposed, two types of expressions are predominant:

$$\sigma_p = \sigma_1 \left(A + B \frac{W}{h} \right) \quad (i)$$

and

$$\sigma_p = K \frac{W^\alpha}{h^\beta} \quad (ii)$$

where σ_p is the pillar strength, σ_1 is the strength of a cubical pillar at the critical specimen size (scaled strength incorporating the size effect), K is a constant characteristic of a coal seam while α and β are constants expressing the shape effect; w is the pillar width and h is the pillar height (usually the same as the seam height).

The Size Effect

It has also been shown in the previous sections that the pillar strength is made up of two effects: the size effect and the shape effect. The size effect must be taken into consideration to account for the difference in the strength reduction between the small size specimens tested in the laboratory and the large size coal pillars mined in situ. Research in the United States has shown (Hustrulid, 1975) that the scaling of coal properties from laboratory measured data to field values can be satisfactorily achieved by the following equations:

$$\sigma_1 = \frac{k}{\sqrt{36}} \quad (\text{iii})$$

applicable for cubical pillars if the height $h > 36$ inches, or

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

applicable for cubical pillars if the height h is less than 36 inches.

In the above equations, the constant k must be determined for each coal seam and is obtained as shown by Gaddy (1956):

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

where σ_c is the uniaxial compressive strength of coal specimens tested in the laboratory and having a diameter or cube side dimension D (in inches).

It should be noted that although there is a difference in laboratory results depending on whether cylindrical or cubical specimens are used, for practical engineering purposes this difference is not significant within the range of D between 2 to 4 inches (Hustrulid, 1975).

The Shape Effect

From all the available pillar strength formulas, the following five expressions are relevant for applications to U.S. room-and-pillar coal mining:

1. Obert-Duvall (Bunting)/CSM Formula:

$$\sigma_p = \sigma_1 (0.778 + 0.222 w/h) \quad (\text{vi})$$

2. Holland/USBM Formula

$$\sigma_p = \sigma_1 \sqrt{w/h} \quad (\text{vii})$$

3. Holland-Gaddy Formula

$$\sigma_p = \frac{k \sqrt{w}}{h} \quad (\text{viii})$$

4. Salamon-Munro Formula

$$\sigma_p = 1320 \frac{w^{0.46}}{h^{0.66}} \quad (\text{ix})$$

5. Bieniawski/PSU Formula

$$\sigma_p = \sigma_1 (0.64 + 0.36 w/h) \quad (\text{x})$$

All these formulas have been applied extensively in coal mining. Using the Pittsburgh coal seam as an example, the above formulas may be compared. From the research by Gaddy (1956) and Hustrulid (1975), the following value is characteristic of the Pittsburgh coal:

$$k = 5580$$

which will be used for equation (viii).

Thus

$$\sigma_1 = \frac{k}{\sqrt{36}} = 930 \text{ psi}$$

The above value will be used for equations (vi), (vii) and (x).

For equation (ix), the constant 1320 represents the strength in psi of a one-foot cube specimen of coal determined in South Africa. For U.S. conditions the appropriate constant may be determined by the Hustrulid equation (iv).

Thus, for Pittsburgh coal:

$$K = \frac{k}{\sqrt{12}} = 1610 \text{ psi}$$

Consequently, the five formulas may now be rewritten as:

$$1. \quad \sigma_p = 930 (0.778 + 0.222 w/h)$$

$$2. \quad \sigma_p = 930 \sqrt{\frac{w}{h}}$$

$$3. \quad \sigma_p = \frac{5580 \sqrt{w}}{h} = \frac{5580}{\sqrt{h}} \sqrt{\frac{w}{h}}$$

$$4. \quad \sigma_p = 1610 \frac{w^{0.46}}{h^{0.66}} = \frac{1610}{h^{0.2}} \left[\frac{w}{h} \right]^{0.46}$$

5. $\sigma_p = 930 (0.64 + 0.36 w/h)$

In Figures 18 and 19, the above five formulas are plotted as the strength versus the width-to-height ratio and the strength versus the pillar width for a selected pillar height.

It is apparent from Figures 18 and 19 that for higher width-to-height ratios the Holland-Gaddy formula predicts the lowest strength while the PSU formula predicts the highest strength. At the same time, the form of the Holland formula is such that it will become very conservative at high width-to-height ratios. The higher strength values predicted by the PSU formula are consistent with the fact that for high width-to-height ratios there is a very rapid strength increase. In fact, pillars are thought to be almost indestructible for width-to-height ratios greater than 10 (Cook, 1978). This is further substantiated by the PSU survey of U.S. coal mines (described in the next chapter) from which it is apparent that pillars are overdesigned for width-to-height ratios of five and over.

In fact, a detailed study of this aspect conducted at Penn State has revealed that the theoretical strength* of coal pillars is considerably higher than that predicted by the PSU formula. Thus, at some stage of this research it was proposed (Belesky, 1981) that an exponent could be added to the PSU formula thus incorporating a higher rate of strength increase with increasing width-to-height ratios. This effect is depicted in Figure 20. However, for the sake of safety and due to the lack of experimental data, it was decided not to pursue the concept of an exponent in the PSU formula (Bieniawski, 1981).

It should nevertheless be noted that the results of underground tests on coal pillars performed by Wagner (1974) have shown that pillars of rectangular cross-section are about 40 percent stronger than square pillars of the same width and height. He measured the stress distribution across pillars at various stages of deformation and found that the perimeter of a pillar is capable of carrying relatively little stress but this portion of the pillar provides lateral confinement which enhances the strength at the center of the pillar. He also concluded that long rectangular pillars having a width greater than 10 times their height are unlikely to fail except for punching into the roof or the floor.

Furthermore, it is important to observe that all the five pillar strength formulas plotted in Figures 18 and 19 make use of the tributary area theory (for determination of factors of safety) which overestimates the pillar load (stress on pillars) by some 40%.

*This method (Bauer, 1980) calculates the pillar strength based on rock mass properties such as: the cohesion and the angle of internal friction of coal, the cohesion and the angle of friction at the pillar-roof interface as well as the pillar dimensions of width, length and height. For example, for $w/h = 12$, the theoretical safety factor is 17.5 for a typical mining situation in Pittsburgh coal.

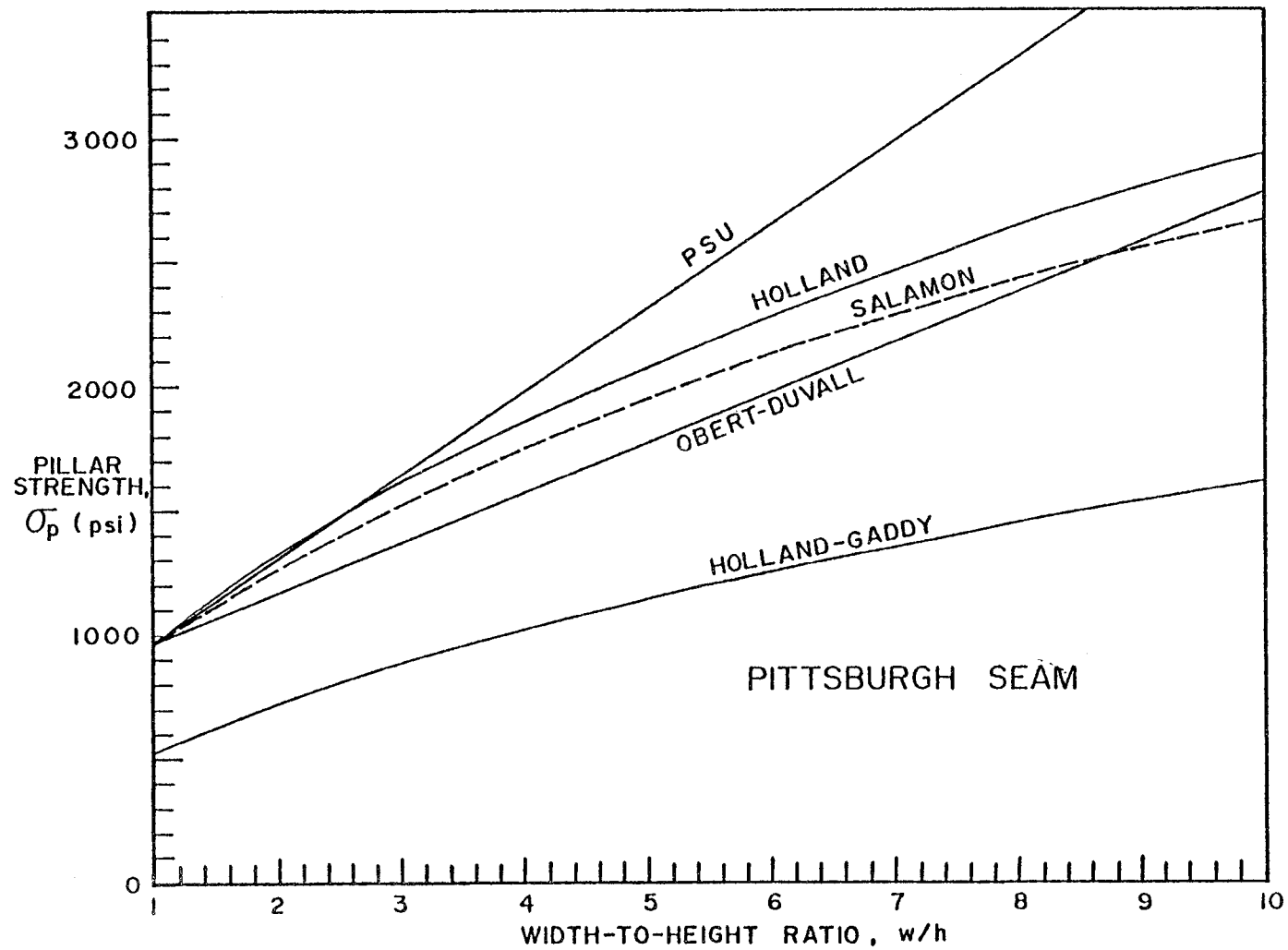


Figure 18. Pittsburgh Seam: Strength vs w/h ratio of pillars.

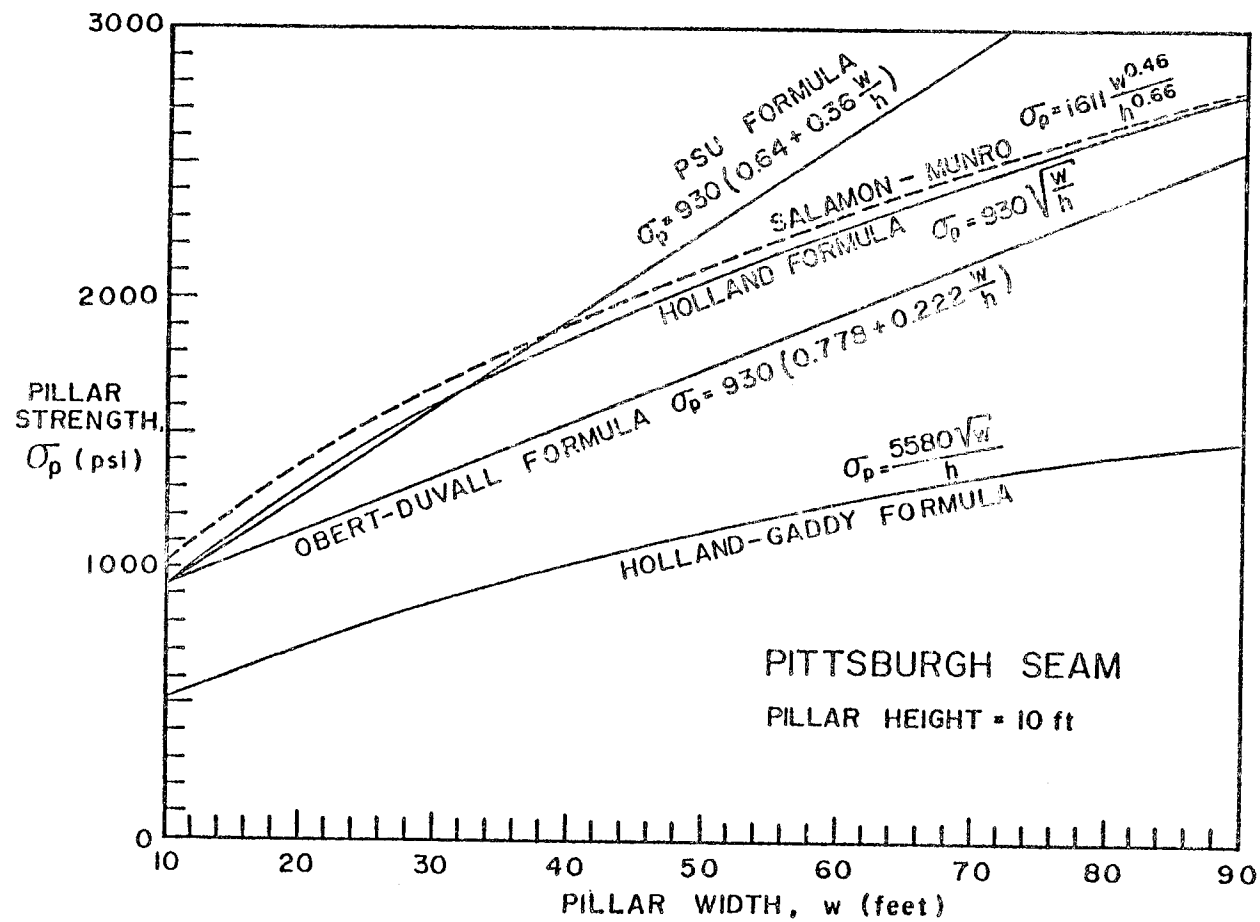


Figure 19. Pittsburgh Seam: Strength vs Pillar width for $h = 10$ ft.

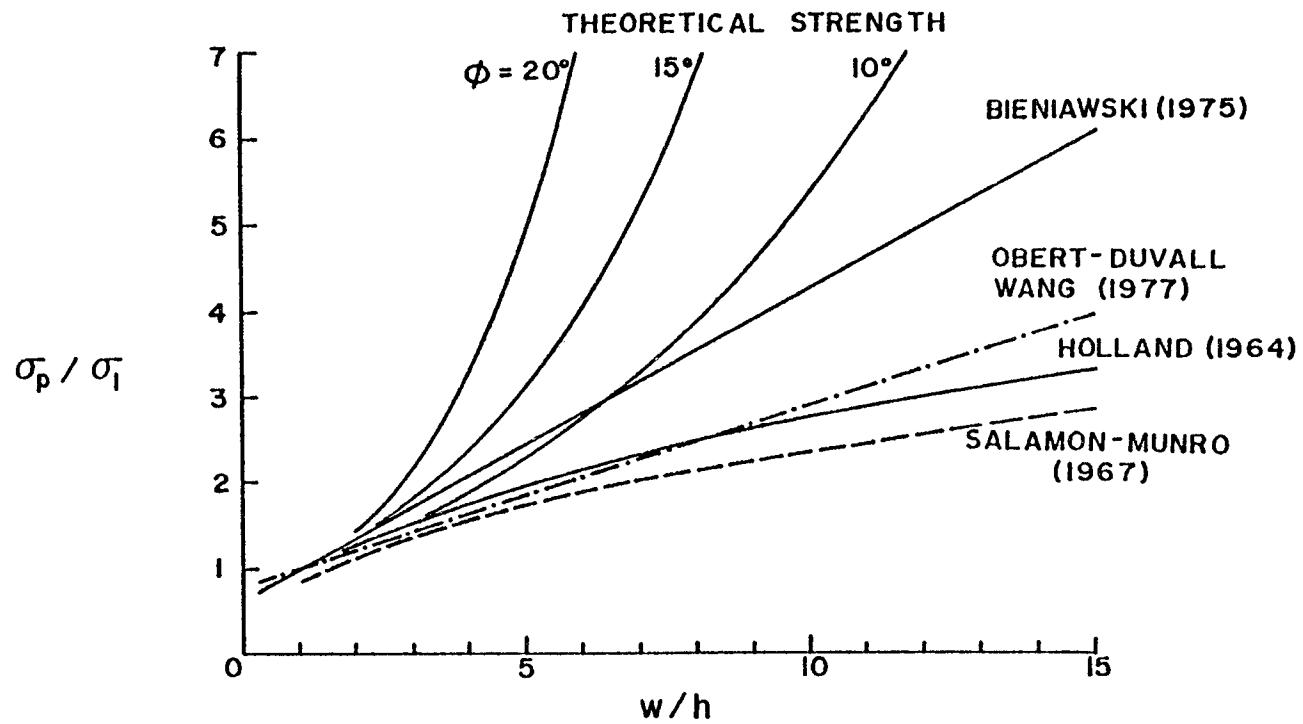


Figure 20: Comparison of pillar strength by empirical formulas and a theoretical strength formula.

Improved Pillar Strength Estimates

Consequently, since the aim of this research is to provide an improved design for room-and-pillar coal mining, it is believed that the PSU pillar strength formula makes the greatest contribution in this respect. This is so because, as discussed above, the formula does provide a safe estimate of pillar strength and also it determines the highest pillar values by comparison with the other formulas and this will lead to a greater recovery of coal and hence will result in an improved, better designed and more economical mining operation.

In addition, it is also obvious from Figures 18 and 19 that the Holland formula and the Salamon-Munro formulas are quite close in their predictions. This is not surprising since Salamon-Munro used the format of the Holland formula to derive their expression for the strength of coal pillars in South Africa. However, as the Holland formula is already widely used in North America, it is logical to reserve the Salamon-Munro formula for South African coal mines for which it was developed.

It is also evident that the Holland-Gaddy formula is quite conservative by comparison with the original Holland formula as well as with the other three formulas.

Thus, for an improved design of coal pillars in the United States, the following formula is recommended:

$$\sigma_p = \sigma_1 (0.64 + 0.36 w/h) \quad \text{PSU Formula}$$

where $\sigma_1 = 930$ psi for Pittsburgh coal and, for general use,

$$\sigma_1 = k/\sqrt{36}$$

In addition, as it is always useful to cross-check the results for exercising engineering judgment, it is suggested that the Holland formula and the Obert-Duvall formula be employed for this purpose.

The safety factors appropriate to the pillar strength formulas will be discussed in Chapter 5.

It should be stated that the above formula is valid for pillar sizes greater than the critical size which is about 3 ft for coal and for width-to-height ratios equal and greater than unity. Other formulas are available for $w/h < 1$ and for smaller specimen sizes (Hustrulid, 1975; Bieniawski and van Heerden, 1975). Furthermore, for width-to-height ratios of 10 and greater, the formula will underestimate the strength of pillars.

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CHAPTER 4

SURVEY OF MINING PARAMETERS FOR ROOM-AND-PILLAR COAL EXTRACTION

A national survey of coal pillar and roof dimensions in the United States was conducted with the aim of identifying factors of safety for room-and-pillar design applicable to the United States coal fields. Any pillar design formula requires an input of a factor of safety, defined as a ratio of pillar strength to pillar load (pillar stress), to assure that the pillar will not fail during the mining process; factors of safety should range above unity for stable pillars. The safety factor provides a margin of pillar strength in reserve to account for uncertainties in the design process and the variability of geologic conditions and structural dimensions encountered in the mining area.

Work on coal pillars performed in South Africa by Salamon and Munro in 1967 has been used as a foundation for the Penn State survey. They established safety factors by surveying stable and failed coal pillars in South Africa with the assumption that accumulated mining experience and practice has led to appropriate factors of safety. Thus, a statistical formula for pillar design was developed which rationalized accumulated experience and practice. The form of the pillar design formula selected in South Africa was based on earlier research conducted in the United States by Greenwald et al. (1939) and by Holland (1964).

It should be noted, however, that the objective of the Penn State survey was not to develop a new pillar strength formula. Unlike in South Africa, in the United States a number of pillar design formulas have been developed and used successfully. The aim here was to collect enough field data in order to rationalize the design by improved procedures and appropriate safety factors.

As an initial step under this task, a three page questionnaire was developed by project personnel to solicit information from active mining operations regarding room-and-pillar sizing practices at their mines. An example of the questionnaire prepared by Penn State researchers for this survey is given in the Appendix. The survey form was kept as brief as possible to increase the chance that a mine-level engineer would take the time to fill it out rather than discarding it; at the same time as much information as possible was requested. This included geology, mining methods, roof control methods and design criteria. It was anticipated that much of this additional information could be directly applicable to the other phases of this project by identifying critical parameters and the range of conditions encountered in the field.

This questionnaire was sent out to 37 major mining groups, representing over 500 coal mines in a variety of conditions and locations. No attempt was made to limit the questionnaire to a particular geographical area of the country or a particular seam; it was

felt that should the responses suggest that this work should be limited to one seam or area, then this narrowing of areas could take place during the latter stage of this work. The coal groups which received the survey questionnaire were primarily selected from a review of Keystone Coal Industry Manual, a publication containing detailed information on all United States coal mining companies and operating mines. Companies chosen had operating mines with an annual production capacity exceeding one million tons, since it was felt that companies having large mines would be more likely to use scientific design methods and have an engineering staff capable of responding to the survey questionnaire. This list was then supplemented with companies known by the Penn State staff to operate several mines and have an emphasis toward engineering in design.

A detailed cover letter explaining the background and concept behind the survey form was included along with detailed instructions of how to fill out the questionnaire and a sample form completely filled out. A copy of the complete mailing is given in the Appendix. The survey questionnaire was followed up, where possible, by a telephone contact by one of the project directors with the individual slated to receive the form within a given coal company.

As a result, 109 responses were received from the various coal mines in the United States. Only one company has formally declined to participate, citing excessive demands on their engineering personnel related to both mine production and compliance with federal laws. It was originally anticipated that each mine would submit as a minimum two case studies, one detailing panel entries and one on main entry development. Unfortunately, most of the responses that have been received consisted only of a single case study for a mine; this was either a result of a desire by mine level engineers to minimize their time spent in filling out forms or standardization procedures. Initial response to the pillar survey form was extremely good, with the majority of responses being received within two-to-three months of the solicitation.

In addition to the 109 case data collected from the survey, 20 case histories were obtained from the literature and 45 cases were available from a previous survey conducted at Penn State in 1977 by S. Suboleski. Thus, in total, 174 cases were available to study the U. S. coal mining practice involving the room-and-pillar system.

The mass of information received from this data collection, including survey questionnaires, mine maps, roof control plans and additional documentation was quite large and had the potential for difficulties in analysis. To insure maximum ease in utilization of this data, a data storage and retrieval system using the facilities of The Pennsylvania State University IMB 370/168 computer system has been developed. Survey responses were typed on the computer system, in formatted input, on a Remote Job Entry (RJE) link. The data storage system included survey responses as written in order to retain the

proper context of the information; as such this was an alpha-numeric storage system. Any portion of the questionnaire response could be recalled for compilation as needed for analysis. Long term storage for this information is on tape. The identification information on the original survey form was replaced by a coding to assure confidentiality of the respondents as promised on the survey form instructions. A copy of the computer program used, input data and sample output are shown in the Appendix.

The most important results obtained from this study are depicted in Figures 21 through 32 and the ranges of data are listed in Table 5. In the study performed only three failed pillar cases were available. As stated earlier, however, pillar failure cases were not essential for establishing safety factors for U.S. coal mining, contrary to an objective of the Salamon-Munro (1967) study in South Africa. There, no pillar strength formula was available at the time, while in the U.S. a number of pillar formulas have been used since 1911 and it was the aim of this study to select one that would be most economical yet safe for coal pillar design.

As will be seen from Figures 21-32, computer analyses were performed to provide information on the following aspects: location by state, type of coal seam, type of coal pillar, depth of overburden (depth below surface), seam and pillar height, entry width, pillar length, pillar width-to-height ratio and percentage extraction of coal.

A major conclusion from this survey was that there is no universally accepted design code for U.S. coal mining and no systematic design procedures are observed. In fact, regression analyses were performed to establish possible correlations between the various parameters. This proved to be inconclusive as can be seen from computer printouts Figures 33 through 39. This lack of statistical control is indicative of a need for a rational design approach.

In the next chapter, the data from the case histories will be used to recommend factors of safety in conjunction with a design code for room-and-pillar coal mining.

Table 5

Range of Room-and-Pillar Parameters Used in U.S. Coal Mining
(174 cases)

		<u>Typical</u>
Depth below surface	75-1500 ft (25-450 m)	500 ft (150 m)
Seam or pillar height	3-15 ft (1-4.5 m)	6 ft (2 m)
Entry width	12-27 ft (3 m-8 m)	16-20 ft (4.9-6.1 m)
Pillar width	15-75 ft (5 m-23 m)	48 ft (15 m)
Pillar length	20-90 ft (6 m-27 m)	60 ft (20 m)
Width to height ratio	2-16	8.0
Width to length ratio	1-3	1.25
Percentage extraction	25%-85%	50%
Shape of pillars	Rectangular	Rectangular

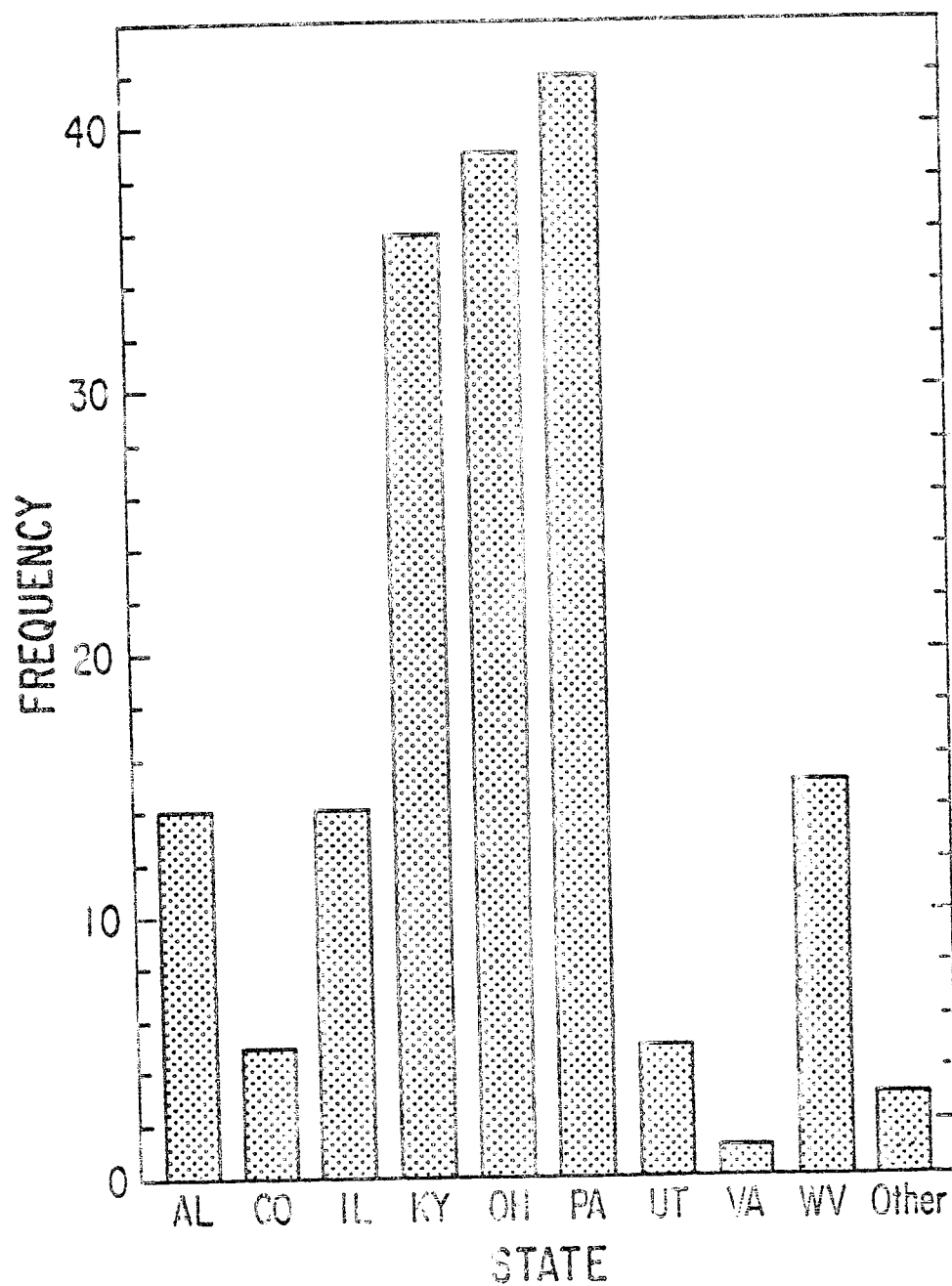


Figure 21

Penn State Survey: data distribution by state

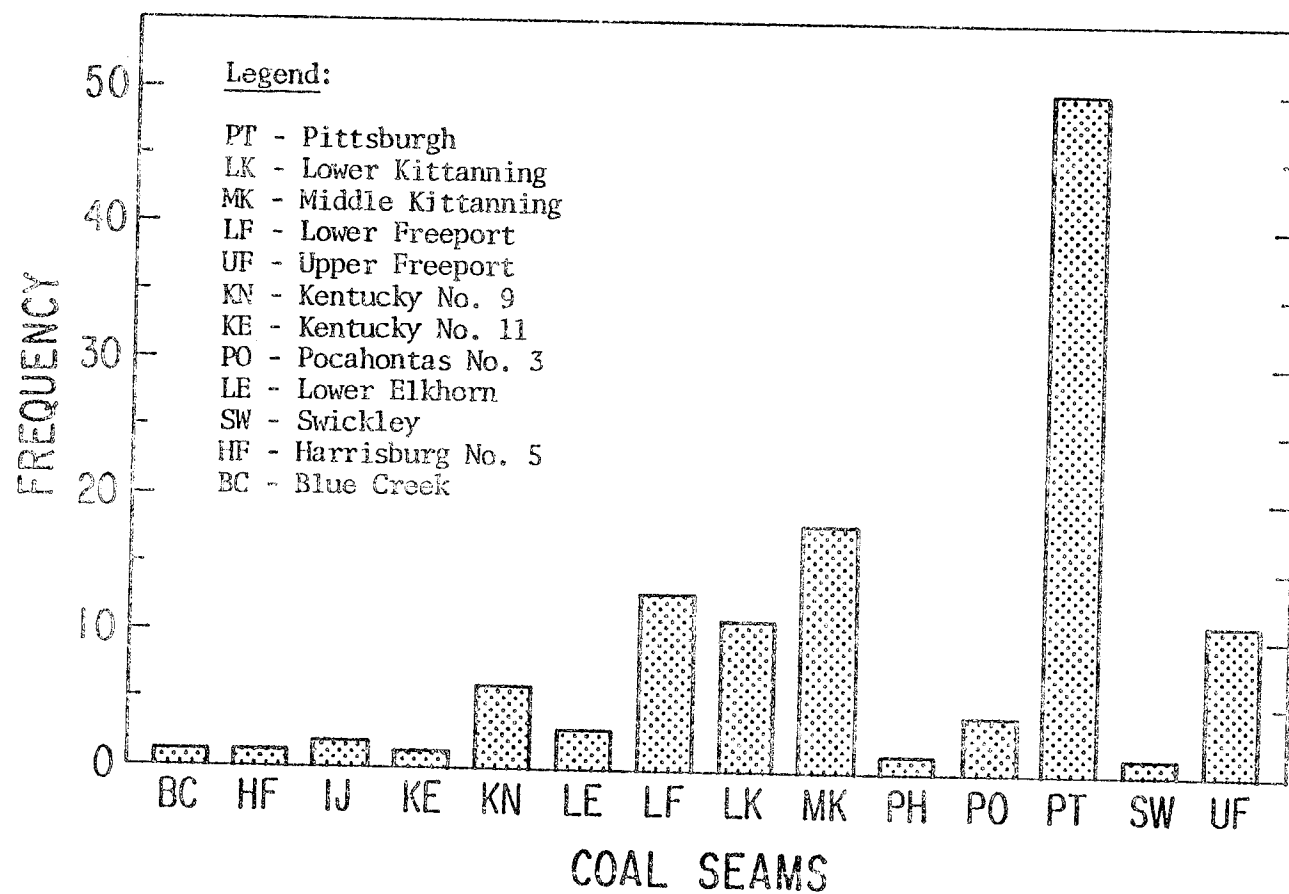


Figure 22
Penn State Survey: data distribution by coal seams

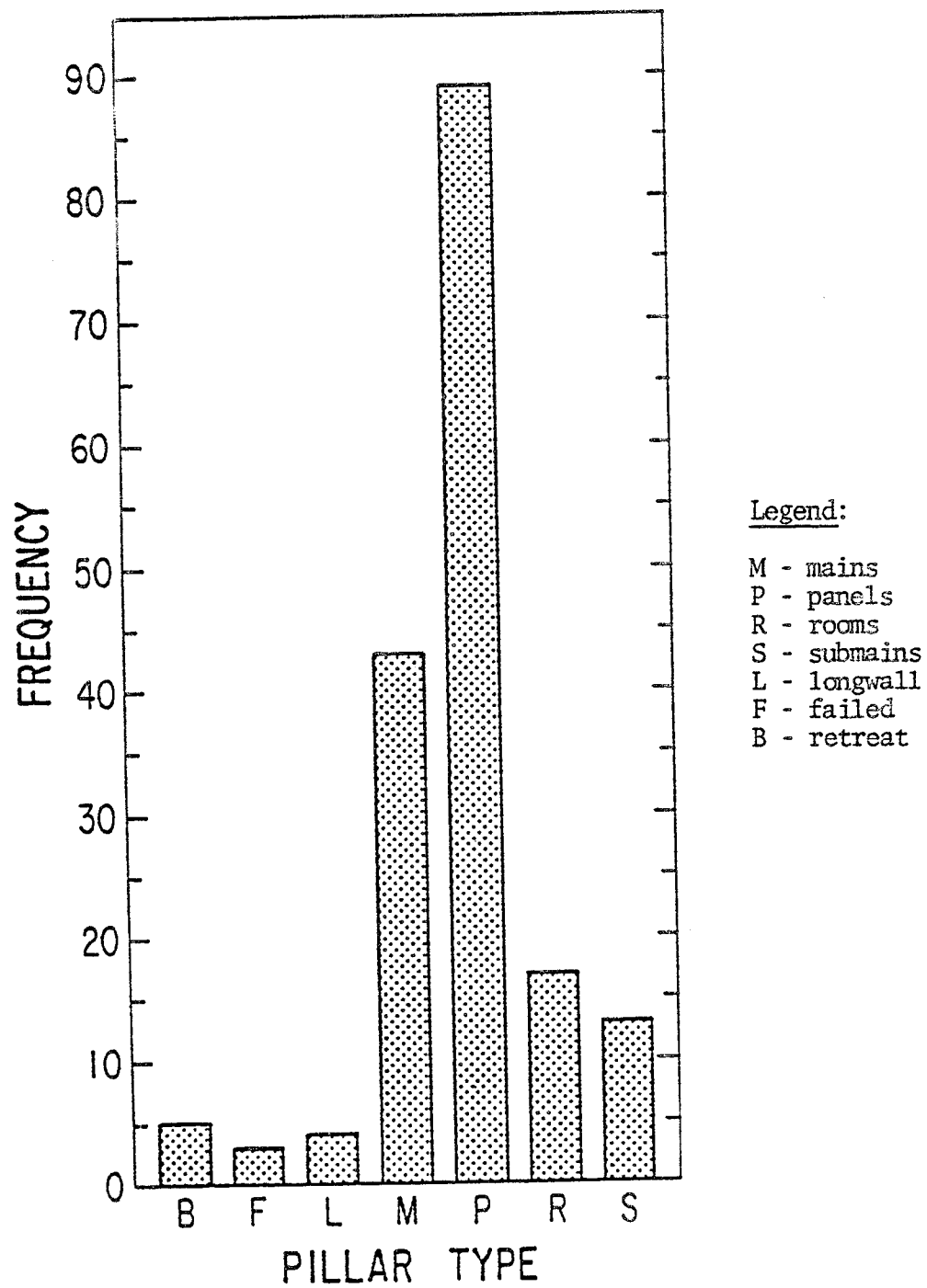


Figure 23

Penn State Survey: data distribution by pillar type

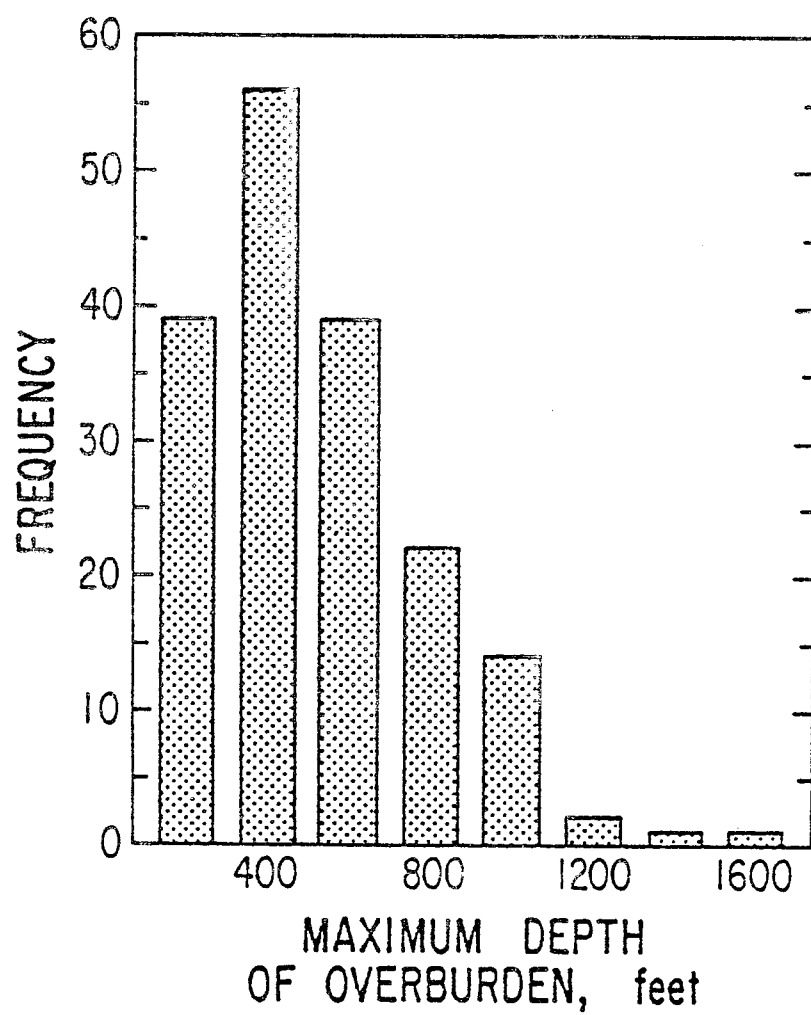


Figure 24

Penn State Survey: depth below surface

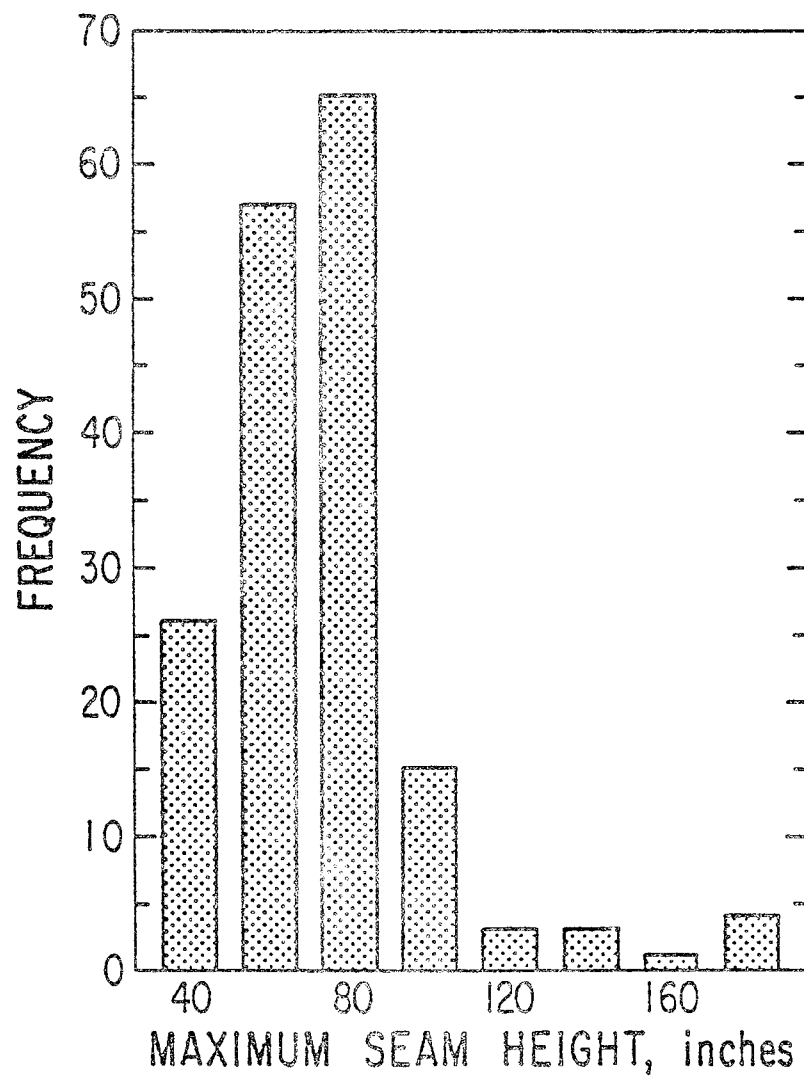


Figure 25

Penn State Survey: seam heights

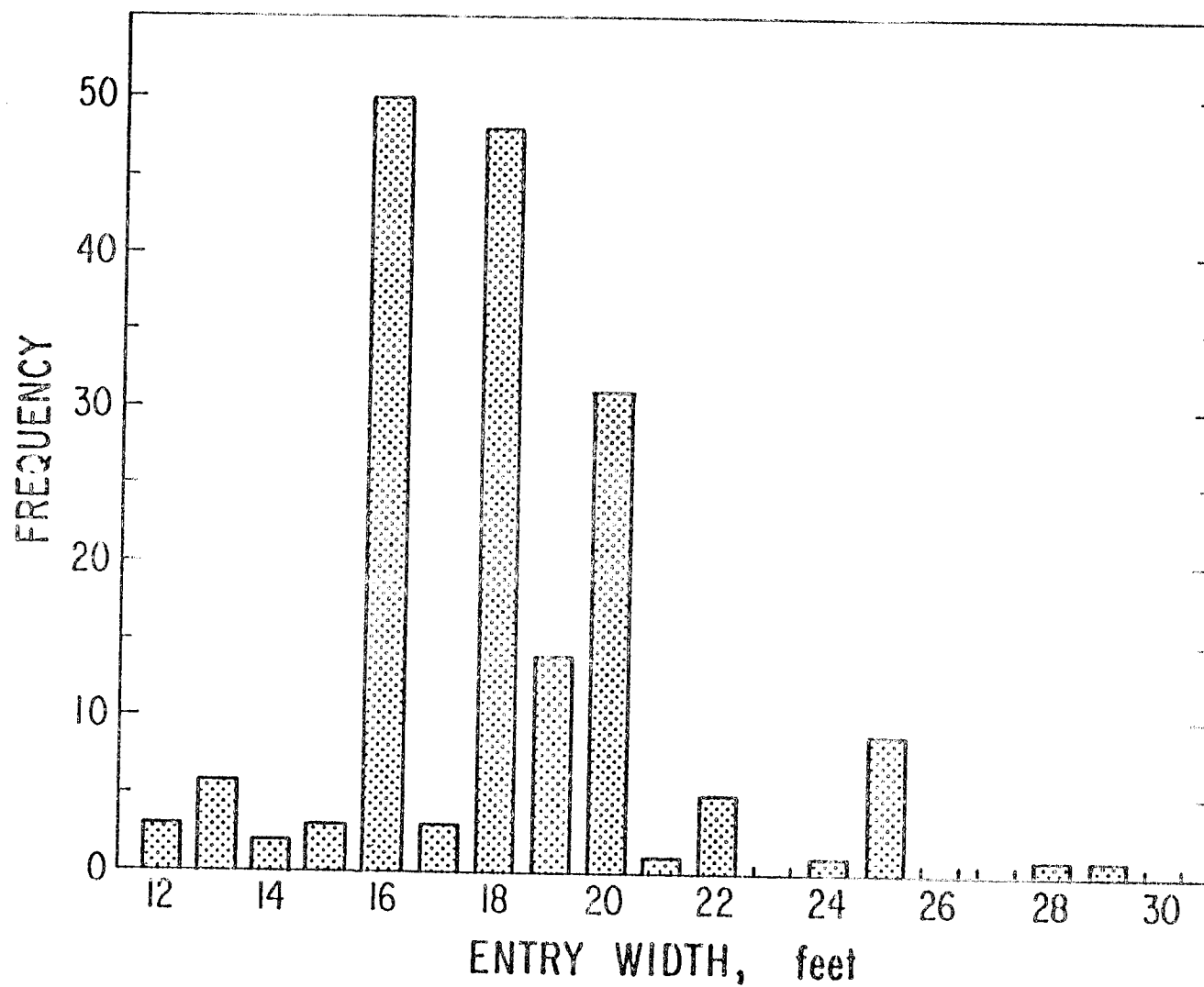


Figure 26

Penn State Survey: roof seams

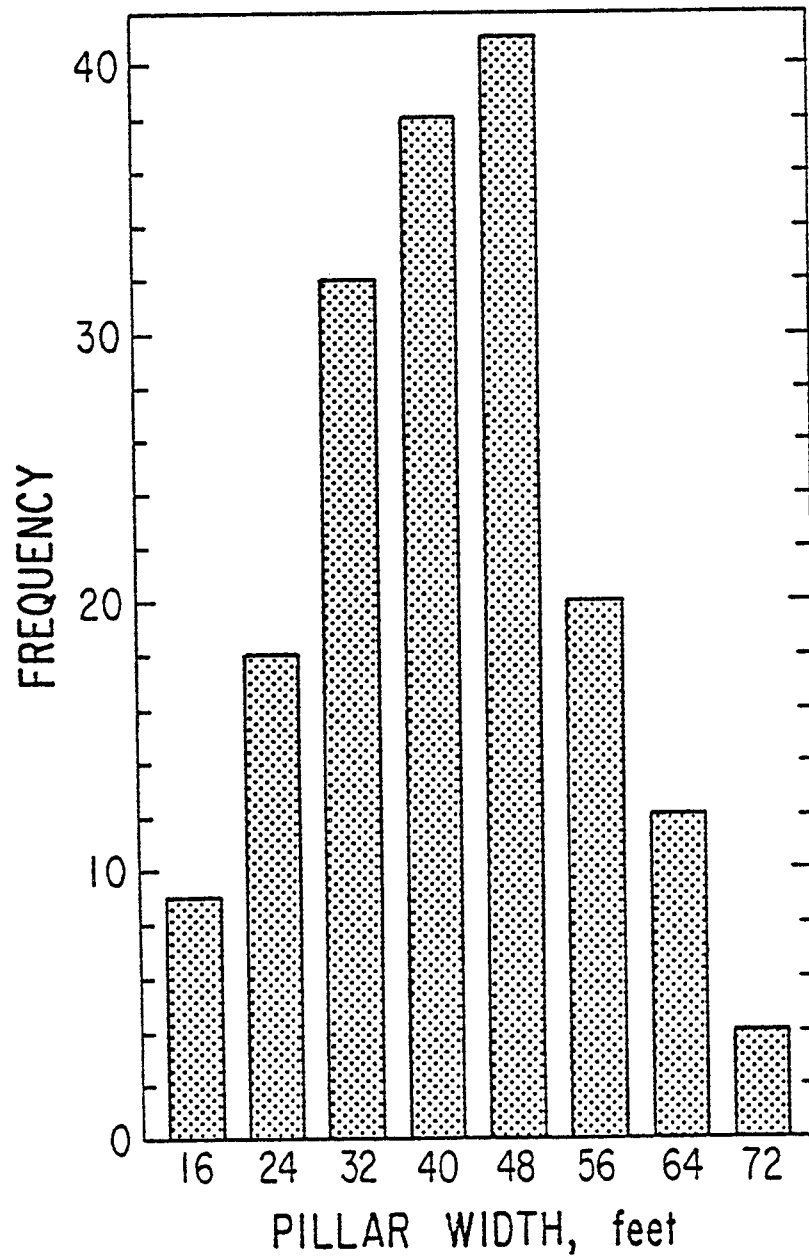


Figure 27

Penn State Survey: pillar width

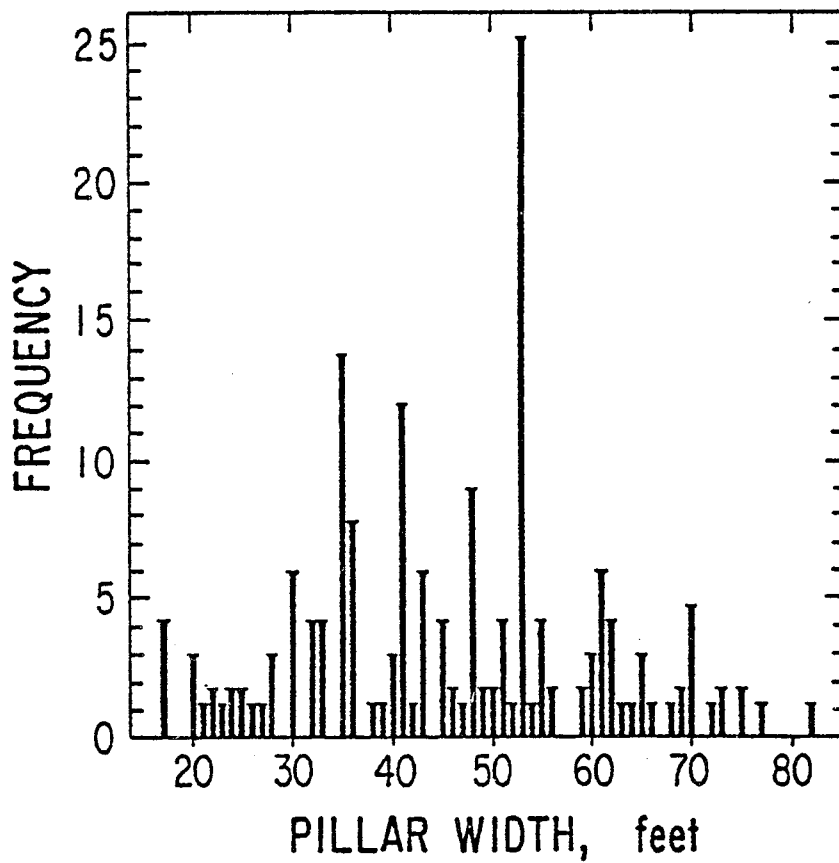


Figure 28

Penn State Survey: Detailed distribution of pillar widths

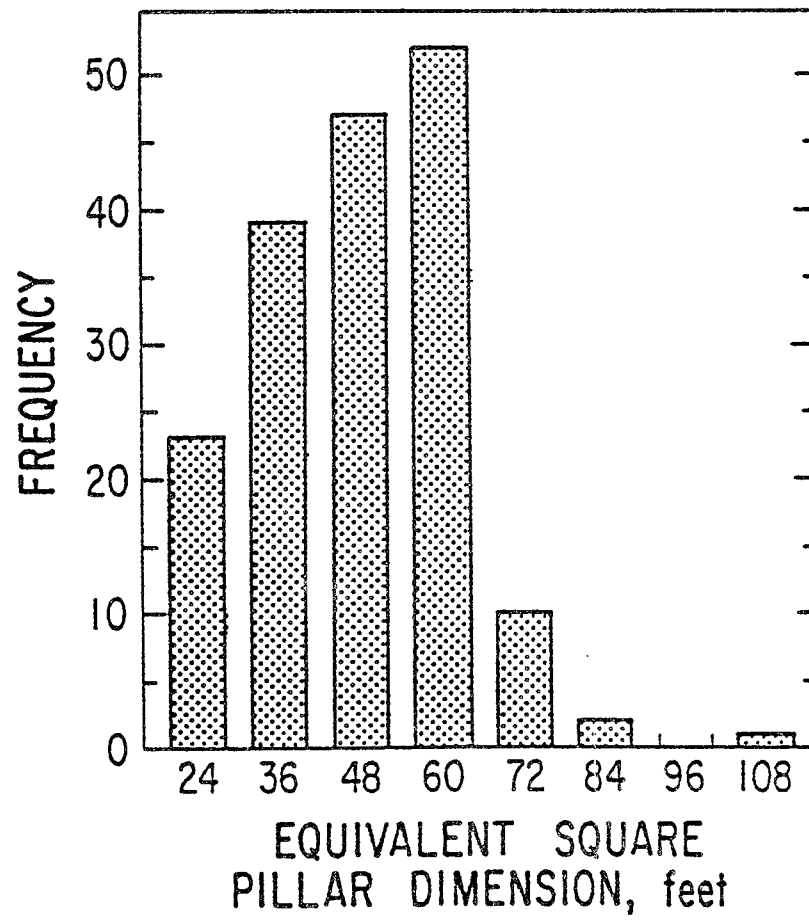


Figure 29

Penn State Survey: square root of areas of rectangular pillars

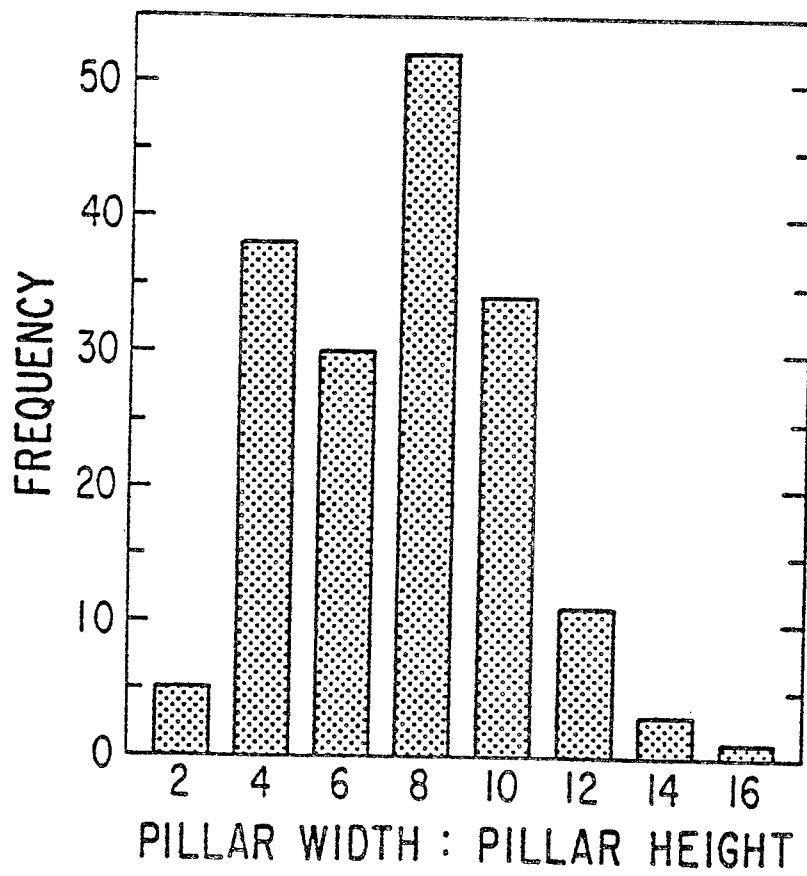


Figure 30

Penn State Survey: width-to-height ratios

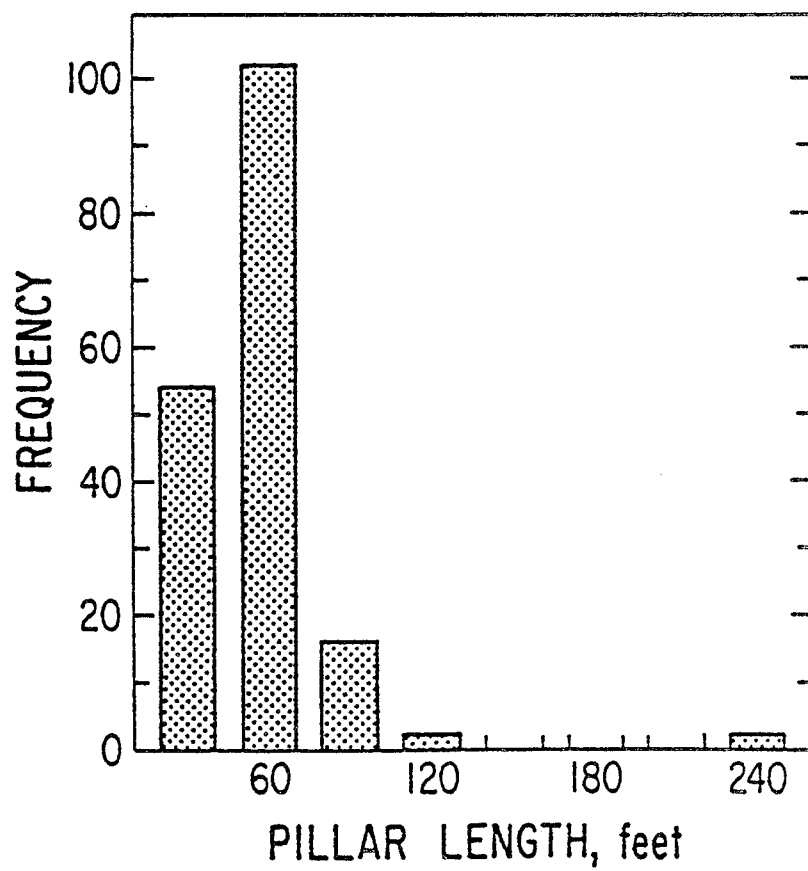


Figure 31

Penn State Survey: pillar lengths

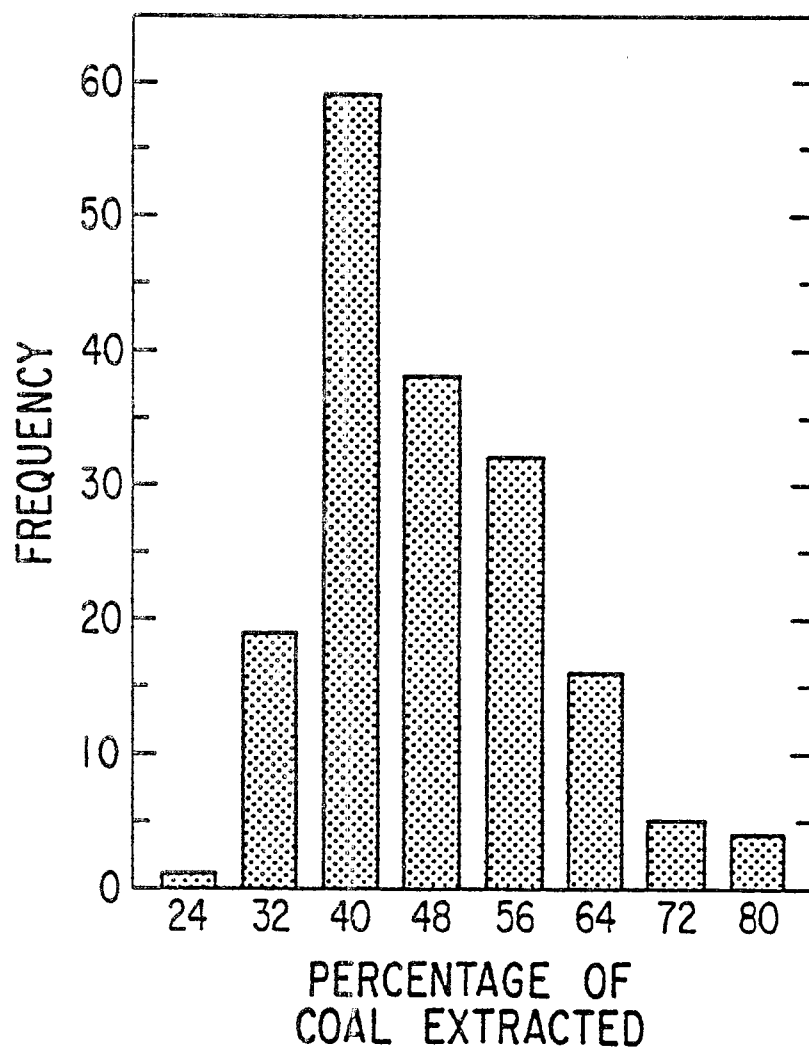


Figure 32

Penn State Survey: coal extraction

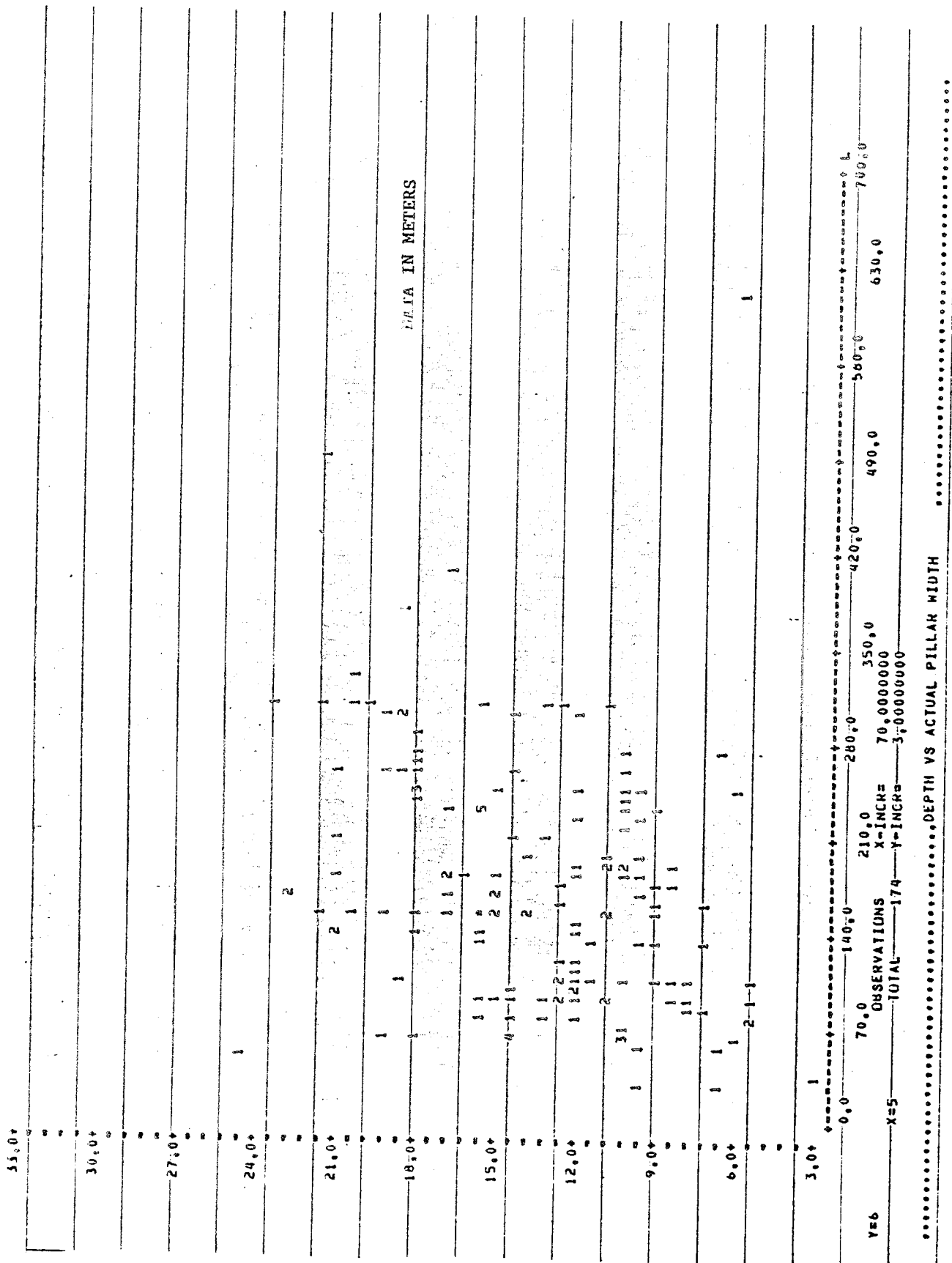


Figure 34

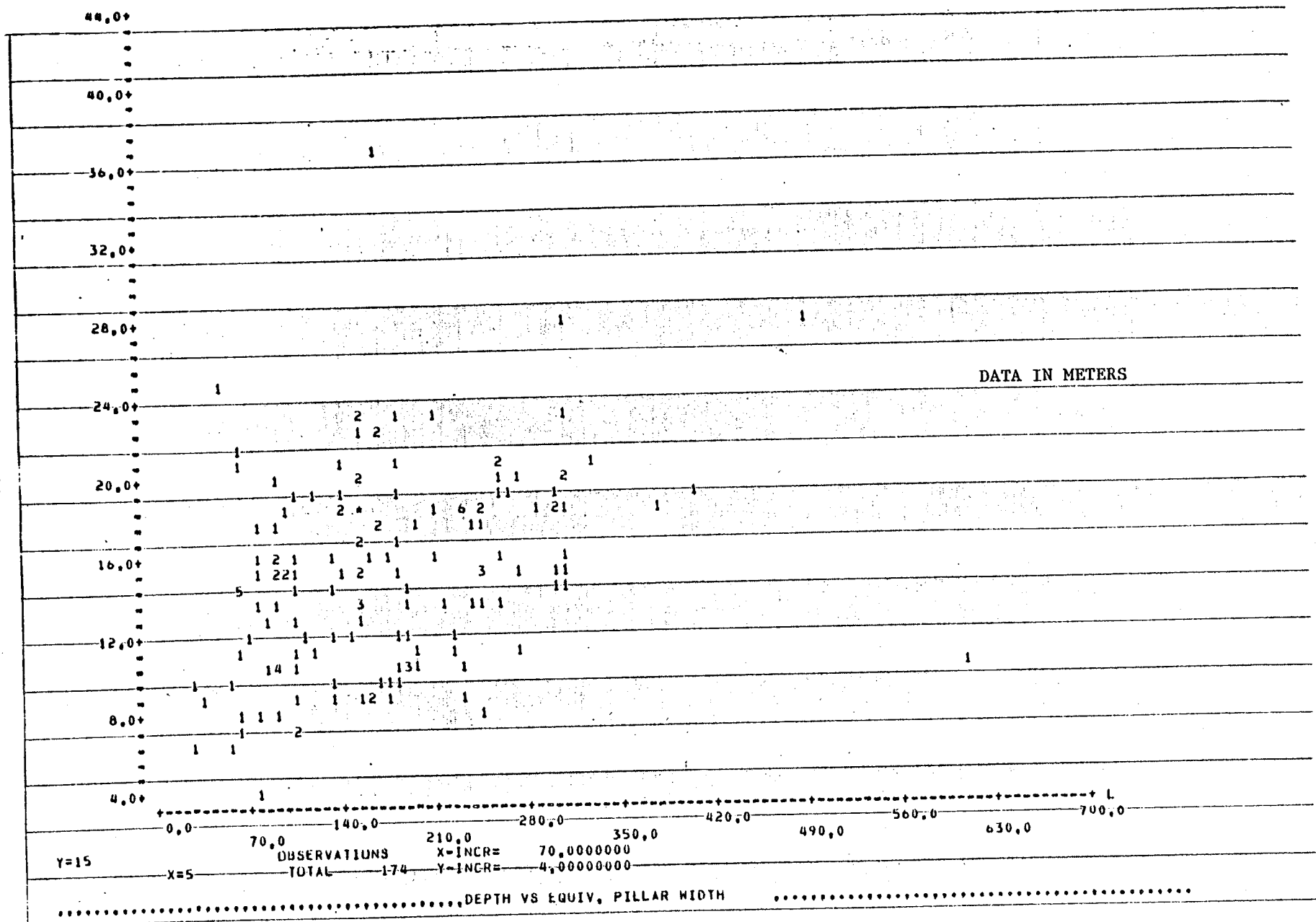


Figure 35

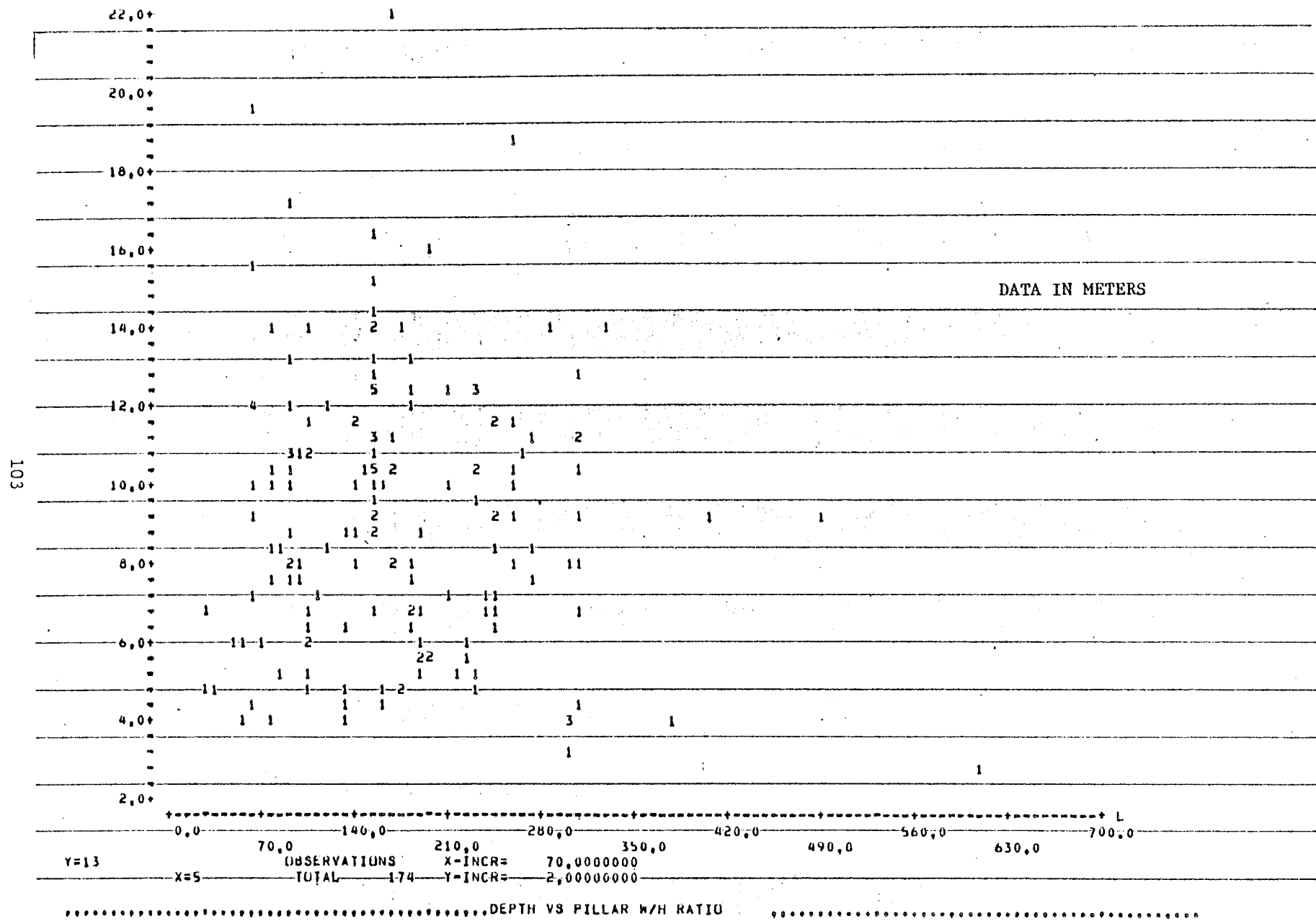


Figure 36

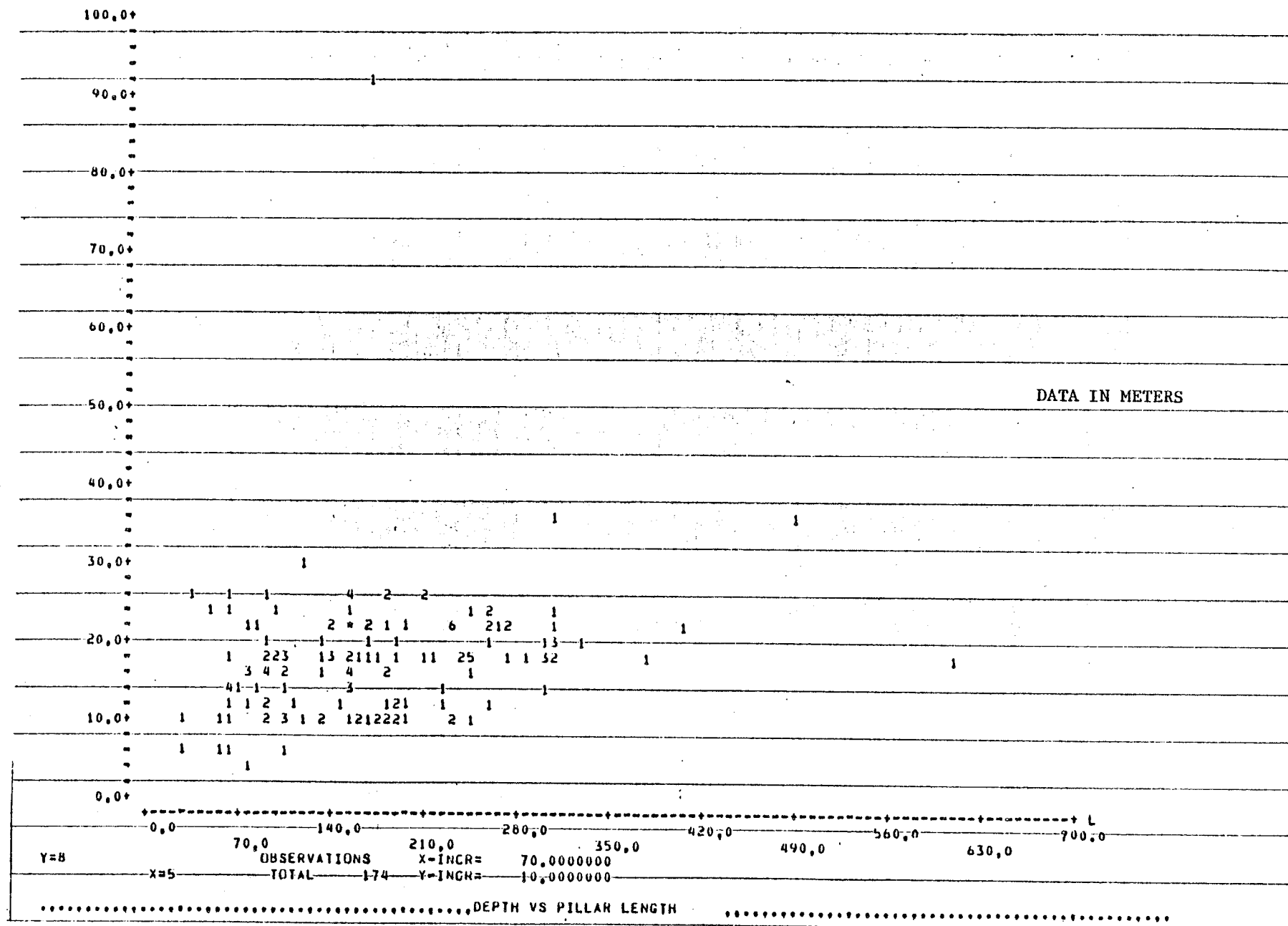


Figure 38

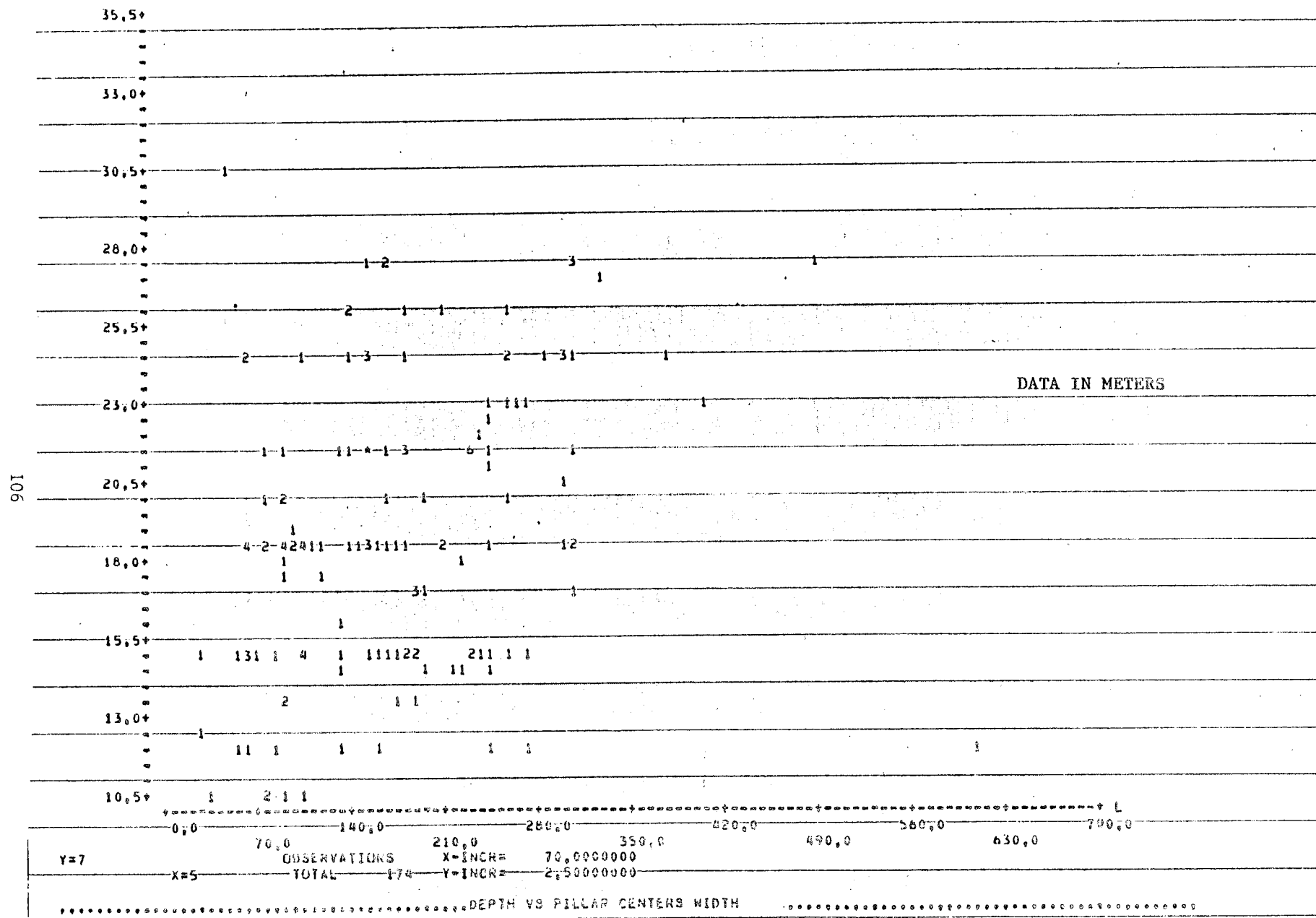


Figure 39

CHAPTER 5

A DESIGN CODE FOR ROOM-AND-PILLAR COAL MINING

The room-and-pillar mining method is used in the United States in some 90% of underground coal mining. Yet, systematic design guidelines are lacking both for roof control and pillar sizing. Since this method of mining will continue to be prominent in U.S. coal mining in the next two decades, in spite of interest in longwall mining, it is obvious that rational room-and-pillar design guidelines are needed.

Based on the findings presented in the previous chapters, it may be concluded that enough scientific and practical data are available to enable consolidating all this information together for the purpose of a design code for room-and-pillar mining. The aim of such a code would be to facilitate selection of roof spans in the coal mines and the sizing of coal pillars in accordance with the coal mining conditions in the United States. This means that the established mining practice and experience should be utilized in preparing the design guidelines. In addition, the emphasis should be on the research findings conducted in the United States because although some research performed in other countries may be interesting and relevant, it may not be directly applicable to U.S. coal mining practice. Finally, the objective is to incorporate as much up-to-date scientific knowledge as is compatible with the practical applications in coal mining to facilitate improved planning of new mines as well as optimizing the mining operations already in progress.

5.1 Basic Concepts

The room-and-pillar method of coal mining as used in the United States involves a system known as retreat mining. In this method, the rooms (openings) and pillars are developed as mining advances but subsequently the pillars are recovered "on retreat" which can result in a high percentage of coal extraction, e.g. 70%.

However, room-and-pillar mining techniques vary widely in the United States, dependent upon specific local conditions. The average coal extraction in the U.S. is just over 50% and the average percentage of mines practicing pillar extraction is only 26% (Kauffman et al., 1981). Consequently, it is necessary to differentiate between primary mining with no pillar extraction and retreat mining including primary development and partial or full pillar recovery on retreat. Clearly, primary mining only (74% in U.S.) requires more precise planning and design because significant coal reserves may be wasted. However, even in the case of full retreat mining, the main entries need be properly designed for their long-term use.

Accordingly, a design code for room and pillar mining should fulfill two functions: (i) enable planning of new mines, and (ii) enable assessing the stability of the current mining layouts and the effects of any improvements introduced.

While pillar stability is of much importance in this room-and-

pillar mining method, the stability of mine roofs is a crucial aspect of strata control in the United States. Hence, selection of roof spans and their reinforcement must be addressed in any design code. This is not a situation in other countries using the room-and-pillar method. There, the pillars receive the main attention as they are generally not recovered on retreat but left permanently in place.

Moreover, the design code will rely on certain geological data as well as the strength properties of the coal seam and the roof rock which are considered as reasonable to expect as being available both from a site exploration program and during mining. This includes the stratigraphic log of the roof strata; spacing, orientation and condition of the geological features (e.g. bedding and jointing), ground water condition, the strength of roof rock and the strength of the coal. The strength of the roof rock and coal can be determined either by laboratory testing or in the mine by the point-load index method.

It should be noted that the U. S. customary units (feet, inches, pounds per square inch, etc.) are used in this report because the mining industry has not as yet converted to the metric system.

5.2 Selection of Roof Spans

It is proposed that estimating safe roof spans in U.S. coal mines be achieved by means of an engineering rock mass classification system, specifically the Geomechanics Classification, as modified at Penn State for coal mining applications. Reliance on beam theories is not recommended due to uncertainties in the assumptions as well as in the input data (e.g. 'modulus of rupture').

As discussed in Chapter 2, rock classifications are well known empirical methods for assessing the stability of underground openings in rock. Developed primarily for the purposes of tunnel and chamber design in civil engineering, they have received increasing attention in recent years as a means of roof stability appraisal. In mining, a detailed study of the application of rock classifications to U.S. hard rock mining was initiated by the Bureau of Mines in 1980 while in coal mining, Penn State researchers have been developing the rock classification approach systematically since 1978. As a result, over 60 mining case histories have been compiled at Penn State enabling estimation of safe roof spans in coal mines by means of the Geomechanics Classification--also known as the Rock Mass Rating (RMR) System.

5.2.1 Geomechanics Classification

The Geomechanics Classification is presented in Table 1. In Section A, five parameters are grouped into five ranges of values. Since the various parameters are not equally important for the overall classification of a rock mass, importance ratings are allocated to the different value ranges of the parameters, a higher rating indicating better rock mass conditions.

To apply the Geomechanics Classification, the rock mass is divided

Table 1

GEOMECHANICS CLASSIFICATION OF JOINTED ROCK MASSES

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial compressive strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating	15	12	7	4	2	1	0	
2	Grill core quality RQD	90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%			
	Rating	20	17	13	8	3			
3	Spacing of discontinuities	>2 m	0.4 - 2 m	200 - 600 mm	60 - 200 mm	<60 mm			
	Rating	20	15	10	8	5			
4	Condition of discontinuities	Very rough surfaces. Not continuous. No separation. Unweathered wall rock.	Slightly rough surfaces. Separation < 1 mm. Slightly weathered walls.	Slightly rough surfaces. Separation < 1 mm. Highly weathered walls.	Slip-sensitized surfaces. Gauge < 5 mm thick. Separation 1-5 mm. Continuous.	Soft gouge > 5 mm thick OR Separation > 5 mm. Continuous.			
	Rating	33	25	20	10	0			
5	Groundwater	Inflow per 10 m tunnel length	None	<10 litres/min	10-25 litres/min	25 - 125 litres/min	> 125		
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0	0.0-0.1	0.1-0.2	0.2-0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating	15	10	7	4	0			

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-3	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-3	-23	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100-61	60-41	40-21	20-1	< 20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	< 40°	35° - 45°	25° - 35°	15° - 25°	< 15°

Table 2.

The Effect of Discontinuity Strike and Dip Orientations in Tunneling.

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0° - 20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Unfavourable

into a number of structural regions (units) so that certain geological features are reasonably uniform within each unit. Although rock masses are discontinuous in nature they may nevertheless be uniform within regions, for example, the type of rock in the mine roof will be uniform within the various stratigraphic units.

After the structural unit has been identified, the classification parameters for each region are determined. They are:

- uniaxial compressive strength of rock material
- rock quality designation RQD (a standard core quality index based on core recovery procedure which incorporates only those pieces of core that are 4 inches or greater in length)
- spacing of discontinuities
- condition of discontinuities (continuity, roughness, separation, infilling)
- orientation of discontinuities
- groundwater conditions

All these parameters are determined in the course of a standard geological survey. These field measurements do not require any special instruments or extra time beyond the usual geological practice.

Once the classification parameters are determined, the importance ratings are assigned to each parameter according to Table 1, section A. Here, five parameters are grouped into five ranges of values.

After the importance ratings of the classification parameters have been established, the ratings for the five parameters listed in Section A of Table 1 are added up to yield the basic rock mass rating for the structural unit under consideration.

The next step is to include the effect of the strike and dip of discontinuities by adjusting the basic rock mass rating according to Section B of Table 1. This step is treated separately because the influence of discontinuity orientations depends upon the engineering applications, e.g., tunnel (mine), slope, or foundation. It will be noted that the "value" of the parameter, "discontinuity orientation," is not given in quantitative terms but by qualitative descriptions such as "favorable" or "fair." To facilitate a decision as to whether strike and dip orientations are favorable or not, reference should be made to Table 2 (at the bottom of Table 1). In the case of coal mine roof, the main discontinuities are the bedding planes oriented horizontally. This orientation is considered as unfavorable and requires an adjustment of -10 in accordance with Section B of Table 1. For mining applications, other adjustments could be made such as taking into consideration the effect of stresses at depth (change of stress during mining) and/or weathering of the roof strata (if shale) for long-life roof stability (see Chapter 2).

After the adjustment for discontinuity orientations has been made, the rock mass is classified according to Section C of Table 1 which groups the final (adjusted) rock mass ratings (RMR) into five rock mass classes. Note that the rock mass classes are in groups of 20 ratings each. Next, Section D of Table 1 gives the practical meaning of each rock mass class by relating it to specific engineering problems. In the case of mine roofs, the output from the Geomechanics Classification is the stand-up time and the maximum stable roof span, for a given rock mass rating.

Before discussing the practical meaning of rock mass classes in coal mining, it may be useful to consider an example to demonstrate the above classification procedures. A claystone structural unit in a coal mine roof was selected for the classification example given in Table 3.

It will be seen from Table 3 that this structural unit is classified as having a final (adjusted) rock mass rating (RMR) of 35. With reference to Section C of Table 1, RMR = 35 falls into Class IV which has a description: "poor rock".

Table 3

Example for Classification of Claystone Structural Unit

<u>Parameter</u>	<u>Value</u>	<u>Rating</u>
1. Strength of intact rock	3,500 psi (24 MPa)	2
2. Drill core quality RQD	50-75%	13
3. Spacing of bedding planes	2 in - 1 ft (50 mm - 0.3 m)	10
4. Condition of discontinuities	Continuous, slightly weathered, separation >1 mm, smooth surfaces, no infilling	10
5. Ground water	No inflow	10
6. Orientation of discontinuities	Unfavorable	<u>-10</u>
Total (adjusted) rock mass rating (RMR)		35
Class IV Poor Rock		

For coal mine roofs, as in the case of tunnels, although the average values of stand-up times and roof spans are given in Section D of Table 1, it is more appropriate to refer to Figure 1 where a relationship is given between the stand-up time and the span for various rock mass ratings.

In the case of the present example, a roof strata rating of RMR = 35 will mean that the maximum possible span in this rock mass is 18 ft (read off from the interception of rating line of 35 with the top line in Figure 1). On the other hand, if the roof span is 4.5 ft, it will last unsupported indefinitely (read off from the intersection of rating line of 35 with the bottom line in Figure 1).

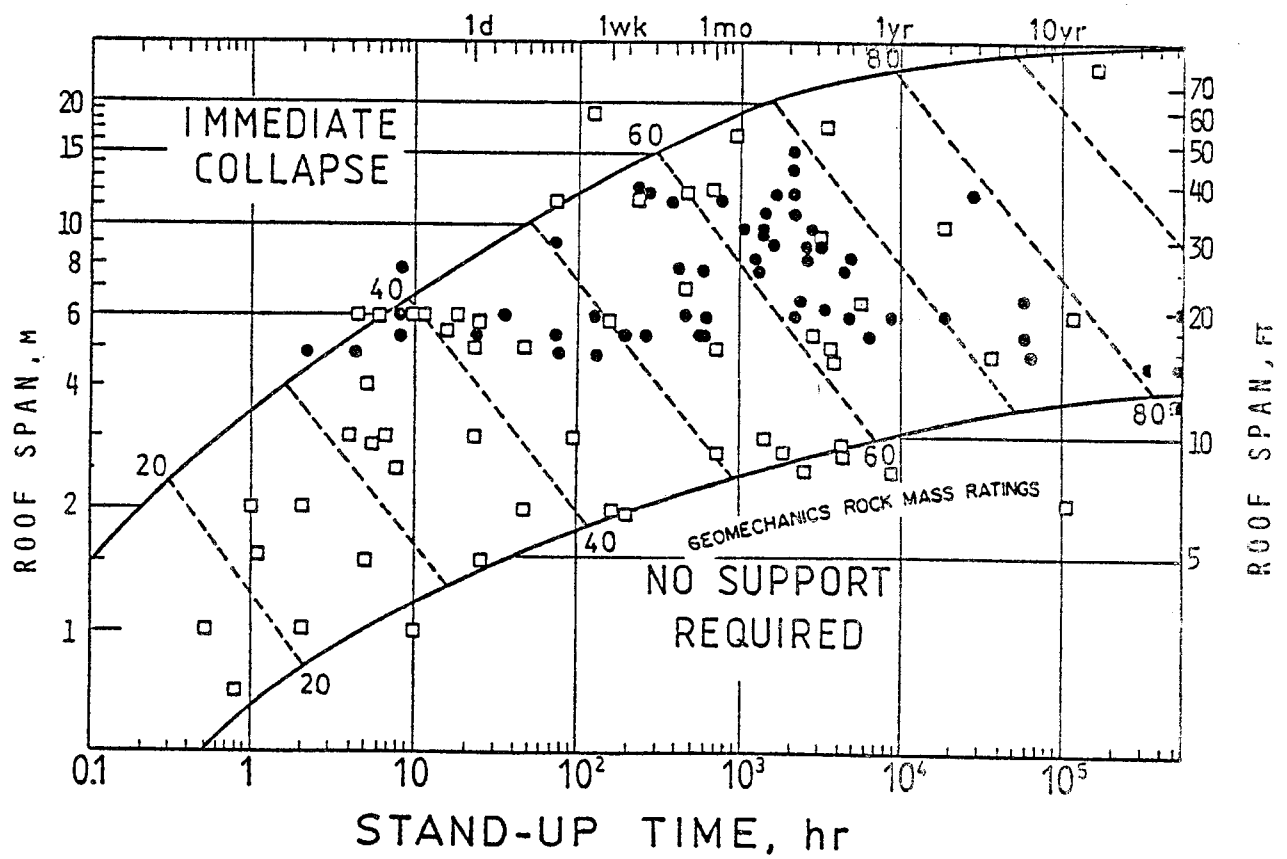


Figure 1. Geomechanics Classification - output for mining and tunneling. Black dots represent coal mining cases in the United States. White squares are tunneling cases. The contour lines are limits of applicability.

If the mining practice requires a roof span of, say, 15 ft in this rock mass of RMR = 35, then it will be seen from Figure 1 that for a span of 15 ft and a rating of 35, the stand-up time is about 8 hours. This means that a 15 ft span can be left completely unsupported 8 hours but it may be expected to collapse after that period of time. What can be done to insure a long-term stability? The answer lies in appropriate support measures.

The Geomechanics Classification provides guidelines for the selection of roof support to ensure long-term stability of various rock mass classes. These guidelines are not given here* except as an example in Table 4 because it is recognized that different objectives must be considered in coal mining by comparison with tunneling. For coal mines appropriate factors such as use and life of entries as well as the effect of intersections must be included in determining roof support measures. In fact, once sufficient information is available on the actual roof behavior in the coal mine, the rock mass classes can be interpreted, by the mine personnel, in terms of the support requirements.

To summarize, the Geomechanics Classification is useful in coal mining applications in three respects: (i) it enables estimating a safe roof span, (ii) where a roof span is decided upon from other considerations, it enables estimating the stand-up time, and (iii) it provides guidelines for roof support.

5.2.2 Applications

The Geomechanics Classification was applied in a number of coal mines in Pennsylvania and West Virginia. It was found that both the top coal and the claystone were of poor rock mass quality, the shale was of a fair quality while the limestone was of very good quality.

Considering the three structural units: top coal, claystone and shale as the overall mine roof and weighting the rock mass ratings in accordance with the thickness of each unit, the overall weighted roof ratings were developed.

*Support guidelines for coal mine roofs in the United States in terms of the rock mass rating RMR were developed at Penn State in a doctoral study by E. Unal (1982): "Design Guidelines and Roof Control Standards for Coal Mine Roofs."

Table 4

ENTRY WIDTH: 20-FT

ROCK CLASS	ROCK MASS RATING (RMR)	ROCK LOAD HEIGHT (FT)	SUPPORT SPECIFICATIONS		ENTRY ALTERNATE SUPPORT PATTERNS		REMARKS
			MECHANICAL BOLTS	RESIN BOLTS	MECHANICAL BOLTS/POSTS	RESIN BOLTS/POSTS*	
I VERY GOOD	90	2.0	L: 2.5' S: 5' x 5' G: 40 φ: 5/8" C: 6.2 tons			8	1) Mining Regulations Considered
	80	4.0	L: 2.5' S: 5' x 4.5' G: 60 (40) φ: 3/4" (7/8") C: 11 tons	L: 2.5' S: 5' x 5' G: 60 φ: 3/4" C: 12 tons			2) Post Specifications** φ _p : 5.5"; S _p : 10'
II GOOD	70	6.0	L: 3.0' S: 4' x 4' G: 60 φ: 3/4" C: 10 tons	L: 3.0' S: 5' x 5' G: 60 φ: 3/4" C: 18 tons	SF = 1,3 		3) φ _p : 6.5"; S _p : 10'
	60	8.0	L: 4.0' S: 5' x 5' G: 60 φ: 5/8" C: 9 tons	L: 4.0' S: 5' x 5' G: 60 φ: 1" C: 23.7 tons	2 		4) φ _p : 6.5"; S _p : 7.5'
III FAIR	50	10.0	L: 5.0' S: 5' x 5' G: 40 φ: 3/4" C: 8 tons	L: 4.0' S: 5' x 4' G: 60 φ: 1" C: 23.7 tons	3 		5) φ _p : 5.0"; S _p : 5'
	40	12.0	L: 6.0' S: 5' x 5' G: 40 φ: 3/4" C: 7 tons	L: 4.0' S: 4' x 4' G: 60 φ: 1" C: 23.7 tons	4 	9 	6) φ _p : 6.0"; S _p : 4.5'
IV POOR	30	14.0	L: 7.0' S: 5' x 5' G: 40 φ: 5/8" C: 6 tons	L*: 5'/2.5' S: 5' x 5' G: 60 φ: 3/4" C: 12 tons	5,7 	10 	7) Planks Recommended
	20	16.0	L: 8.0' S: 4' x 4.5' G: 40 φ: 5/8" C: 5 tons	L*: 5'/2.5' S: 5' x 5' G: 60 φ: 3/4" C: 12 tons	6,7 	11,7 	8) Not Economical

* Bolt Length - 5.0'
Resin Length - 2.5'

** Post diameter should be checked with H/R

In any engineering design, findings should always be cross-checked by different approaches. For roof span selection in coal mines, practical experience and mining regulations should be used to assess the predictions from the Geomechanics Classification.

The Federal Law (CFR30) stipulates that openings should not exceed 20 ft in width where roof bolting is the sole means of support or 30 ft where roof bolts and other support, such as wood posts, are used. Section 75 of CFR30 deals specifically with roof control in underground coal mines and specifies that in no case should the length of roof bolts be less than 30 inches plus 12 inches if anchored in the stronger strata to suspend the immediate roof. The bolt spacing and the distance between the bolt and the rib or the face should not be less than 5 ft.

5.3 Pillar Sizing

The design of mine pillars involves the determination of proper sizes of pillars compatible with the expected load. Thus, in deciding the most suitable dimensions for mine pillars, the following need be considered: the pillar load (the stress on pillars), the pillar strength and the factors of safety.

There are two approaches to coal pillar design. The ultimate strength approach contends that pillars will fail when the applied load reaches the compressive strength of the pillars. It presumes that the load-bearing capacity of a pillar reduces to zero the moment its ultimate strength is exceeded. The progressive failure approach emphasizes the non-uniform stress distribution in the pillar. Failure initiates at the most crucial point and propagates gradually to ultimate failure. In the United States, the 1969 Coal Mine Health and Safety Act prescribes the prevailing safety concept for underground structures (i.e., entries and pillars), namely, that the structural elements (roof, floor and ribs) of the opening must be kept in perfect or nearly perfect condition to be safe for use. Thus, the U.S. safety concept uses failure initiation as the criterion rather than ultimate failure, contrary to what has been practiced in other countries. This results in an additional margin of safety.

5.3.1 Pillar Load

The simplest approach to determine the pillar load, or more correctly the average pillar stress, is by the tributary area theory. If a number of simplifying assumptions, such as listed in Section 5.4 are made, the pillar load can be calculated from (see Figure 2):

$$S_p = \frac{1.1 H (w + B)(L + B)}{w \times L} \quad (1)$$

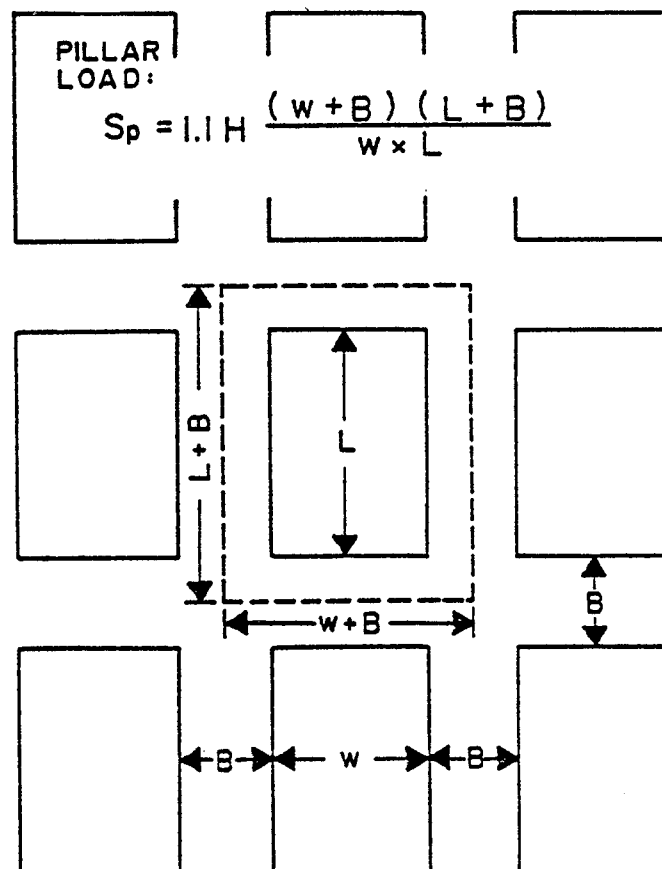
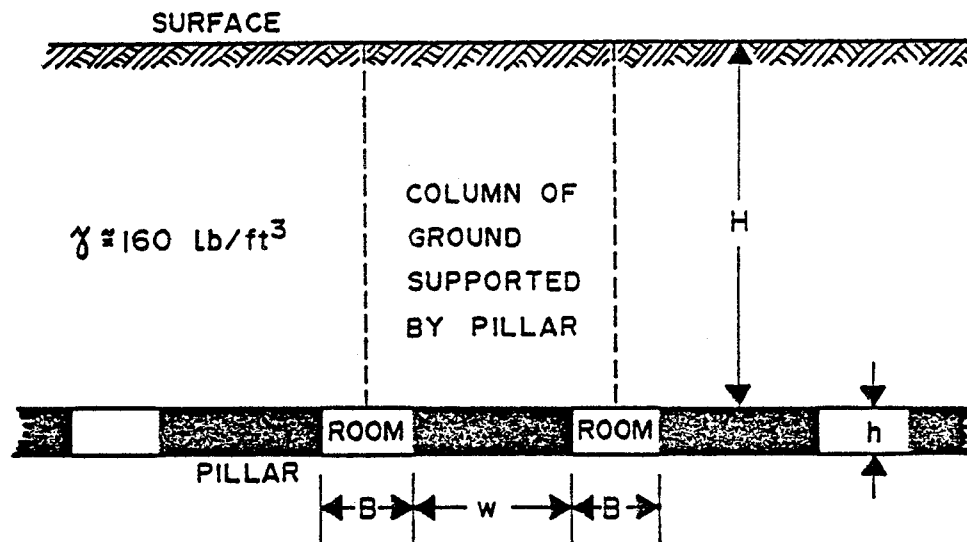


Figure 2. Tributary area concept for pillar loading.

where

S_p is the pillar load (average pillar stress) in psi

H is the depth below surface in ft

B is the width of the opening (room or entry) in ft

w is the pillar width in ft

L is the pillar length in ft

It should be noted that the tributary area theory represents the upper limit of the average pillar stress. In fact, measurements have shown that the theory overestimates the pillar load by about 40%.

In equation (1), the vertical overburden pressure increases at a rate of 1.1 psi per foot of the depth which is substantiated by field measurements.

For square pillars, that is when $w = L$, equation (1) becomes:

$$S_p = 1.1 H \left[\frac{w + B}{w} \right]^2 \quad (2)$$

If the term "extraction" e is introduced (100e is percentage extraction) which is defined as the ratio of the mined out area to total area, then for square pillars

$$e = 1 - \left[\frac{w}{w + B} \right]^2 \quad (3)$$

Thus, the pillar load may also be expressed as:

$$S_p = \frac{1.1 H}{1 - e} \quad (4)$$

5.3.2 Pillar Strength

The strength of coal pillars, that is the ultimate load or bearing capacity per unit area is dependent upon three elements: (i) the size or volume effect (strength reduction from a small laboratory specimen of coal to a full size pillar; (ii) the effect of pillar geometry (shape effect); and (iii) the properties of the coal material. For U.S. coal mining conditions, it is believed that, of the many formulas discussed in Chapter 3, three formulas are appropriate, all of which incorporate the above elements:

(I) Obert-Duvall/CSM Formula

$$\sigma_p = \sigma_1 (0.778 + 0.222 w/h) \quad (5)$$

(II) Holland/Greenwald/USBM Formula

$$\sigma_p = \sigma_1 \sqrt{w/h} \quad (6)$$

(III) Bieniawski/PSU Formula

$$\sigma_p = \sigma_1 (0.64 + 0.36 w/h) \quad (7)$$

where σ_1 is the unit strength of the rock mass (the uniaxial compressive strength) of cubical coal pillars, i.e. pillar width w equals pillar height h at the critical size.

The concept of the "critical size" is based on the observation that the strength decreases with increasing specimen size until it becomes constant from a certain "critical specimen size" onwards. This means that these data could be directly applied to full size pillars. The critical size phenomenon has been observed in a number of rock masses including coal. It was shown (Bieniawski, 1967) for coal to be about 5 ft but Hustrulid (1975) has indicated that the scatter in the experiments was such that the critical size of 3 ft is equally appropriate for coal.

For scaling of laboratory result, Hustrulid recommended the following:

$$\sigma_1 = \frac{k}{\sqrt{36}} \quad h > 36 \text{ inches} \quad (8)$$

or

$$\sigma_1 = \frac{k}{\sqrt{h}} \quad h < 36 \text{ inches} \quad (9)$$

where

σ_1 = uniaxial compressive strength (psi) of a cube having side length h

h = sample height or seam height (ins)

k = constant for each particular coal

The constant k can be determined by laboratory tests from the Gaddy (1956) equation:

$$k = \sigma_c \sqrt{D} \quad (10)$$

where σ_c is the uniaxial compressive strength of laboratory specimens having a diameter D or cube side dimension D .

It should be noted that equation (8) through (10) serve to estimate

the unit strength of the coal strata σ_1 , by scaling the strength from the laboratory size to the critical size applicable in the field. Once this size effect is determined, the shape effect is considered by the pillar design formulas represented by equations (5), (6), and (7). These three formulas are recommended because they were developed or applied extensively in the United States and they will enable effective cross-checking of the results.

Furthermore, it must be emphasized that the scaling of the laboratory determined results to field values has not only the economical merit (in situ large scale tests are very expensive and time consuming) but has also been well documented to be acceptable for practical engineering purposes (Hustrulid, 1975). However, since there is much variation in the coal strength from various U.S. seams, it is important that the k values, which represent the properties of coal, are determined for each mine locality.

Finally, it should be noted that when the strength of rectangular pillars is considered, the pillar strength formulas will utilize for w the pillar least dimension that is as if the pillar were of a square cross-section. Yet, in situ experiments have shown (Wagner, 1974) that pillars of rectangular cross-section are about 40 percent stronger than square pillars of the same width and height because of the relatively smaller perimeter of square pillars. It was found that although the perimeter of a pillar is capable of carrying relatively little stress, this portion of the pillar provides lateral confinement which enhances the strength at the center of the pillar.

A comparison of the various pillar strength formulas is given in Section 3.5 of Chapter 3. Another comparison is depicted in Figure 3. It will be noted that the Bieniawski/PSU formula predicts the highest strength. At the same time, the form of the Holland formula is such that it will become very conservative at high width-to-height ratios. Yet, it is at these high ratios that the strength increases rapidly as shown in Figure 3. It is therefore believed that the PSU formula, although predicting the highest strength of the three approaches, is justified to use because of the strength reserves on the following counts:

- (i) the tributary area theory overestimates the pillar load;
- (ii) rectangular pillars are considerably stronger than the square pillars of the same width and height;
- (iii) high width-to-height ratios lead to rapid strength increase.

It is thus clear that the PSU formula while safe for the above reasons will also lead to an improved design of room-and-pillar layouts by permitting the highest coal extraction ratios.

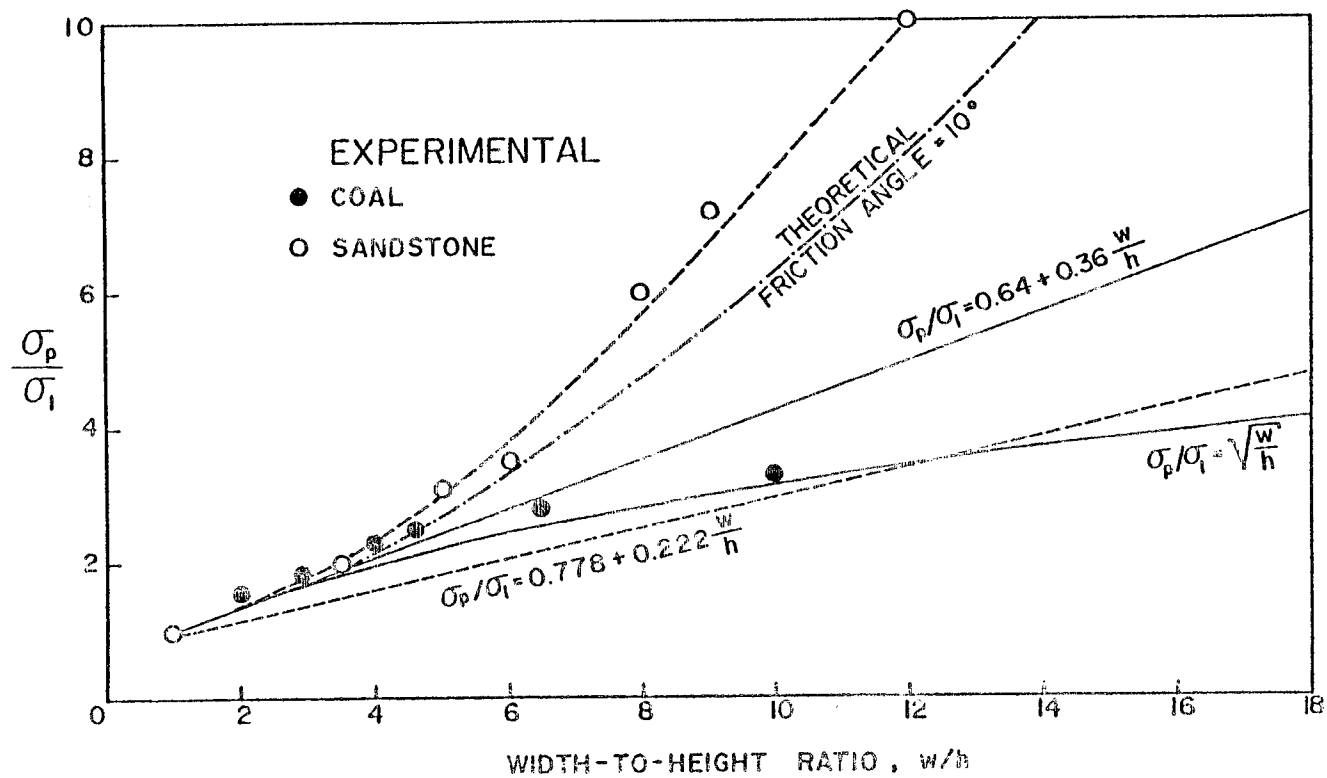


Figure 3. Comparison of pillar strength formulas with respect to w/h ratio

5.4 Factors of Safety

A factor of safety is defined as the ratio of the strength of a pillar to the load acting on it. Thus:

$$F = \frac{\sigma_p}{S_p} \quad (11)$$

A factor of safety is necessary because in order to derive the load and strength formulas presented in the previous section, certain assumptions had to be made.

A structure is defined as stable when $F > 1$. In engineering practice the factor of safety used is smaller when the conditions are well explored and understood. Sarcastically the factor of safety may be called a "factor of ignorance". Its values, which are derived from experience, take into consideration the likely accuracy of the predicted value of the strength and that of the expected load.

It is important to note that if the factor of safety is unity, the probability of failure is 50%. The factor of safety should be greater than unit to achieve a low probability of failure. What probability of failure is acceptable in coal mining?

Before considering a suitable factor of safety for room-and-pillar coal mining, the assumptions and uncertainties should first be considered.

5.4.1 Assumptions for Pillar Load Determination

1. It is assumed that an element of ground at a depth is subjected to vertical pressure only. However, besides the vertical pressure, lateral pressure also exists. The magnitude of this lateral pressure may vary from zero to a value greater than the vertical pressure, depending on local conditions.

2. It is assumed that the vertical pressure is evenly distributed over a mine area. Yet, certain regions underground may be partially relieved from vertical pressure as a result of "bridging" of superincumbent strata while others may be subjected to increased pressure, namely, if they are the abutments of such bridges. Moreover, it is known that the ground contains faults, slips, bed separation planes and other geological features which not only disturb the homogeneity and anisotropy of the ground but may also be associated with additional inherent stress.

3. It is assumed that each pillar supports the column of rock over an area which is the sum of the cross-sectional area of the pillar plus a portion of the room area, the latter being equally shared by all neighboring pillars. However, this is not valid if the area of development is small since the pillars in the center of the excavation are under more stress than the pillars near the sites. It is usually only accepted as valid if the mined-out area is greater than the depth below surface.

4. It is assumed that the load is uniformly distributed over the cross-sectional area of the pillar. However, research has shown that:

- (a) the stress is not evenly distributed over the cross-section of an individual pillar, the maximum stress occurring at the edge of the pillar. The maximum stress in pillars occurs in the corners formed by the intersection of three orthogonal planes, namely, two sidewalls of the pillar and the roof or the floor. The actual value of the difference between average stress and maximum stress, i.e., stress concentration factor, is not known but may be of the order of 3;
- (b) the stress on pillars increases with percentage extraction; and
- (c) the stress distribution in pillars depends upon the ratio of pillar width to pillar height.

It must be emphasized, however, that the conclusions in item 5 have been drawn from the results of tests using photo-elastic or analytical models. Since the ground at depth certainly exhibits non-elasticity, the stress concentration of an individual pillar may be lower than found--on the other hand, stress relief due to plastic yielding of one pillar may throw increased load onto others.

5. It is assumed that the weight per unit volume of the strata is constant at 160 lb/cu ft as for sedimentary rocks. However, the weight per unit volume of the strata is certainly not constant due to geological variations.

5.4.2 Assumptions for pillar strength determinations

1. It is assumed that roof strength has no effect on the pillar strength. However, although the pillar itself may be safe, the roof of the entry may collapse.

2. It is assumed that the floor has no effect. Yet, floor heaving may lead to splitting of the pillar at its corners (see: Rockaway, J. and Stephenson, R. "Evaluation of the Effects of Weak Underlays on the Support of Coal Pillars in Illinois Basin Mines." Proc. Conf. on Ground Control, Southern Illinois University, Carbondale, 1980, pp. 59-69.

5.4.3 Selection of Safety Factors

The survey of coal mining dimensions and design practice conducted at Penn State provided a means for deciding on safety factors for U.S. room-and-pillar coal mining. A total of 174 cases were collected, the data base of which is given in Table 5.

Table 5

Range of Room-and-Pillar Parameters used in U.S. Coal Mines
(Penn State Survey of 174 cases)

		<u>Typical</u>
Depth below surface	75-1500 ft	500 ft
Seam or pillar height	3 - 15 ft	6 ft
Entry width	12 - 27 ft	18 ft
Pillar width	15 - 75 ft	48 ft
Pillar length	20 - 90 ft	60 ft
Width-to-height ratio	2 - 16	8.0
Length-to-height ratio	1 - 3	1.25
Percentage extraction	25% - 85%	50%

Using the PSU computer storage and retrieval system, the above room and pillar dimensions were used together with the three pillar strength formulas. A histogram of safety factors for the PSU Strength Formula is given in Figure 4 for U.S. case histories (174) which included 3 failure cases.

It is concluded from Figure 4 that factors of safety ranging from 1.5 to 2.0 are appropriate for U.S. coal mining conditions when using the PSU Strength Formula. It must be emphasized, however, that this range should be used as a guide only and the local mining experience should be taken into consideration.

As far as the other two formulas are concerned, the factors of safety have been recommended by their developers. For the (Holland) Formula it has been recommended to use a safety factor of 2.0 as an average and 1.8 as a minimum and 2.2 as the maximum. However, Holland (1973) stated that in some situations a safety factor of 1.3 or 1.4 might be acceptable for retreat partial extraction mining. The Obert-Duvall formula had a factor of up to 4 suggested if no size effect is included.

During the present study, all the three formulas were evaluated by the 174 case histories for selection of the safety factors. As shown in Figures 5 and 6, it may be concluded that all three formulas would be acceptable for the range of safety factors of 1.5 to 2.0.

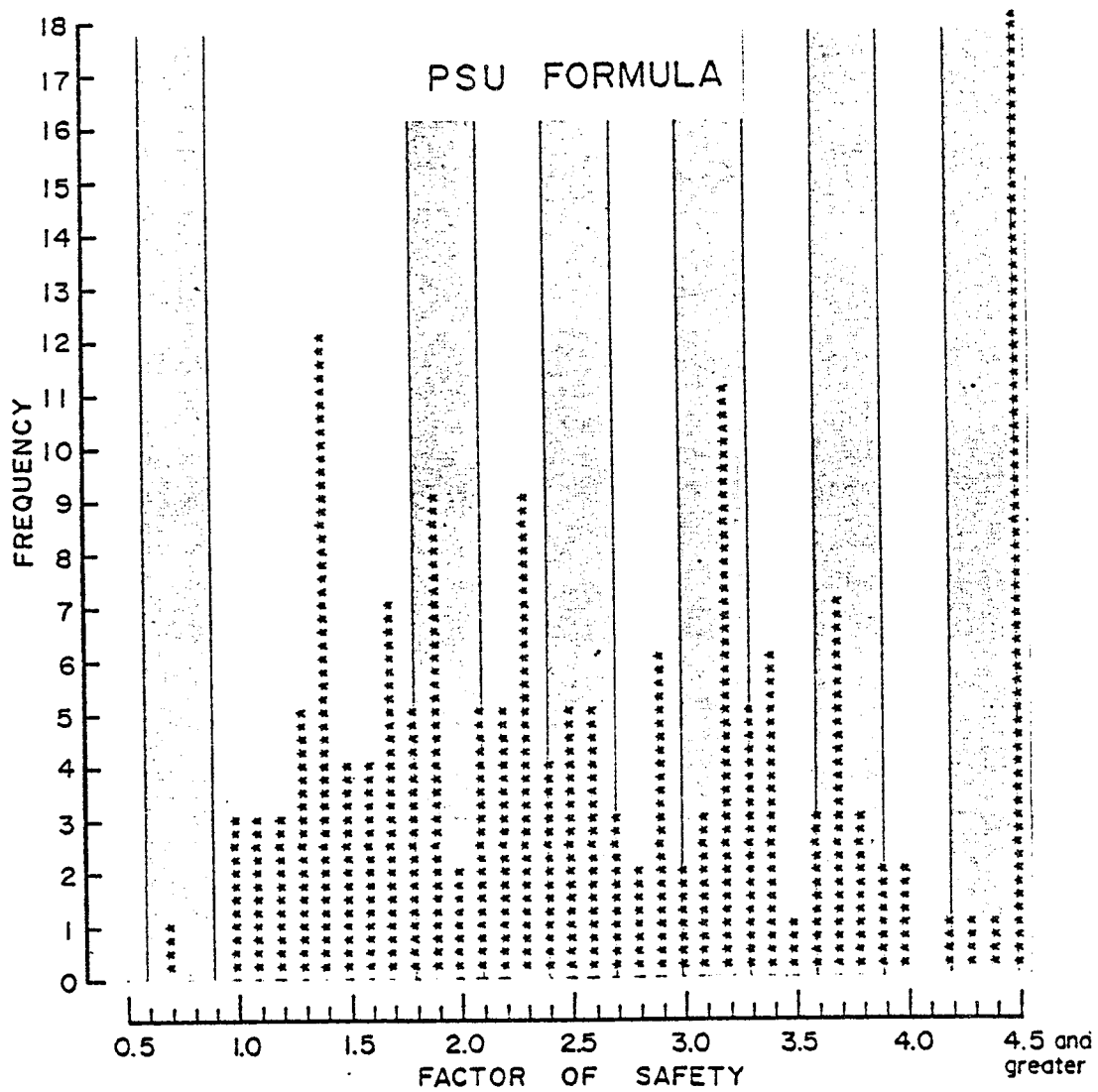


Figure 4. Histogram of safety factors - PSU strength formula.

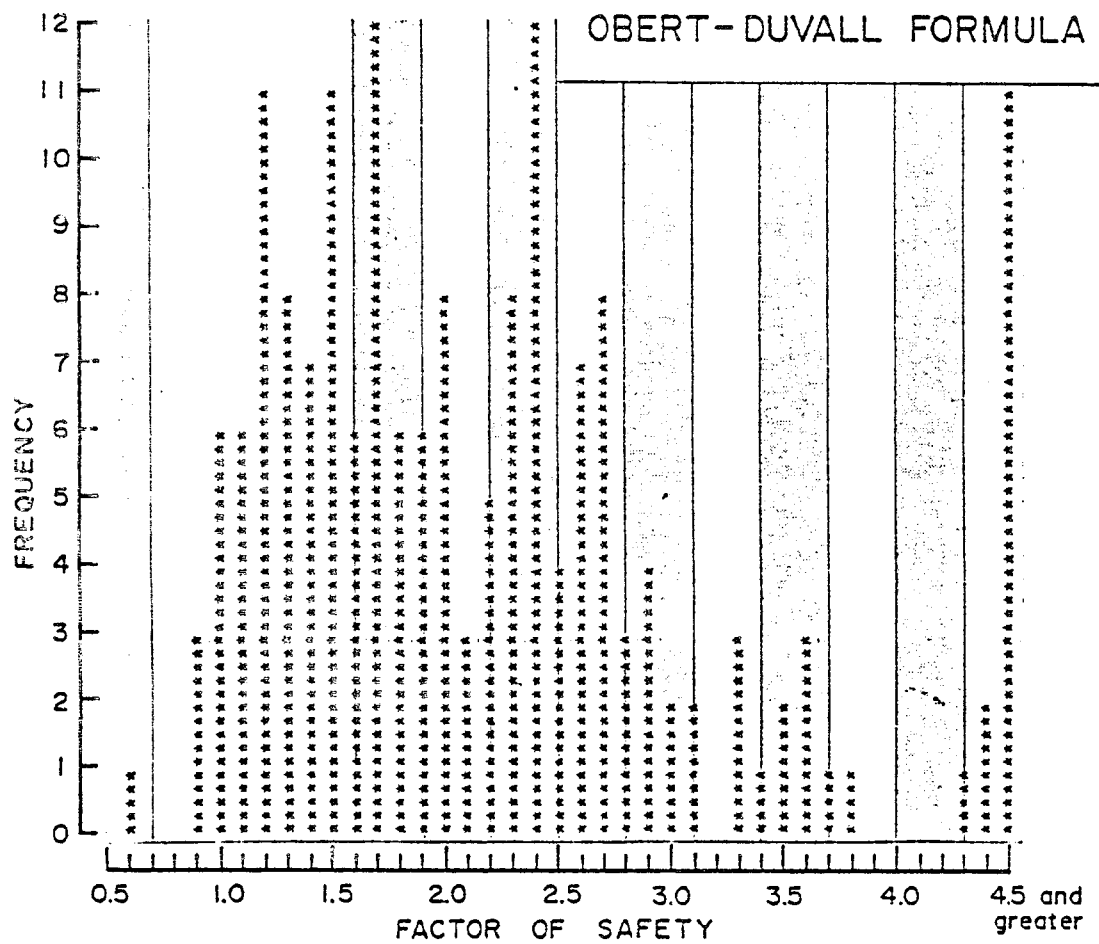


Figure 5. Histogram of safety factors: Obert-Duvall strength formula.

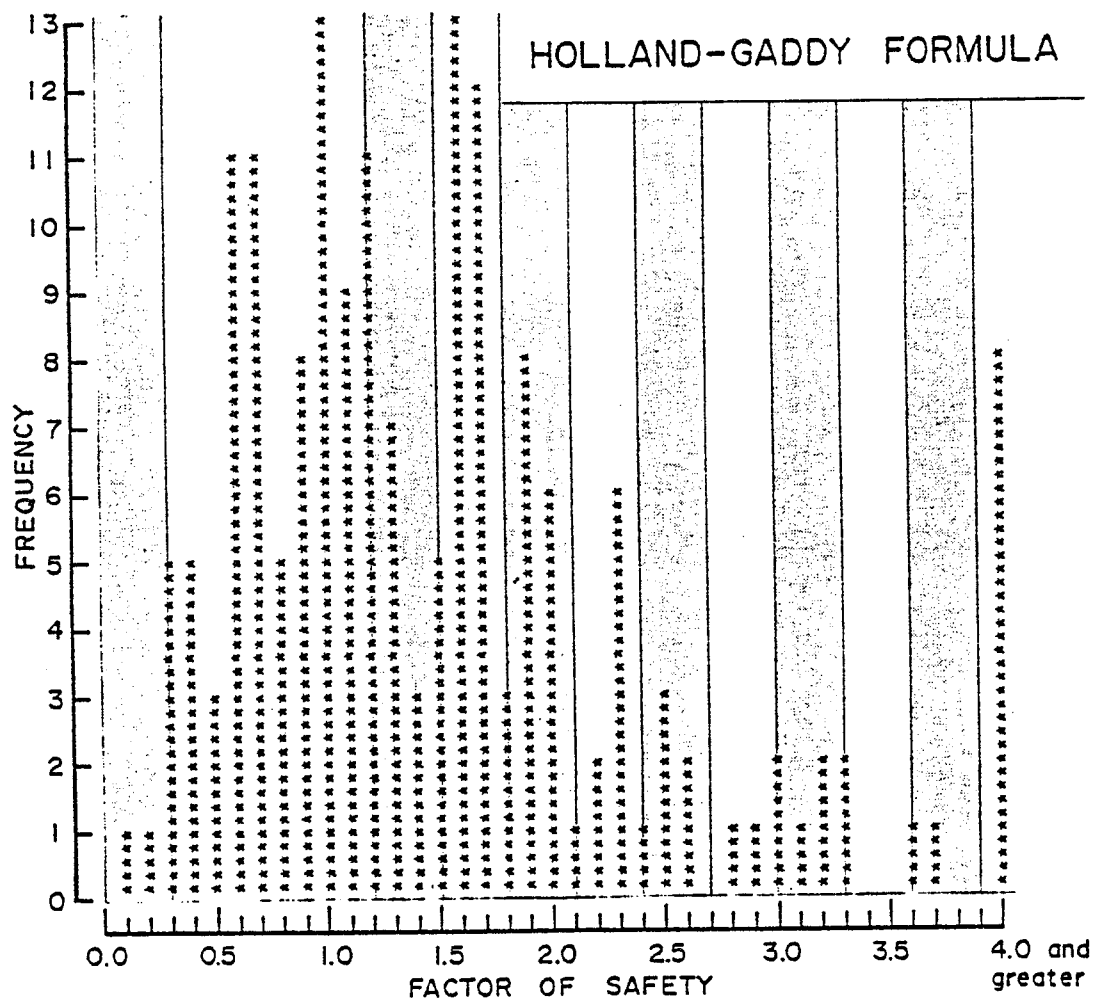


Figure 6. Histogram of safety factors: Holland-Gaddy strength formula.

5.5 Design Procedure

The following step by step procedure is recommended:

5.5.1 Planning New Mines

- Step 1 - From geological data, borehole logs and rock and coal specimen testing (2 inch core or cubes), tabulate the following: uniaxial compressive strength of roof rock and coal, σ_c ; spacing, condition and orientation of geological discontinuities, rock quality designation (RQD) if core is available, ground water conditions.
- Step 2 - Determine Rock Mass Rating (RMR) for roof rock in accordance with Table 1 and the example given in Table 3.
- Step 3 - Select the roof span B from Figure 1 and decide on roof support, e.g. Table 4.
- Step 4 - Based on the uniaxial compressive strength of coal, σ_c , determined either from 2-3 in core or 2-3 in cubes (both acceptable for practical purposes), determine the value of k for the mine locality:

$$k = \sigma_c \sqrt{D} \quad \text{Gaddy Formula}$$

where D is the specimen diameter or cube size.

- Step 5 - Determine the unit strength of the rock mass of cubical coal pillars at the critical size (no further size effect)

$$\sigma_1 = \frac{k}{\sqrt{36}} \quad \text{Hustrulid Formula 1}$$

if the seam height h is equal or more than 3 ft.

or

$$\sigma_1 = \frac{k}{\sqrt{h}} \quad \text{Hustrulid Formula 2}$$

if the seam height is less than 3 ft.

- Step 6 - Write down the PSU pillar strength formula which will be used to estimate the pillar width w, for a known seam height h:

$$\sigma_p = \sigma_1 (0.64 + 0.36 w/h) \quad \text{Formula (Penn State)}$$

Note that the unknowns in this formula are the pillar strength σ_p and the pillar width w . To simplify the design procedure, assume that the pillars are of a square cross-section which will result in an underestimated pillar strength as shown in the previous section. This can be compensated for by using a smaller factor of safety.

- Step 7 - Write down the equation for the pillar load (the stress on the pillar) based on the tributary area theory:

$$S_p = 1.1 H \left[\frac{w + B}{w} \right]^2 \quad \dots \dots \dots \text{Tributary Area Formula}$$

where S_p is the pillar load in psi, H is the depth below surface, B is the entry span from Step 3 and w is the pillar width. Note that the unknowns in the above formula are the pillar load S_p and the pillar width w .

- Step 8 - From economic considerations, decide what percentage extraction (100e) is needed for a profitable mining venture. From the equation:

$$e = 1 - \left[\frac{w}{w + B} \right]^2 \quad \dots \dots \dots \text{Extraction Ratio}$$

or with the aid of Chart I prepared for convenience, select a pillar width w which would give the required coal extraction.

- Step 9 - Test the selected pillar width w to determine whether it is acceptable from the mine stability point of view or whether the pillar width needs to be changed. This requires calculation of the factor of safety:

$$F = \sigma_p / S_p \quad \dots \dots \dots \text{Factor of Safety}$$

where σ_p is the pillar strength from Step 6 while S_p is the pillar load from Step 7.

- Step 10 - The safety factors should be between 1.5 and 2.0. Cross-check the results using the Holland formula and the Obert-Duvall formula. Exercise engineering judgment by considering a range of parameters, to assess the various options for mine planning. This can be obtained from Figures 7 through 11.

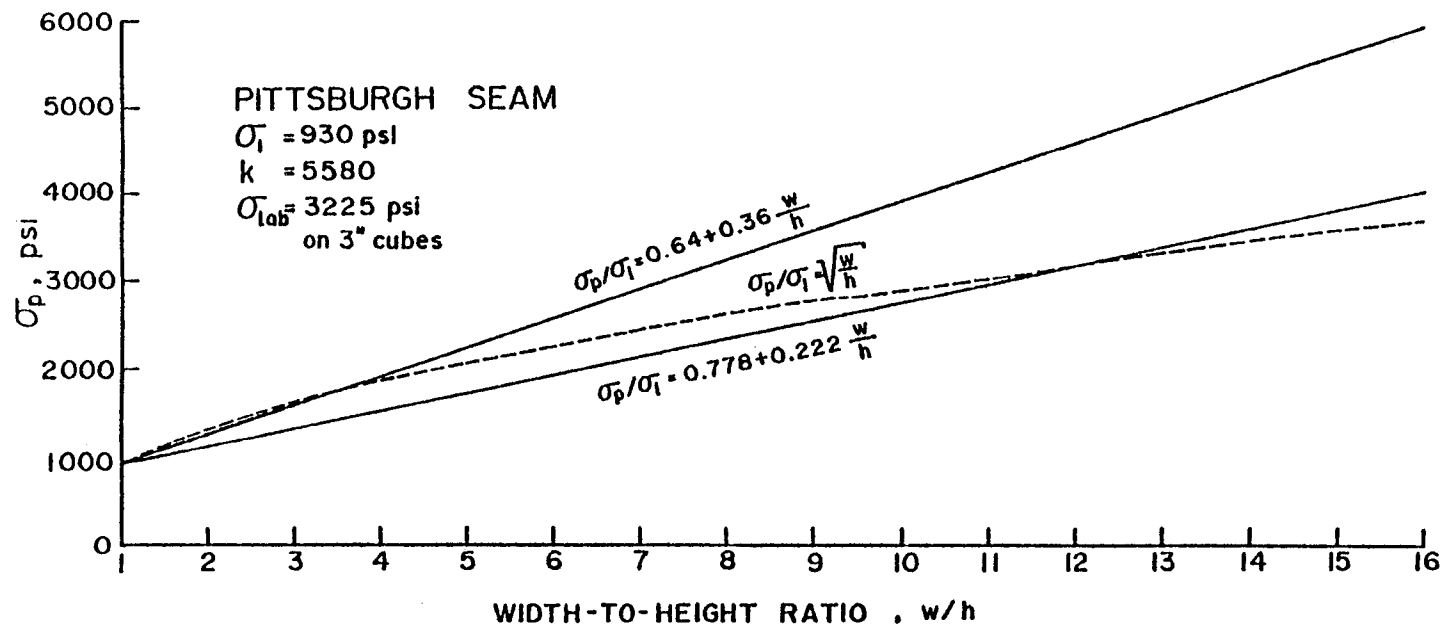


Figure 7. Pillar strength in Pittsburgh seam vs w/h ratio as predicted by three formulas.

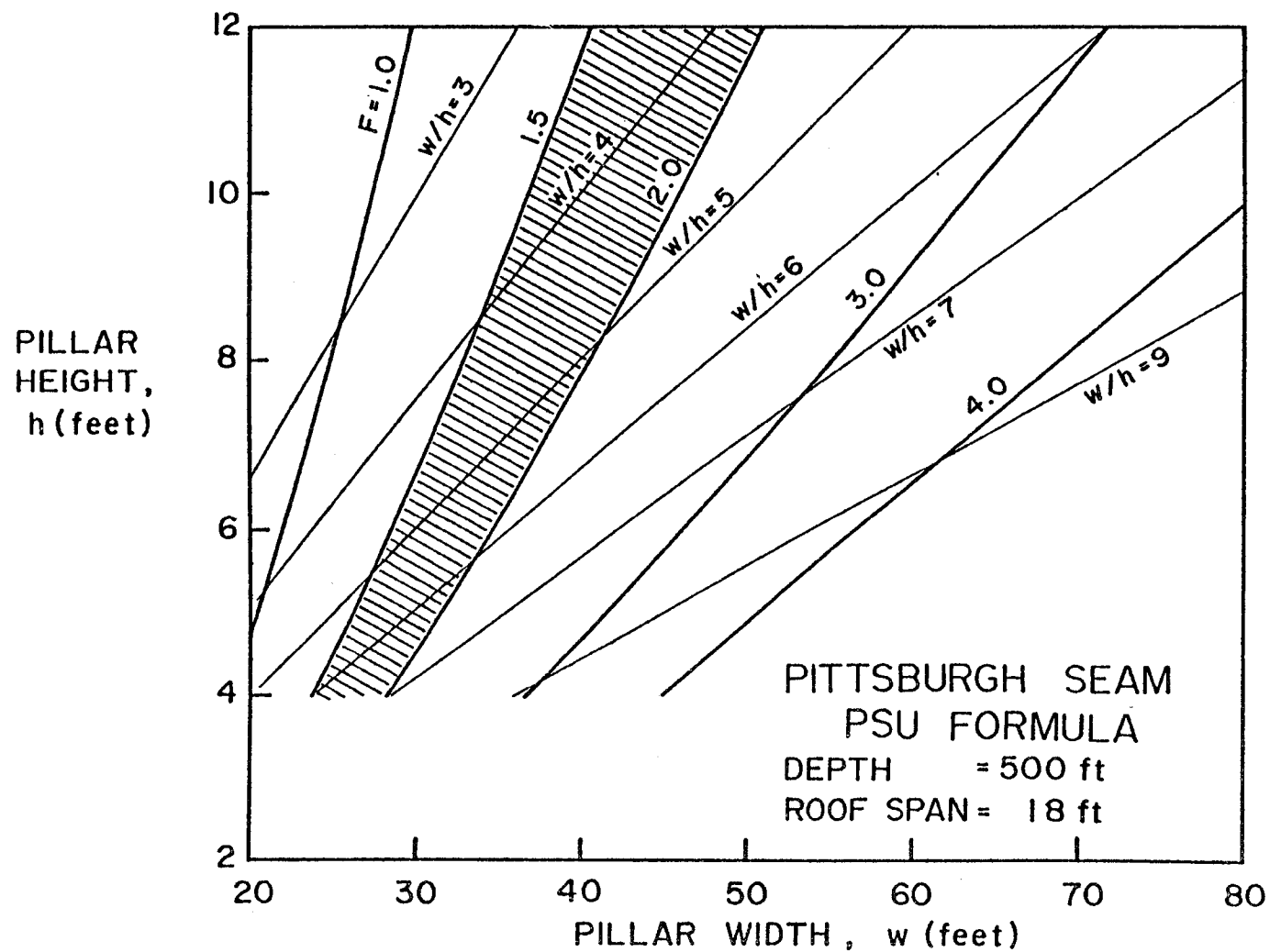


Figure 8. Pittsburgh seam: PSU formula, depth 500 ft, span 18 ft.

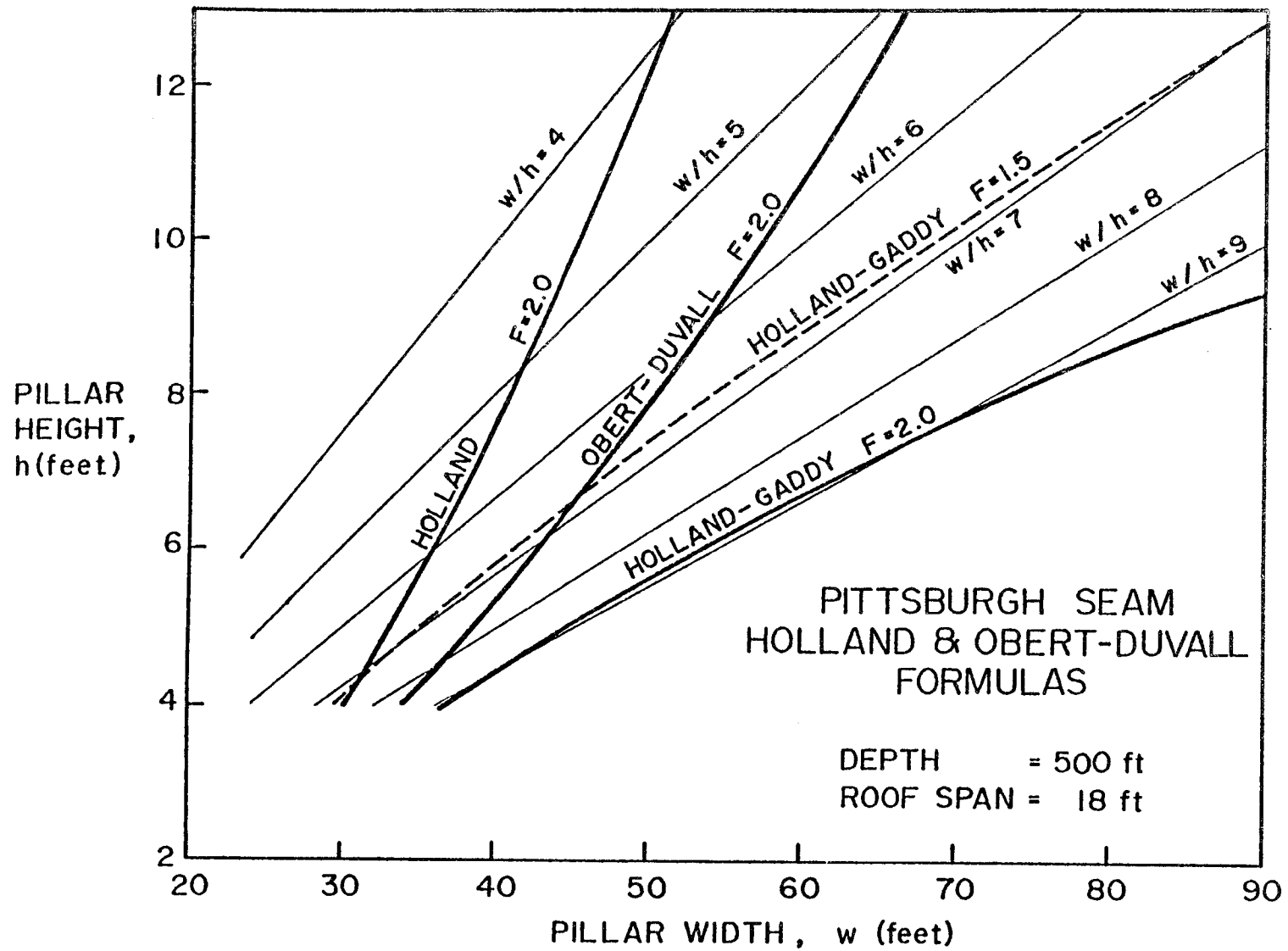


Figure 9. Pittsburgh seam: Holland and Obert-Duvall formulas, depth 500 ft, span 18 ft.

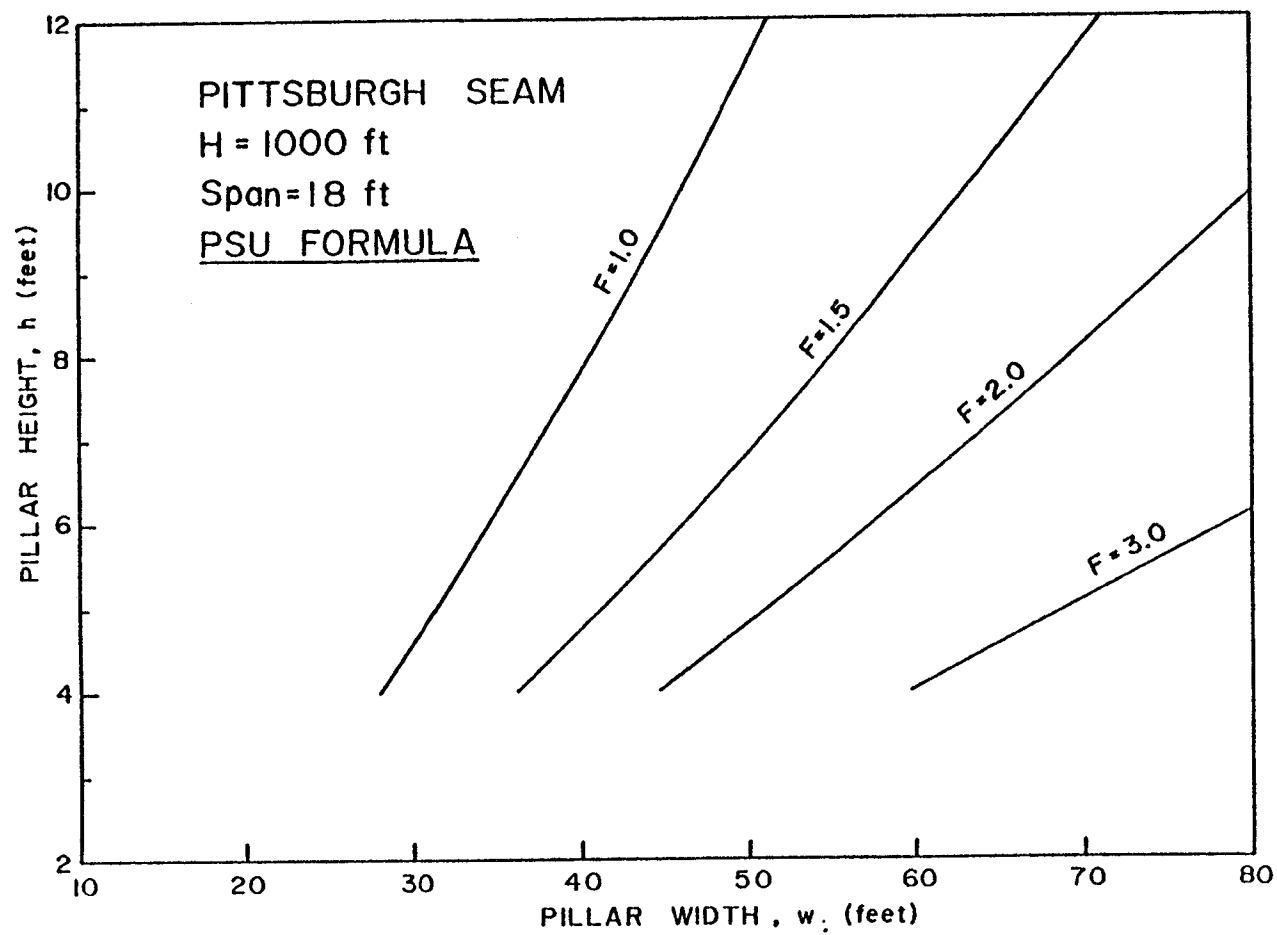


Figure 10. Pittsburgh seam: PSU formula, depth 1000 ft, span 18 ft.

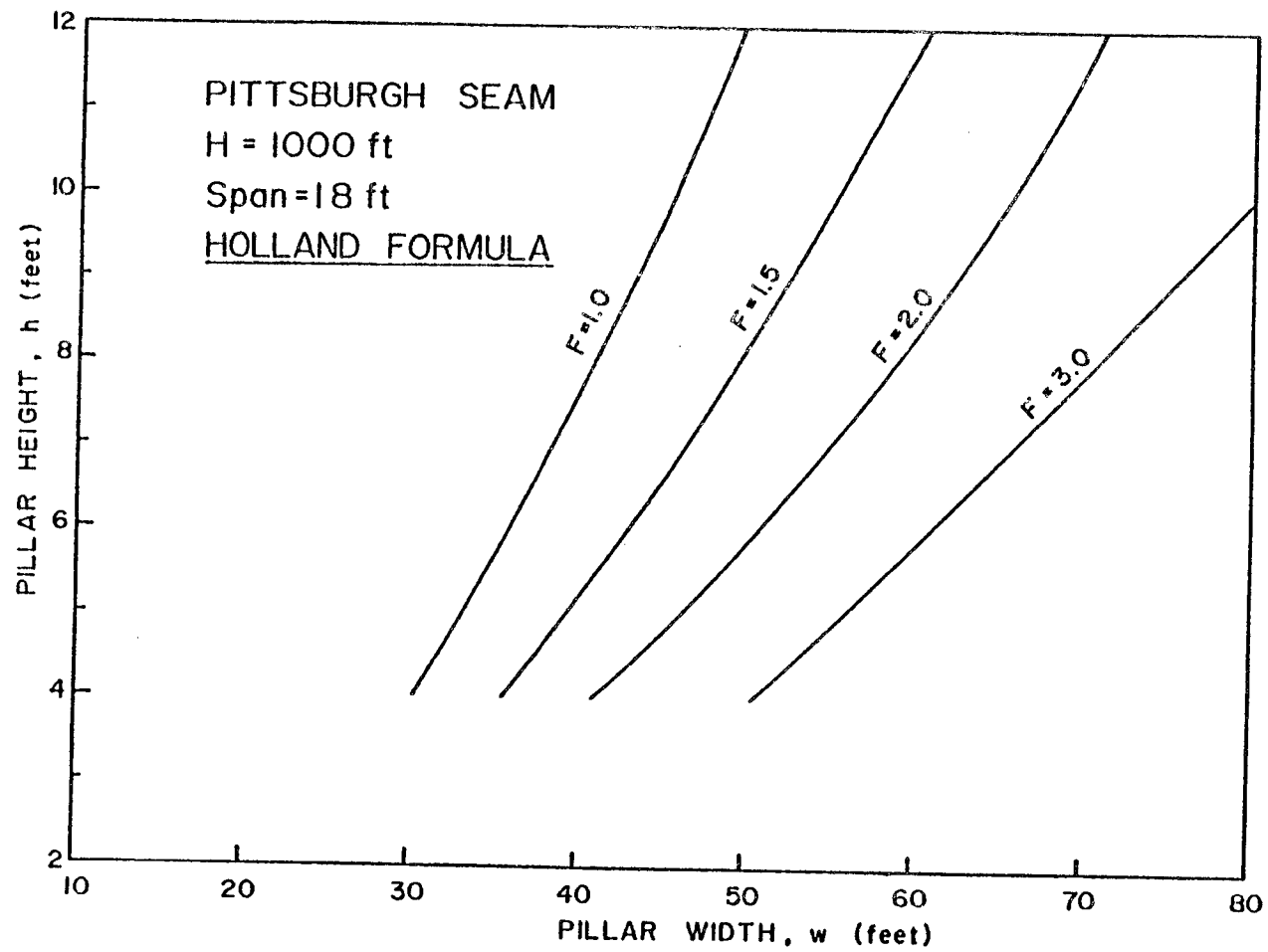


Figure 11. Pittsburgh seam: Holland formula, depth 1000 ft, span 18 ft.

5.5.2 Optimizing mining in progress

In order to assess the stability of an existing mining operation, all the mining dimensions are known and the factor of safety can be determined for use in future design.

To facilitate this process, design Chart II and Figure 12 are used as explained below.

5.6 Practical Examples of Mine Design

Research in the United States has provided a considerable amount of information on the strength of coal from the various coal seams. The typical k values for some of the seams are as follows:

<u>Seam</u>	<u>k</u>
Pittsburgh	5551-5862
Pocahontas No. 4	4313-4825
Clintwood	4289-5205
Elkhorn No. 4	6000-6250
Harlan	8859-9458
Marker	10120-10600

Furthermore, the typical data for the Eastern and Western coal fields are as follows:

	<u>Overall East</u>	<u>Pittsburgh Seam</u>	<u>Overall West</u>
k	5160	5580	8000
σ_c (lab)	2775 psi	3160 psi	4000 psi
σ_1 (36" cube)	860 psi	930 psi	1330 psi

Studies performed at Penn State featuring engineering geological mapping of the Pittsburgh coal seam, laboratory testing of coal and the use of the Protodykanov formula (see Chapter 3) have estimated that the unit strength of Pittsburgh coal σ_1 (36" cube) is about 930 psi.

Accepting a factor of safety of 1.5, the pillar strength formulas for the Pittsburgh seam are as follows:

$$\begin{aligned} \sigma_p &= 620 (0.64 + 0.36 w/h) \dots \dots \dots \text{PSU Formula} \\ \sigma_p &= 620 (0.778 + 0.222 w/h) \dots \dots \text{Obert-Duvall Formula} \\ \sigma_p &= 620 \sqrt{w/h} \dots \dots \dots \text{Holland Formula} \end{aligned}$$

Taking, as an example, the PSU Formula and equating it with the pillar load (note that a factor of safety of 1.5 has already been incorporated) one obtains:

$$\frac{1.1 H}{1 - e} = 620 (0.64 + 0.36 \frac{w}{h})$$

or

$$H = 564 (0.64 + 0.36 w/h) (1 - e)$$

The above equation is plotted as Chart II featuring the depth below surface H (feet) versus width-to-height ratio (w/h) for various extraction ratios e (percentage extraction is 100 e). This chart is valid for a factor of safety of 1.5 and the Pittsburgh seam. However, similar charts can easily be constructed for other seams and different safety factors.

Example 1: Check existing mining operation (Pittsburgh seam)

Depth below surface: 500 ft
 Entry width: 16 ft
 Pillar width: 60 ft
 Pillar length: 80 ft
 Seam height: 7 ft
 What is the factor of safety: ?

Coal extraction

$$e = 1 - \frac{w}{w + B} \frac{L}{L + B} = 1 - \frac{60}{60 + 16} \frac{80}{80 + 16} = 0.34$$

Actual percentage extraction is 34%.

$$F = \frac{\sigma_p}{S_p} = \frac{930 (0.64 + 0.36 w/h)}{\frac{1.1 H}{1 - e}}$$

$$= \frac{930}{550} (0.64 + 0.36 \times 8.57) \times 0.66 = 4.15$$

Example 2: Improve on extraction in the above example within safety limits.

From Chart II, for a depth of 500 ft and extraction of 60%, the recommended width to height ratio is 4.5, thus for a seam height of 7 ft giving the pillar width of 31 ft. Entry width will be 18 ft for 60% extraction.

Figure 12, constructed for factor of safety of 1.5, is a useful chart for estimating the pillar widths in the Pittsburgh seam.

Example 3

A coal mine in Pittsburgh seam is mining at 500 ft with stable pillars and roofs and mining is planned at 1000 ft. What pillar sizes should be selected at that depth?

$$\begin{aligned}\text{Given: } H &= 500 \text{ ft} \\ h &= 10 \text{ ft} \\ w &= 40 \text{ ft} \\ B &= 18 \text{ ft}\end{aligned}$$

(i) Determine the factor of safety found satisfactory at the mine:

$$\text{Pillar load } S_p = 1.1 H \left[\frac{w + B}{w} \right]^2 = 550 \left[\frac{40 + 18}{40} \right]^2 = 1156 \text{ psi}$$

$$\text{Pillar strength for Pittsburgh seam: } \sigma_p = 930 (0.64 + 0.36 w/h) \\ = 1934 \text{ psi}$$

$$\therefore F = \sigma_p / S_p = 1.67$$

(ii) Check on the current percentage extraction:

$$e = 1 - \left[\frac{w}{w + B} \right]^2 = 1 - \left[\frac{40}{40 + 18} \right]^2 = 0.52$$

$\therefore$ Percentage extraction is 52%

(iii) It can be deduced from Figure 10 that at the planned depth of 1000 ft and the same roof span $B = 18$ ft, the new pillar width could be $w = 65$ ft. Check the factor of safety at that depth:

$$S_p = 1100 \left[\frac{65 + 18}{65} \right]^2 = 1793 \text{ psi}$$

$$\sigma = 930 (0.64 + 0.36 \times \frac{65}{10}) = 2771 \text{ psi}$$

$$\therefore F = \frac{2771}{1793} = 1.6$$

The factor of safety is acceptable and the pillar width of 65 ft may be used at the depth of 1000 ft.

(iv) Check the percentage extraction for planned mining:

$$e = 1 - \left[\frac{65}{65 + 18} \right]^2 = 0.4$$

The percentage extraction will be 40% which is economically acceptable at that depth

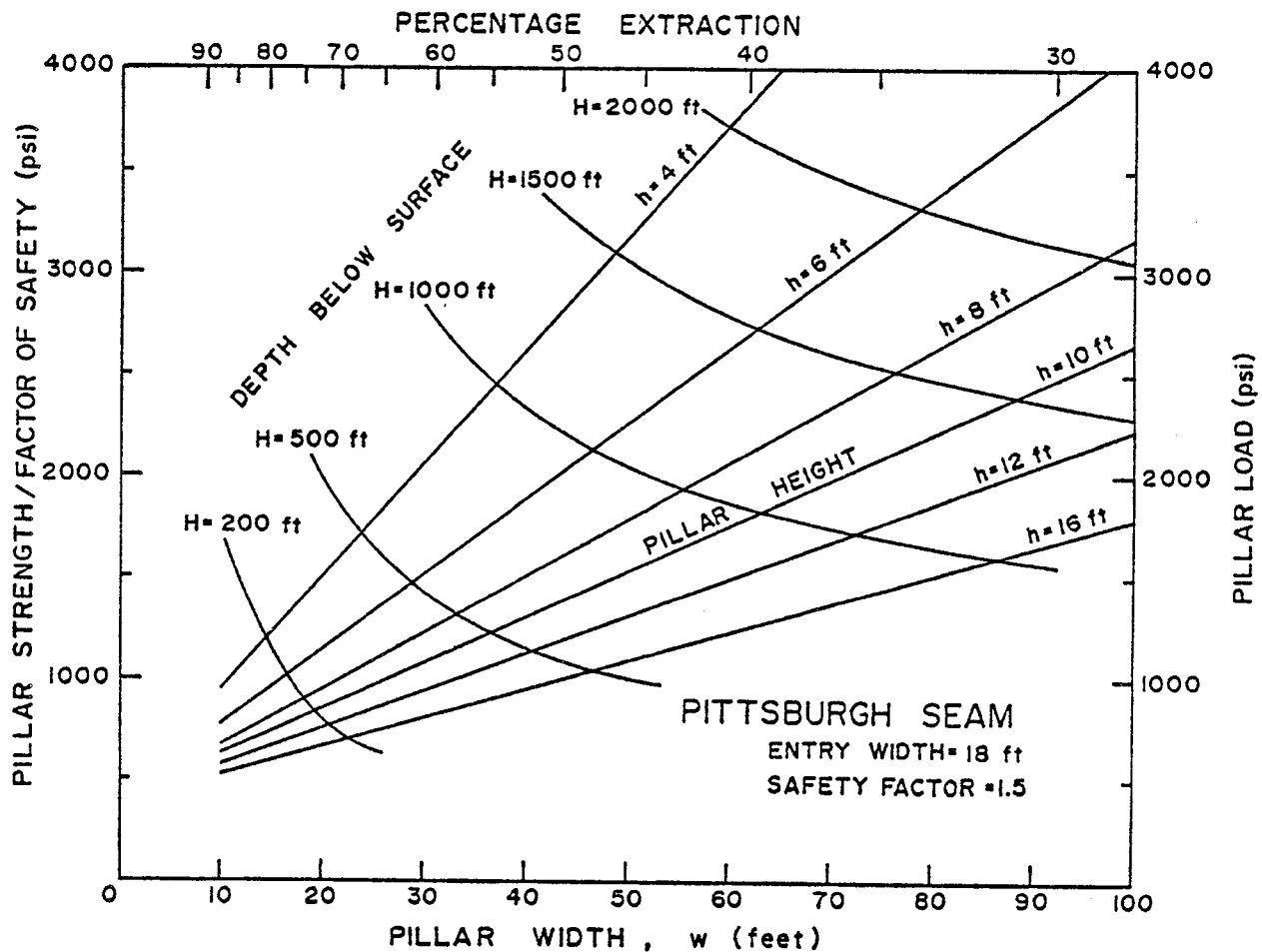
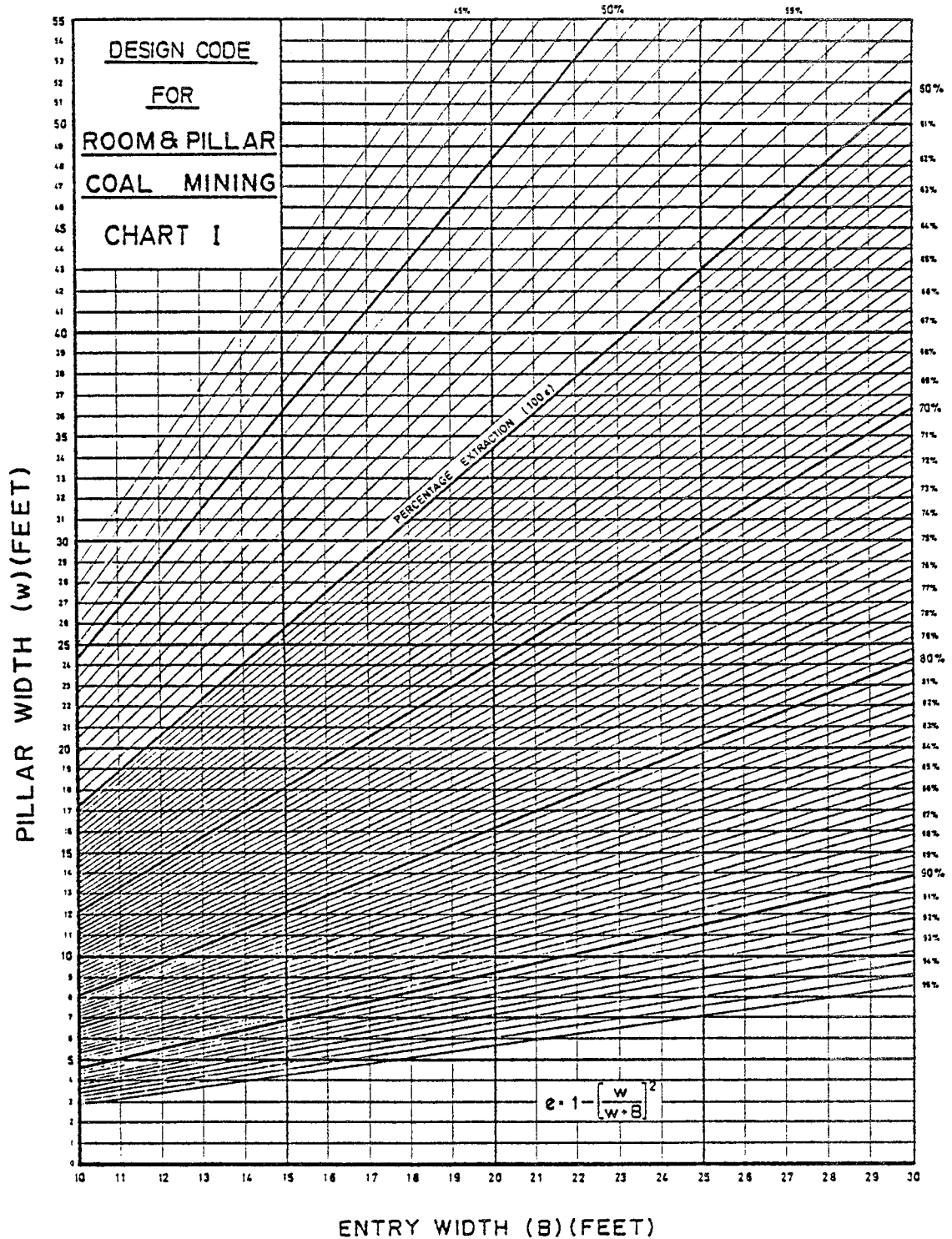
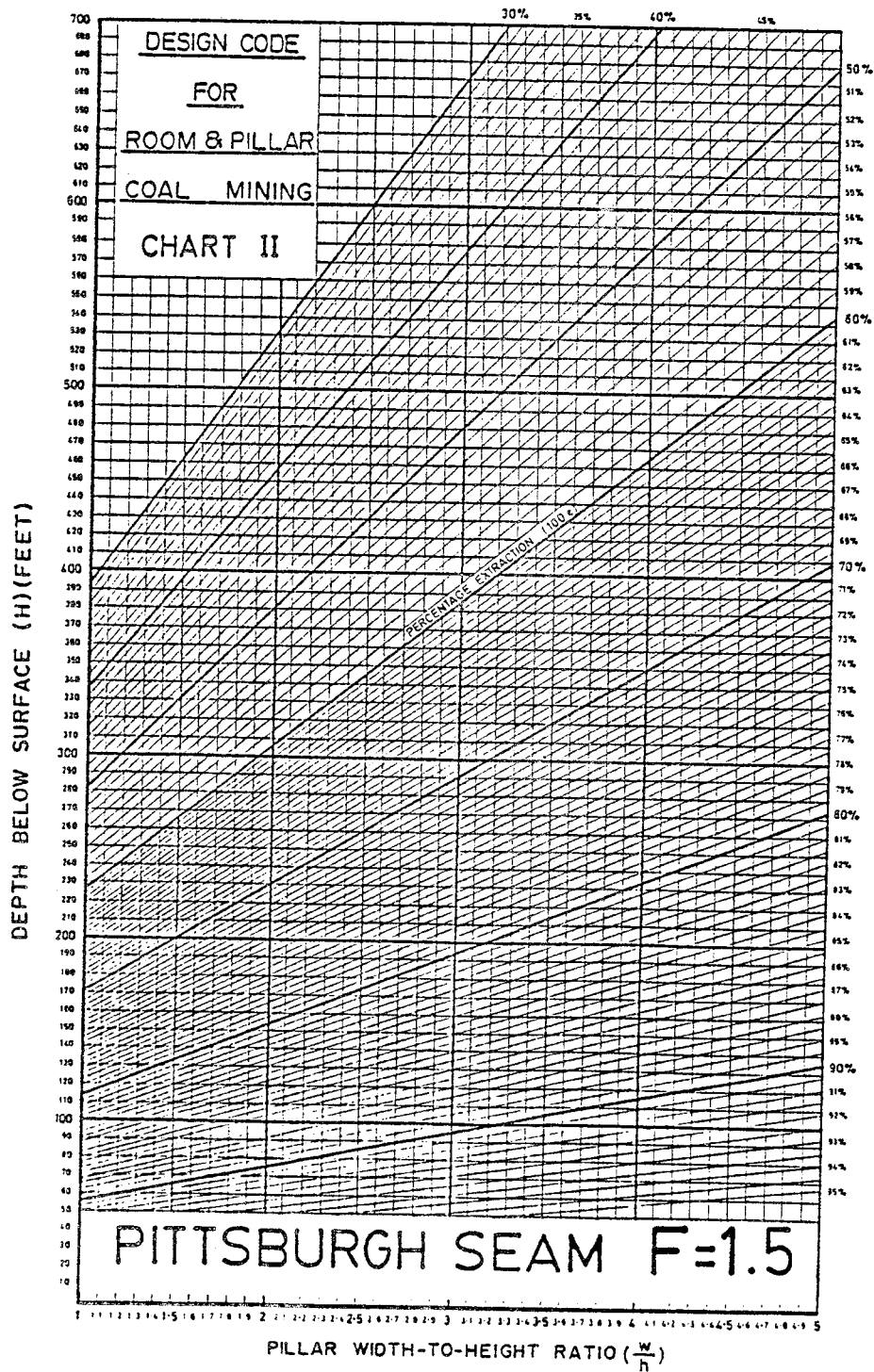


Figure 12. Pittsburgh seam: Pillar strength vs pillar width for different pillar heights and different depth. Span 18 ft PSU strength formula $F = 1.5$.





PITTSBURGH SEAM

* SAFETY	* SEAM	* ENTRY	DEPTH BELOW SURFACE					*	
* FACTOR	* HEIGHT	* WIDTH	IN FEET					*	
*	*IN FEET	*IN FEET*	200	400	600	800	1000	*	

*	*	* 16 *	12.8	19.3	24.8	30.0	34.8	*	
*	*	* (%) *	80.2	70.1	63.0	57.4	53.0	*	
*	*	*-----*	*****						*
*	* 4	* 18 *	13.8	20.6	26.3	31.6	36.6	*	
*	*	* (%) *	81.1	71.5	64.7	59.4	55.0	*	
*	*	*-----*	*****						*
*	*	* 20 *	14.7	21.8	27.8	33.3	38.4	*	
*	*	* (%) *	82.0	72.8	66.1	60.9	56.7	*	

*	*	* 16 *	14.8	23.1	30.5	37.5	44.2	*	
*	*	* (%) *	76.9	65.0	56.9	50.8	46.0	*	
*	*	*-----*	*****						*
*	* 1.5	* 6	* 18 *	16.0	24.6	32.3	39.4	46.3	*
*	*	* (%) *	77.8	66.6	58.7	52.8	48.1	*	
*	*	*-----*	*****						*
*	*	* 20 *	17.1	26.1	34.0	41.3	48.3	*	
*	*	* (%) *	78.7	67.9	60.3	54.6	49.9	*	

*	*	* 16 *	16.4	26.4	35.5	44.2	52.7	*	
*	*	* (%) *	74.3	61.2	52.4	46.0	41.1	*	
*	*	*-----*	*****						*
*	*	* 18 *	17.7	28.1	37.5	46.5	55.1	*	
*	*	* (%) *	75.4	62.8	54.3	48.0	43.1	*	
*	*	*-----*	*****						*
*	*	* 20 *	19.0	29.8	39.4	48.6	57.4	*	
*	*	* (%) *	76.2	64.1	56.0	49.8	45.0	*	

*	*	* 16 *	15.1	23.0	30.0	36.4	42.6	*	
*	*	* (%) *	76.4	65.2	57.4	51.7	47.1	*	
*	*	*-----*	*****						*
*	* 4	* 18 *	16.2	24.5	31.6	38.3	44.6	*	
*	*	* (%) *	77.5	66.7	59.4	53.7	49.2	*	
*	*	*-----*	*****						*
*	*	* 20 *	17.3	25.9	33.3	40.1	46.5	*	
*	*	* (%) *	78.4	68.1	60.9	55.4	51.1	*	

*	*	* 16 *	17.7	28.1	37.5	46.3	55.0	*	
*	*	* (%) *	72.4	59.3	50.8	44.7	39.9	*	
*	*	*-----*	*****						*
*	* 2.0	* 6	* 18 *	19.1	29.8	39.4	48.5	57.3	*
*	*	* (%) *	73.4	61.1	52.8	46.8	42.0	*	
*	*	*-----*	*****						*
*	*	* 20 *	20.3	31.4	41.3	50.6	59.6	*	
*	*	* (%) *	74.6	62.6	54.6	48.6	43.9	*	

*	*	* 16 *	19.9	32.5	44.2	55.5	66.6	*	
*	*	* (%) *	69.2	55.0	46.0	39.7	34.9	*	
*	*	*-----*	*****						*
*	* 8	* 18 *	21.4	34.4	46.5	57.9	69.2	*	
*	*	* (%) *	70.4	56.9	48.0	41.8	37.0	*	
*	*	*-----*	*****						*
*	*	* 20 *	22.8	36.3	48.6	60.3	71.7	*	
*	*	* (%) *	71.6	58.4	49.8	43.6	38.8	*	

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CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

This three-year research project has maintained an excellent progress throughout its duration and the objectives set have been achieved. In view of the novel and high-risk techniques introduced during the study, ample possibilities still exist to pursue further the ideas introduced in the course of this research grant.

6.1 Conclusions

1. An improved design method for room-and-pillar coal mining in the United States has been developed.
2. Estimating safe roof spans in coal mines by means of the Geomechanics Classification (Rock Mass Rating System) represents a practical approach to roof control and sufficient geological parameters are usually available for this purpose either during the mine planning stage or the mine operation stage.
3. In the current stage of development, the Geomechanics Classification gives acceptable estimates of stable roof spans (some 60 roof fall data were collected) but the technique could be improved by including the effects of the induced stresses due to mining and providing systematic guidelines for roof support and reinforcement.
4. The petite sismique technique was found to be a promising method for estimating strata deformability. This novel method will, however, require a considerable research effort before it can be put into full practical use. The major shortcoming is the lack of a reliable shear wave source.
5. Sufficient research results and practical experience are available in the United States for a reliable design of coal mine pillars. An improved design approach has been proposed which would enable greater extraction of coal compatible with roof and pillar stability.
6. The concept of the critical pillar size (about 3 ft for coal) which represents the size effect in pillar design must be considered in conjunction with the shape effect when selecting pillar dimensions.
7. The strength of rectangular pillars is probably about 40% higher than the strength of square pillars having the same width and height.
8. The tributary area method for determining the pillar loads overestimates the stresses acting on pillars by some 30-40%.

9. The Penn State survey of mining dimensions and design practice in the United States (174 cases) has provided a good data base for selecting the factors of safety for the recommended pillar strength formulas.
10. A design code for improved room-and-pillar coal mining in the United States has been proposed and its practical use demonstrated by means of a number of selected examples.

6.2 Recommendations for Future Research

1. The proposed design code should be tried systematically in a number of case histories involving stable and failed coal pillars and mine roofs.
2. More case histories are also needed for the Geomechanics Classification in terms of the maximum roof spans, stand-up times, support measures and roof quality (RMR values).
3. The petite sismique technique should be systematically studied with particular reference to the shear wave source.
4. The effects of pillar punching and floor heaving in mine entries should be researched and incorporated into design recommendations.
5. Numerical modeling of progressive failure in coal mine pillars, accompanied by field measurements, should be undertaken for further improvement of design procedures.
6. Adequate geotechnical data base requirements for a typical mine property need to be established.

APPENDIX

THE PENNSYLVANIA STATE UNIVERSITY

110 MINERAL SCIENCES BUILDING
UNIVERSITY PARK, PENNSYLVANIA 16802

College of Earth and Mineral Sciences

Continuing Education

814-865-6335

RE: Survey of Coal Pillar Sizing

Dear

The Geomechanics Section of the Mineral Engineering Department here at The Pennsylvania State University has recently been awarded a Department of Energy contract to investigate improved design procedures for room and pillar coal mining. Using both prior PSU work in the area of roof control and many new ideas not utilized to date in this country, it is our hope to develop design procedures which benefit the coal industry as a whole by minimizing pillar sizes while maximizing entry (or roof) stability during the life of the opening.

Current pillar design formulas in common use in the United States are based on either test studies (Greenwald, Howarth, and Hartmann at the USBM in the 1930's) or laboratory tests (Holland and Gaddy at VPI in the 1950's). The limitations of these approaches is reflected in initiation of additional research that has been conducted on pillar design recently at Colorado School of Mines, Ohio State and the University of Utah.

One of the several design methods being investigated at Penn State involves the development of a statistical formula which rationalizes many years of mining experience. This method was originally developed in South Africa, and I have enclosed a copy of the paper describing the South African work for your information. We anticipate a similar development approach (not subject to some of the limitations and restrictions of the South African work) to a statistical method applicable to U.S. coal mines, and hence have proposed a questionnaire containing the required design information. From this questionnaire we hope to develop a sizable data base which will then insure a statistically meaningful interpretation.

Although I realize the demands on your time are great, your help in this matter would be greatly appreciated as well as yielding a contribution to the development of improved mine design parameters. I have enclosed a copy of our questionnaire and would like to request that you distribute it to each mining operation under your supervision. The type of design

approach this questionnaire represents is only meaningful if the responses are submitted by the mining industry, hence your company's consideration of this request is extremely important. The identification of any individual response will be kept in strict confidence while information on the results of this segment of our research will be sent to all responding companies.

I anticipate that each mine will require at the minimum two copies of this form, one for panel entries and the other for main entry development. If you require additional questionnaire copies for other areas of a mine I will gladly provide them or any type of copy would be acceptable. In addition, our staff on this segment of our project, Dr. Z. T. Bieniawski, Professor of Mineral Engineering, G. W. Luxbacher, Research Assistant in Mining Engineering and I would be happy to discuss any phase of this project or survey questionnaire with you in greater detail if you desire.

In addition to enclosing a copy of our survey questionnaire, I have also enclosed comprehensive instructions explaining each part of the questionnaire. A filled out example of this form has been attached for any further clarification you may require.

I look forward to receiving your response to our survey. Thank you for your cooperation.

Yours truly,

Robert Stefanko
Associate Dean for
Continuing Education

RS/as

Enclosure

attach.

CASE HISTORY - PILLAR SIZING

COMPLETION DATE¹

IDENTIFICATION INFORMATION²

Mining Company³

Mine⁴

Individual Filling Out Form⁵

Individual to be Contacted for Further Information⁶

Designation for Mining Area⁷

Mains

Submain

Panels

_____ (Other)

CASE STUDY STATISTICS

Seam Height⁸

Overburden Height⁹

Pillar Width¹⁰ , Centers Width

Pillar Length¹¹ , Centers Length

Accuracy of Pillar Measurements¹²

Entry Width¹³ , Crosscut Width

Approximate Diminsion of Mining area¹⁴ (attach map to clarify if necessary)

CASE STUDY GEOLOGY

Immediate Roof¹⁵

Immediate Bottom¹⁶

General Overburden¹⁷

Coal Seam¹⁸

MINING INFORMATION

Conventional or Continuous Mining¹⁹

Mining Type²⁰

Full Retreat

Partial Retreat

Partial Mining

Type Information²¹

Full Retreat²²

-stand up time prior to retreat mining

avg. ,maximum

-date of mining

Partial Retreat²³

-blocks split or shaved?

-initial pillar dimensions

-final pillar dimensions

-date of mining

STABILITY INFORMATION

Pillar Problems Observed²⁴

pillar

roof

bottom

Stand up Time Prior to Failure²⁵

avg. ,maximum

Surface Indication of Failure²⁶

Further Description of Failures²⁷

PILLAR DESIGN

Design Criteria²⁸

Design Life²⁹

Estimated Coal Strength³⁰

ROOF CONTROL

Roof Control Procedures³¹

Span Determination Criteria³²

ADDITIONAL INFORMATION

Geotechnical Studies³³

Publications or Reports³⁴

Estimated Design Safety Factor³⁵

Comments³⁶

INSTRUCTIONS

GENERAL:

The purpose of this form is to facilitate the compilation of information on industry practice of pillar sizing. Each area within a particular mine where a pillar size change has been made may be considered a separate case history, hence several forms may be required for each mining operation. Case histories where pillar sizes have been tried and abandoned due to unsatisfactory results should also be included, since these examples indicate inadequate pillar sizing. If any information required on the form is not available, indicate as such in the response space provided. This form may be supplemented by other available documentation, such as maps or reports, to assure proper clarity.

GENERAL INFORMATION

01. Completion Date. Enter the date this form was prepared.
02. Identification Information. All information within this section is confidential and will not be included in any documentation on this project. It is included only to permit adequate follow-up on case study data provided.
03. Mining Company. Enter company name and address.
04. Mine. Enter mine name and address
05. Individual Filling Out Form
06. Individual to be Contacted } Enter name, title and phone number
07. Designation for Mining Area. Enter the mine designation for the case study area such as 1 Left panel, 5 Right, North Mains, etc. If the described information pertains to the overall mine, please indicate. Also circle the type of development, mains, submains or panels.

CASE STUDY STATISTICS

- 08. Seam Height. Enter seam height being mined.
- 09. Overburden Height. Enter overburden height above the mining area, as accurately as possible.
- 10. Pillar Width. } Enter both actual pillar dimensions and pillar centers
- 11. Pillar Length } dimensions (i.e., 80' x 80' actual, 94' x 94' centers
 with 14 ft. wide entries)
- 12. Accuracy of Pillar Measurements - Note the source of the above pillar.
dimensions, i.e., measured, from mine projections, assumed.
- 13. Entry and Crosscut Width. Enter mining width for entries and crosscuts
- 14. Approximate Dimensions of Mining Area. Enter dimensions of area actually
pillared, for instance 2000 ft by 650 ft panels separated by a 200 ft
barrier pillar with the pillar mined on retreat from the adjacent panel.

CASE STUDY GEOLOGY

- 15. Immediate Roof. Enter a general description of the immediate roof
above the mined seam. This information may be of the type used on the
roof control plan submitted to MSHA.
- 16. Immediate Bottom. Enter a general description of the immediate bottom
rock.
- 17. General Overburden. If available, describe the overburden rock types
as found in any exploratory borings in the vicinity. A core log may be
included to give the information required for this section.
- 18. Coal Seam. Give the seam name, both local and, if different, regional.

MINING INFORMATION

- 19. Conventional or Continuous Mining. Specify mining method.
- 20. Mining Type: Is the mining described in this case study for full re-
retreat, partial retreat (original pillars reduced in size during some

form of retreat mining), or partial mining (pillars as originally mined left intact during the life of the mine, possibly retreated in the final years of the mine life or left intact upon closing).

21. Type Information: Information to further describe either full retreat or partial retreat mining.
22. Full Retreat. Enter the average and maximum stand up time of the mining area prior to retreat mining. Also give the approximate date of mining.
23. Partial Retreat. Describe the partial retreat mining method used and the pillar (not centers) dimensions prior to and after retreat mining. Also give the approximate date of mining.

STABILITY INFORMATION

24. Pillar Problems Observed. Indicate any problems which have been observed, such as pillar failure, excessive pillar spalling (such as occurs on a rib line), bottom heave, roof failure due to pillar punching, etc. The extent (one pillar, row of pillars, etc) of these problems should also be noted. Note also if a zone of fracturing or spalling exists around the pillars.
25. Stand up Time Prior to Failure. Enter the approximate time between mining and the observance of the stability problem.
26. Surface Indications Note if any surface subsidence resulted from the indicated problem.
27. Further Description of Failures Describe, in as much detail possible, the circumstances surrounding the pillar problem, how it was alleviated and the suspected causes. Please attach a map if available.

PILLAR DESIGN

28. Design Criteria. Describe the design method used to size the pillars in this case study. If the design criteria is solely experience, please note it.
29. Design Life. Enter the pillar standup time used for planning purposes in pillar design.
30. Estimated Coal Strength. If the design formula used requires coal strength, indicate the value used and how it was obtained.

ROOF CONTROL

31. Roof Control Procedures. Briefly describe roof control procedures in this area of the mine, i.e., 6 ft resin bolts on 4 ft centers. If possible attach any pertinent information such as a copy of the mine roof control plan.
32. Span Determination Criteria. Enter the basis for the selection of entry width used in this area of the mine such as maximum width without roof problems, equipment characteristics, etc.

ADDITIONAL INFORMATION

33. Geotechnical Studies. Have any geotechnical studies to identify poor roof conditions prior to mining or to classify roof rock been conducted?
34. Publications of Reports. If any publications or reports have been released which relate to pillar sizing at this mine please note.
35. Estimated Design Safety Factor. Note any estimate of the safety factor used in terms of your pillar design. Also comment on what you feel would be an adequate factor of safety to be applied.
36. Comments. Please include any comments which you feel are pertinent and elaborate on any questions contained in the survey which you feel require additional description.

```

/*FULLSKIPS
// EXEC RUN,PROG=WILCG
//DATA.INPUT DD *
PILLAR
SURVEY
RESPONSES
/*
// EXEC FWCG,PARM=ISNS
//SYSIN DD *
      INTEGER DIR,CONT
      COMMON CARD(13)
      CHARACTER*5 CAED
      READ(5,500) N
500 FORMAT(I5)
      DO 100 I=1,N
      WRITE(6,505)
505 FORMAT('1',51X,'PILLAR DESIGN CASE HISTORY'/ 50X,'PENNSYLVANIA
1STATE UNIVERSITY'/ 36X,'DEPARTMENT OF MINERAL ENGINEERING - GEOME
2CHANICS SECTION'/ 38X,'DEPARTMENT OF ENERGY RESEARCH PROJECT ET-7
39-G-01-3427'///)
50 READ(5,55) DIR,(CARD(J),J=1,13),CONT
55 FORMAT(I5,13A5,1X,I1)
      IF (DIR.EQ.99) GO TO 100
      IF (CONT.EQ.0) GO TO 51
      CALL CCNTIN
      GO TO 50
51 GO TO (1,2,3,4,5,6,7,8,9,10,11,12,13,14,15,16,17,18,19,20,21,22,23
1,24,25,26,27,28), DIR
1 WRITE(6,510) (CARD(K),K=1,3)
510 FORMAT(' ',5X,'IDENTIFICATION INFORMATION'/5X,' -----
1-----'//10X,'MINING COMPANY: ',A5//10X,'MINE: ',10X,A5//10X,'CA
2SE NUMBER: ',3X,A5/)
      GO TO 50
2 WRITE(6,515) (CARD(K),K=1,12)
515 FORMAT(' ',9X,'DESIGNATION FOR MINING AREA: ',10A5//39X,2A5/),
      GO TO 50
3 WRITE(6,520) (CARD(K),K=1,12)
520 FORMAT(' ',5X,'CASE STUDY STATISTICS'/5X,' -----'//
1/10X,'SEAM HEIGHT (CM): ',3A5,7X,'OVERBURDEN HEIGHT (M): ',3A5//
2 10X,'PILLAR WIDTH (M): ',A5,17X,'CENTERS WIDTH (M): ',A5// 10X,'P
3ILLAR LENGTH (M): ',A5,16X,'CENTERS LENGTH (M): ',A5// 10X,'ENTRY
4WIDTH (M): ',A5,18X,'CROSSCUT WIDTH (M): ',A5/)
      GO TO 50
4 WRITE(6,525) (CARD(K),K=1,13)
525 FORMAT(' ',9X,'ACCURACY OF PILLAR MEASUREMENTS: ',5X,13A5/)

```

```

      GO TO 50
      5 WRITE (6,530) (CARD(K),K=1,13)
530  FORMAT(' ', 9X,'APPROXIMATE DIMENSIONS OF MINING AREA: ',5X,13A
      15/)
      GO TO 50
      6 WRITE (6,535) (CARD(K),K=1,13)
535  FORMAT(' ',5X,'CASE GEOLOGY'/5X,' ---- -'// 10X,'ROOF DESCRI
      1PTION: ',13A5/)
      GO TO 50
      7 WRITE (6,540) (CARD(K),K=1,13)
540  FORMAT(' ', 9X,'BOTTOM DESCRIPTION: ',13A5/)
      GO TO 50
      8 WRITE (6,545) (CARD(K),K=1,13)
545  FORMAT(' ', 9X,'OVERBURDEN DESCRIPTION: ',13A5/)
      GO TO 50
      9 WRITE (6,550) (CARD(K),K=1,12)
550  FORMAT(' ', 9X,'SEAM IDENTIFICATION -LOCAL: ',3X,6A5// 31X,'REGION
      1AL: ',6A5/)
      GO TO 50
      10 WRITE (6,555) (CARD(K),K=1,9)
555  FORMAT(' ',5X,'MINING INFORMATION'/5X,' -----'//10X,
      1'MINING METHOD: ',3A5// 15X,'EQUIPMENT TYPE, IF GIVEN: ',2A5//10X,
      2'MINING TYPE: ',4A5/)
      GO TO 50
      11 WRITE (6,560) (CARD(K),K=1,12)
560  FORMAT(' ',14X,'STAND-UP TIME PRIOR TO RETREAT MINING: ',6A5// 15X
      1,'DATE OF MINING: ',6A5/)
      GO TO 50
      12 WRITE (6,565) (CARD(K),K=1,13)
565  FORMAT(' ',14X,'PARTIAL MINING METHOD: ',3A5// 15X,'INITIAL PILLAR
      1 DIMENSIONS (M): ',4A5// 15X,'FINAL PILLAR DIMENSIONS (M): ',4A5,
      2//15X,'DATE OF MINING: ',2A5/)
      GO TO 50
      13 WRITE (6,570) (CARD(K),K=1,13)
570  FORMAT(' ',5X,'STABILITY INFORMATION'/5X,' -----'//
      1/10X,'PILLAR PROBLEMS OBSERVED'/15X,'PILLAR: ',13A5/)
      GO TO 50
      14 WRITE (6,575) (CARD(K),K=1,13)
575  FORMAT(' ',14X,'BOCF: ',2X,13A5/)
      GO TO 50
      15 WRITE (6,580) (CARD(K),K=1,13)
580  FORMAT(' ',14X,'BOTTOM: ',13A5/)
      GO TO 50
      16 WRITE (6,585) (CARD(K),K=1,13)
585  FORMAT(' ', 9X,'STAND-UP TIME PRIOR TO FAILURE: ',13A5/)

```

```

      GO TO 50
17  WRITE (6,590) (CARD(K),K=1,13)
590  FORMAT(' ', 9X,'SURFACE INDICATION OF FAILURE: ',13A5/)
      GO TO 50
18  WRITE (6,595) (CARD(K),K=1,13)
595  FORMAT(' ', 9X,'FUTHER DESCRIPTION: ',13A5/)
      GO TO 50
19  WRITE (6,600) (CARD(K),K=1,13)
600  FORMAT('-',5X,'PILLAR DESIGN'/5X,' -----'// 10X,'DESIGN CR
1ITERIA: ',13A5/)
      GO TO 50
20  WRITE (6,605) (CARD(K),K=1,13)
605  FORMAT(' ', 9X,'DESIGN LIFE: ',13A5/)
      GO TO 50
21  WRITE (6,610) (CARD(K),K=1,13)
610  FORMAT(' ', 9X,'CCAL STRENGTH: ',13A5/)
      GO TO 50
22  WRITE (6,615) (CARD(K),K=1,13)
615  FORMAT('-',5X,'ROOF CONTROL'/5X,' ----'// 10X,'ROOF CONTRO
1L PROCEDURES: ',13A5/)
      GO TO 50
23  WRITE (6,620) (CARD(K),K=1,13)
620  FORMAT(' ', 9X,'SPAN DETERMINATION CRITERIA: ',13A5/)
      GO TO 50
24  WRITE (6,625) (CARD(K),K=1,13)
625  FORMAT('-',5X,'ADDITIONAL INFORMATION'/5X,' -----'
1'//10X,'GEOTECHNICAL STUDIES: ',13A5/)
      GO TO 50
25  WRITE (6,630) (CARD(K),K=1,13)
630  FORMAT(' ', 9X,'PUBLICATIONS OR REPORTS: ',13A5/)
      GO TO 50
26  WRITE (6,635) (CARD(K),K=1,13)
635  FORMAT(' ', 9X,'ESTIMATED DESIGN SAFETY FACTOR: ', 5X,13A5/)
      GO TO 50
27  WRITE (6,640) (CARD(K),K=1,13)
640  FORMAT(' ', 9X,'CCMMMENTS: ',13A5/)
      GO TO 50
28  WRITE (6,645) (CARD(K),K=1,13)
645  FORMAT(' ', 9X,'ADDITIONAL SUBMITTED MATERIAL: ',13A5/)
100  CONTINUE
      STOP
      END
      SUBFOUTINE CONTIN
      COMMON PNT(13)
      CHARACTER*5 PNT

```

```

        WRITE (6,1000) (PNT(K),K=1,13)
1000 FORMAT(' ',17X,13A5/)
        RETURN
        END
//DATA.INPUT DD *
      78
/*INCLUDE GWL01.SURVEY1
/*INCLUDE GWL01.SURVEY2
/*INCLUDE GWL01.SURVEY3
/*INCLUDE GWL01.SURVEY4
/*INCLUDE GWL01.SURVEY5
/*

```

1 10 01 C1
 2 FIRST PANEL LEFT OFF SOUTHWEST MAINS PANEL PANEL
 3 119 120-150 12.2 18.3 12.2 18.3 6.1
 4 FROM PROJECTIONS, WITHIN 4 PERCENT OF NOMINAL SIZE
 5 137.2M X 609.6M BUT LIMITED TO 244M DUE TO EXCESSIVE CAVING
 6 IMMEDIATE ROOF IS 3.05M OF SHALE WITH SANDSTONE STREAKS
 7 10.2 CM OF FIRECLAY, NOT TAKEN DURING MINING
 8 SHALESTONE, SANDSTONE, AND ECNEY COAL
 9 POND CREEK LOWER ELKHORN
 10 CONTINUOUS JOY 14 CM PARTIAL RETREAT
 12 SPLIT 12.2M X 12.2M LITTLE AS POSSIBLE 7/78
 13 NONE
 14 MAJOR FALLS IN LAST CROSSCUTS LIMITED PANEL LENGTHS TO 244M
 15 NONE
 16 FALLS AVE. ONE HALF DAY AFTER PULLING THE 2ND LINE OF PILLARS
 17 NONE
 18 NONE
 19 EXPERIENCE AND THE PACE OF PULLING PILLARS
 20 TWO TO THREE YEARS IN LARGE PANELS
 21 N.A.
 22 106.7CM H.S.S. BOLTS, 15.9MM DIA., EXPANSION SHELL, TORQUED AT
 120-160 FT. LBS., 1.22 X 1.22 M CENTERS
 23 TWO CUTS OF CONTINUOUS MINER
 24 NONE
 25 NONE
 26 BELIEVED TO BE GREATER THAN 3
 99

1

PILLAR DESIGN CASE HISTORY
PENNSYLVANIA STATE UNIVERSITY
DEPARTMENT OF MINERAL ENGINEERING - GEOMECHANICS SECTION
DEPARTMENT OF ENERGY RESEARCH PROJECT ET-78-G-01-3427

IDENTIFICATION INFORMATION

MINING COMPANY: 10
MINE: 01
CASE NUMBER: 01
DESIGNATION FOR MINING AREA: FIRST PANEL LEFT OFF SOUTHWEST MAINS PANEL.
PANEL

CASE STUDY STATISTICS

SEAM HEIGHT (CM): 119 OVERBURDEN HEIGHT (M): 120-150
PILLAR WIDTH (M): 12.2 CENTERS WIDTH (M): 18.3
PILLAR LENGTH (M): 12.2 CENTERS LENGTH (M): 18.3
ENTRY WIDTH (M): 6.1 CROSSCUT WIDTH (M):
ACCURACY OF PILLAR MEASUREMENTS: FROM PROJECTIONS, WITHIN 4 PERCENT OF NOMINAL SIZE
APPROXIMATE DIMENSIONS OF MINING AREA: 137.2M X 609.6M BUT LIMITED TO 244M DUE TO EXCESSIVE CAVING

CASE GEOLOGY

ROOF DESCRIPTION: IMMEDIATE ROOF IS 3.05M OF SHALE WITH SANDSTONE STRIPS
BOTTOM DESCRIPTION: 10.2 CM OF FIRECLAY, NOT TAKEN DURING MINING
OVERBURDEN DESCRIPTION: SHALESTONE, SANDSTONE, AND BONEY COAL
SEAM IDENTIFICATION -LOCAL: POND CREEK
REGIONAL: LOWER ELKHORN

MINING INFORMATION

MINING METHOD: CONTINUOUS
EQUIPMENT TYPE, IF GIVEN: JUY 14 CM

MINING TYPE: PARTIAL RETREAT

PARTIAL MINING METHOD: SPLIT

INITIAL PILLAR DIMENSIONS (M): 12.24 X 12.24

FINAL PILLAR DIMENSIONS (M): LITTLE AS POSSIBLE

DATE OF MINING: 7/79

STABILITY INFORMATION

PILLAR PROBLEMS OBSERVED

PILLAR: NONE

ROOF: MAJOR FALLS IN LAST CROSSCUTS LIMITED PANEL LENGTHS TO 244M

BEITON: NONE

STAND-UP TIME PRIOR TO FAILURE: FALLS AVE. ONE HALF DAY AFTER PULLING THE 2ND LINE OF PILLARS

SURFACE INDICATION OF FAILURE: NONE

FUTHER DESCRIPTION: NONE

PILLAR DESIGN

DESIGN CRITERIA: EXPERIENCE AND THE PACE OF PHILING PILLARS

DESIGN LIFE: TWO TO THREE YEARS IN LARGE PANELS

COAL STRENGTH: N.A.

ROOF CONTROL

ROOF CONTROL PROCEDURES: 106.7CM H.S.S. BOLTS, 15.9MM DIA., EXPANSION SHEL, TORQUED AT
120-160 FT. LBS., 1.22 X 1.22 M CENTERS

SPAN DETERMINATION CRITERIA: TWO CUTS OF CONTINUOUS MINER

ADDITIONAL INFORMATION

GEOTECHNICAL STUDIES: NONE

PUBLICATIONS OR REPORTS: NONE

