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Mine Ground Control

Proceedings: Bureau of Mines Technology Transfer Seminars, Pittsburgh, PA, December 6-7, 1983, and Denver, CO, December 8-9, 1983

Compiled by Staff, Bureau of Mines

UNITED STATES DEPARTMENT OF THE INTERIOR

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UNITED STATES DEPARTMENT OF THE INTERIOR
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PREFACE

This Information Circular summarizes recent Bureau of Mines research results concerning ground control in mining environments. The papers represent only a sample of the Bureau's total research effort in this area, but they outline major portions of the program.

The papers reproduced here were presented at Technology Transfer Seminars on Mine Ground Control given in December 1983 in Pittsburgh, PA, and Denver, CO. Those desiring more information on the Bureau's ground control program or health and safety technology programs in general, or information on specific situations, should contact the Bureau of Mines, Division of Health and Safety Technology, 2401 E Street, NW., Washington, DC 20241, or the appropriate author listed in these proceedings.

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UNIT OF MEASURE ABBREVIATIONS USED IN THESE PAPERS

A	ampere	lb	pound
cfm	cubic foot per minute	L/min	liter per minute
cm	centimeter	L/s	liter per second
deg	degree	m	meter
ft	foot	MHz	megahertz
ft/min	foot per minute	mi	mile
ft/s	foot per second	min	minute
gal	gallon	μin/in	microinch per inch
g/cm^3	gram per cubic centimeter	mm	millimeter
GPa	gigapascal	μm	micrometer
gpm	gallon per minute	MPa	megapascal
h	hour	ms	millisecond
hp	horsepower	μs	microsecond
Hz	hertz	mV	millivolt
in	inch	ns	nanosecond
in/min	inch per minute	pct	percent
kHz	kilohertz	psi	pound per square inch
km	kilometer	s	second
km/s	kilometer per second	V	volt
kV	kilovolt	wt pct	weight percent
L	liter	yr	year

MINE GROUND CONTROL

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ABSTRACT

These proceedings consist of papers presented at Bureau of Mines Technology Transfer Seminars in December 1983 for the purpose of disseminating recent advances in mining technology in the area of mine ground control. The Bureau of Mines conducts several of these seminars each year in order to bring the latest results of Bureau research to the attention of the mining industry as quickly as possible.

INTRODUCTION

By Chi-shing Wang[1] and John M. Karhnak[1]

The basic goal of the Bureau of Mines Ground Control research program is to provide the mining industry with technology that will lead to the reduction of accidents due to falls of ground. The problems of ground control are the inability to "see" geologic anomalies ahead of the mine workings, the difficulty in predicting ground movements induced by excavation, and the need to provide efficient ground control over the widely varying, and frequently unexpected, conditions encountered from one place to another. Current program objectives include--

Exploring new methodologies and validating available tools for predicting hazardous geologic features in advance of mining and for selecting and designing safe and efficient mining systems and layout plans.

Developing instrumentation methods and equipment able to detect and warn of imminent failure of ground during mining.

Ascertaining the most effective permanent support methods for specific ground conditions.

Investigating roof support installation methods that improve working conditions at the face and remove machine operators from dangerous unsupported areas.

Advance knowledge of potentially hazardous geologic features is a prerequisite to success in designing out the associated safety risks from the mining plan during the early stage of mine design and development. To enable mine operators to satisfy this requirement, the Bureau has continued its efforts in assessing the applicability of various geologic mapping methods and geophysical techniques to detection and delineation of geologic anomalies and mine voids from the surface, boreholes, or mine workings.

The ability of miners to predict potential hazards ahead of working faces and to detect impending ground failures at the working place during mining is crucial to timely evacuation and safety precautions to avoid injuries. Therefore, Bureau research continues to assess inseam geophysical techniques for detecting geologic anomalies and mine voids ahead of coal faces in underground coal mines, automatic microseismic roof fall warning systems, and mechanical and ultrasonic devices for continuous measurement of roof-to-floor closure rates.

Better understanding and prediction capability of mine geology and ground movements serve as preventive measures permitting mine operators to design out potential ground hazards, and enabling miners to avoid or escape from dangerous areas in time. However, protective measures for assurance of safe mine workings with an adequate ground support are essential to maintaining active mining operations. Hence, the Bureau has invested considerably in research of more cost-effective and more efficient temporary and permanent roof support systems, alternative ground support materials, and remotely operated roof bolt installation equipment and procedures.

This technology transfer seminar introduces to the mining industry the significant technological advances made by Bureau research in recent years in various aspects of ground control in mines. Emphasis is placed on those innovations that have been validated by industrial applications or by extensive field demonstrations. Although many of those studies and innovations were conducted in or designed for use in coal mine ground

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control, most of the new technologies can be adapted for use in various mining environments including metal and nonmetal mines. As the current research policy of the U.S. Government is to shift emphasis toward long-term, high-risk, and high-payoff basic research, the mining industry and mining equipment manufacturers are encouraged to assess the potential of immediate uses or private investments in commercial development of Bureau research products that are offered at this seminar.

MINE DESIGN

GEOLOGIC STUDIES IN GROUND CONTROL

By Noel N. Moebs[1]

ABSTRACT

Several geologic techniques are reviewed that are useful in planning mines and in solving ground control problems. A summary is presented of a study of coal mine roof structures and the use of this information in improving supplementary support and anticipating support problems.

INTRODUCTION

The Bureau of Mines has been conducting studies directed toward increasing the utilization of geologic methods in coal mine ground control, and more recently in quarry operations, for the purpose of reducing hazards from roof and rock falls. It is commonly recognized that geology is the key to effective ground control; that is, a sound knowledge of the character and structure of rock provides a sound basis for mine planning and selection of appropriate roof support methods.

For example, studies by the Bureau of Mines have identified geologic structures in mine roof rock that contribute to many roof falls in Appalachian coal mines. These structures, including paleochannels, kettlebottoms, scours, pinchouts, slickensides, clay veins, crevasse splays, and joints, can often be identified during, and sometimes before, mine development. Mine projections can be revised to reduce the adverse effects of discontinuities in roof structure, large roof areas of laminated sandstone or incompetent strata generally can be delineated or inferred from exploratory drill-hole data, and the need for supplementary support can be anticipated. Accurate descriptions of roof geology also provide some indication of optimum length and type of roof bolts that should be installed.

In addition to studying geologic structures in coal mine roof, which are described in another section, the Bureau investigated the concept of a hazard map that would delineate potentially hazardous zones prior to mining. A method of preparing a hazard map through the computer processing of drill-log data was developed, and the method was demonstrated by preparing a hazard map of a small portion of the Pittsburgh Coal Basin. This methodology also should be applicable to individual mines where an adequate density of exploratory drill holes is available. A computer program is available for drill-hole data processing, which results in a printout map of anticipated mining conditions. The identification and weighting of hazardous geologic variables are provided for by modification of the program.

Another geologic tool applied to ground control studies by the Bureau is linear analysis,[2] whereby an attempt is made to correlate linear features from aerial photographs or satellite images with subsurface rock stability conditions. The results of this research in the Appalachian coal region have, to date, been inconclusive in that at some locations a

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[2]Linear, or photolineament, is a line on an aerial photograph (or transposed from satellite imagery) that is controlled by an alignment of surface or near-surface structures or conditions.

correlation was discovered but at many it was not. The lack of precise data on the location of roof failure, the failure to identify the "ground truth" or true geologic character of a linear, and the inability to selectively choose valid lineaments all have severely limited the practical application of linear analysis to ground control studies. Continued research is urgently needed to establish the limitations and the potential of linear analysis as a valid geologic method in ground control studies.

Virtually the only direct method of obtaining data on subsurface strata prior to mining is through core drilling. Unfortunately, most drill core in the coal regions has been logged by the driller himself, and while this might have been adequate to determine coal thickness and general rock units, it is entirely inadequate for use in the assessment of anticipated roof conditions, cavability, or paleoenvironmental reconstructions to determine coal continuity and quality. Even with the increasing use of trained professional geologists to log drill core, there is still much to be desired in the accurate and uniform identification of rock types. Recognizing that ground control investigations prior to mining require the best possible drill-core records, especially where no provision is made to preserve the core, the Bureau has developed field guides for the identification of cored rock in the Pittsburgh Basin of Pennsylvania, Ohio, and West Virginia, and in the Pocahontas Basin of southeastern West Virginia. These guides are pocket size, on weatherproof paper, with a printed key to rock types and color photographs of the actual core. The Bureau produced these guidebooks on an experimental basis. They have proven to be immensely popular and useful, and as they become commercially available a great deal more information should be extracted from drill core than before.

While most geologic studies have been directed toward improving coal mine ground control, a project recently initiated in slate quarry operations utilizes some of the same research methodologies and thus is included in this overview. The ground control problems of slate quarry operations result partly from geologic discontinuities in the vertical quarry walls, which range from 80 to 560 ft in height (fig. 1). These geologic discontinuities account for an occasional rockfall or sudden collapse of an entire highwall. These failures occur despite scaling and pinning. Bolts are not employed for highwall support. Some rock pressure problems are evident on the quarry floors; the integrity of rock barriers separating quarries sometimes is in doubt; and during the winter months ice accumulations at the quarry brinks are severe, and large masses or ice stalactites that become dislodged constitute a real hazard. Research to abate these ground control problems includes the use of strain gauges to detect imminent falls of rock, the identification and mapping of geologic features that have contributed to highwall collapse, the measurement of developed rock stresses, and the testing of both mechanical and resin bolts to secure wall rock.

Following this brief review of some geologic studies for ground control, the remainder of this paper will summarize the results of research on hazardous geologic structures in coal mine roof. This research provides an example of how the recognition of coal mine roof structures can lead to improved mine planning and supplementary support. A bibliography of selected references on this topic is provided at the end of this paper. Because of the wide variance and diversity of geologic structures in mine overburden, only the most significant ones are discussed in the following sections.

FIGURE 1. - Highwall of slate quarry operation.

COAL MINE ROOF STRUCTURES

Among the causes of coal mine roof failure are the hazardous roof conditions produced by certain geologic structures. The role of these structures in mine roof falls is not always apparent because the formations involved are sometimes poorly exposed or difficult to recognize without special training and underground experience. Most of the structures are small scale, less than a few tens of feet in width. In the underground coal production environment, these structures commonly are unrecognized or are ignored by all except those responsible for the proper roof support installation.

PALEOCHANNELS

A paleochannel, or miners' "roll," is the trough-shaped remnant of an ancient stream channel that has been cut into older rock, such as the roof shale and coalbed. The channel subsequently has been filled in with younger sediments, usually sandstone. The paleochannels most troublesome in mine roof support seldom exceed about 30 ft (9.3 m) in width and range from a few tens to a few hundreds of feet in length.

Channels that are more than 30 to 50 ft (9.3 to 15.5 m) wide and have cut down through most of the coalbed are termed "washouts." Although washouts can be a severe problem in mine development or longwall operations, they generally do not cause serious roof fall problems for more than about 50 ft (15.5 m) outside the margins of the channel.

The most typical paleochannel is a linear feature, often extending across several adjacent entries. The slickensides on the undersurfaces of the U-shaped channel constitute planes of weakness and a lack of bonding. Thus the weak shales and coal adjacent to and directly beneath the sandstone-filled channel separate readily from it as soon as the underlying coalbed is mined. The most adverse roof condition occurs when the channel trend is parallel to an entry, resulting in virtually continuous roof support problems. Some shale-filled channels also exist and constitute a similar problem.

Paleochannels are best recognized by their troughlike shape, slickensided margins, "horse-belly" appearance, and generally hard, crossbedded, poorly sorted sandstone filling. Methods that have been most often successful in supporting shale channel margins are strapping and the use of wood headers, and, for severe cases, posts and crossbars. The use of angle bolts that anchor into the hard channel filling (fig. 2) is suggested as another possible means of support. Underground mapping is the best method of determining the trends of channels. Sometimes mine intersections can be relocated to avoid an individual channel, but, more often, several parallel channels occur, and the only option is to revise the entry projections to intersect channels at an obtuse angle.

Paleochannels in roof strata can rarely be detected in advance of mining by normally spaced exploratory core drilling (approximately 2,500-ft centers) because of their small dimensions. However, their presence can be inferred in areas where drill core data indicate that

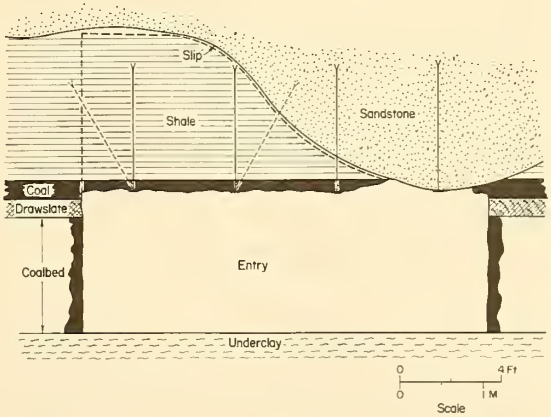

FIGURE 2. - Use of angle bolts to support shale strata at channel margins. Dashed line outlines rock not fully supported by vertical bolting.

thick, lenticular, crossbedded sandstone occurs close to the top of the coalbed.

KETTLEBOTTOMS

A kettlebottom is a columnar mass of rock embedded in and comprising a part of the coal mine roof strata. Kettlebottoms are the preserved casts of ancient tree stumps that grew in the peat swamp, now the coalbed. Once undermined, unsupported kettlebottoms can detach from the roof at any time without warning, presenting a hazard to miners. The size and frequency of kettlebottoms in mine roof is dependent upon geologic events and biological processes active during deposition of roof sediments. The preserved casts tend to be small local features of erratic occurrence, which cannot be detected by core drilling. They occur in the coal measures throughout the Appalachians, but are most abundant in the Pottsville age deposits of southern West Virginia and eastern Kentucky. All kettlebottoms not dislodged after initial mining should be properly supported.

SCOURS

The erosional action of an ancient stream produces an almost endless variety of cut-and-fill structures in rock. Scours, high curved, oblong, or saucer-shaped channel structures, must be approached with great caution, and the same roof support methods should be utilized as with the linear paleochannels.

Scours cannot be projected from entry to entry as can the persistent and linear channels, although, as with channels, their presence in mine roof is more probable where drilling indicates thick, lenticular, crossbedded sandstone close to the top of the coalbed.

PINCHOUTS

The term "pinchout" is used here to designate the abrupt termination of a roof stratum (fig. 3). Some pinchouts have been observed near the flanks of channels and washouts, and presumably were formed by the same cutting action. Others appear to be due to very rapid thinning during normal sediment deposition.

If a pinchout occurs in a stratum that is fairly thick (more than 1.5 ft or 0.5 m), competent, and located in the immediate roof, the beam strength of the roof will be weakened seriously by the discontinuity. Pinchouts are not easily detected until exposed by a roof fall. They can sometimes be discovered or inferred during drilling of roof bolt holes when the roof bolter finds a pronounced contrast in penetration rates between opposite sides of the entry. A pinchout requires effective roof bolting to reinforce the weakened beam structure in the roof through the normal roof support plan or supplementary longer bolts that anchor into competent overlying strata.

The pinchout most troublesome in mine roof is a relatively small local feature and cannot be detected by normal exploratory core drilling. However, its presence can be inferred in areas where the same individual sandstone strata close to the coalbed do not occur in adjacent holes.

SLICKENSIDES

Probably the most common hazardous roof structure occurring throughout the Appalachian coal region is the slickenside, or "slip." It is best developed and most common in highly argillaceous rocks such as shales, claystones, or miners' "clod." A slickenside is a smooth, polished, and sometimes striated or grooved surface resulting from movement of rock on either side of the surface. A slickenside usually is curved and in coal mine roof rock generally is convex toward the coalbed.

The slickenside constitutes a discontinuity in the beam roof structure and therefore weakens the roof, creating a potential hazard unless detected and properly supported. It is difficult to estimate the extent of the slickenside simply by observing the trace in the mine roof, but it can probably be assumed that the longer the trace, the greater the extent.

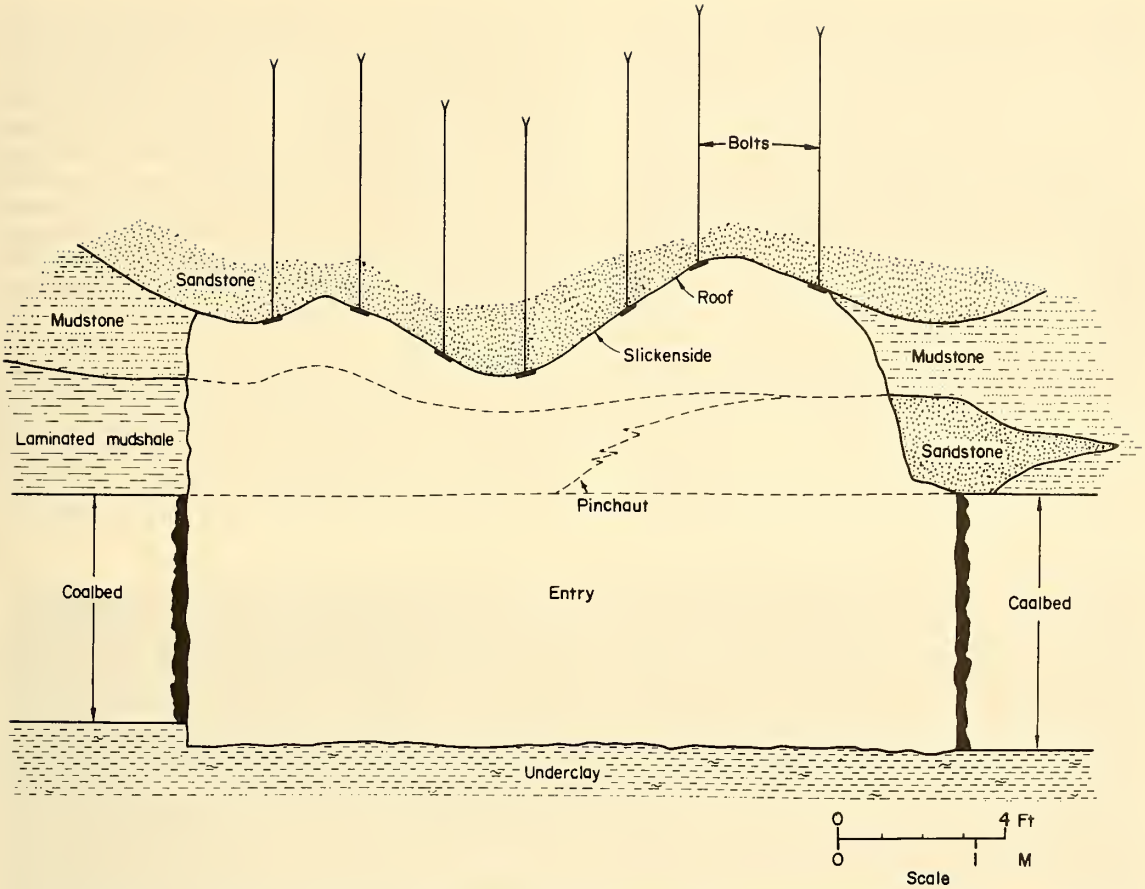

FIGURE 3. - Abrupt termination of sandstone roof stratum. Dashed lines show roof structure when first exposed; original roof was at top of the coalbed.

Small-sized slickensides (less than 3 ft or 0.9 m), nearly always found in mine roof, are adequately supported by the normal roof support plan at each mine. However, large slickensides should be regarded with extreme caution, and some form of supplementary support used. In practice, this commonly consists of extending the support area of the roof bolts by using straps or wood headers, installing additional bolts, or angle bolting to more likely penetrate the slickenside at a right angle and avoid cantilevered segments of roof rock (fig. 4). In situations where heavy concentrations of slickensides are encountered, it is advisable to consider using posts and crossbars, full column resin bolts, or, in severe cases, roof trusses.

There is currently no practical method of detecting slickensides in advance of mining, although, if core drilling indicates that the immediate roof consists of a nonlaminated claystone, then slickensides almost certainly will be present.

CLAY VEINS

Clay veins, or more properly, claystone dikes, are wedge-shaped masses of slickensided claystone or mudstone filling a crevice in a coalbed (fig. 5). These generally range up to 6 ft (2 m) in width and persist for sometimes hundreds of feet in length.

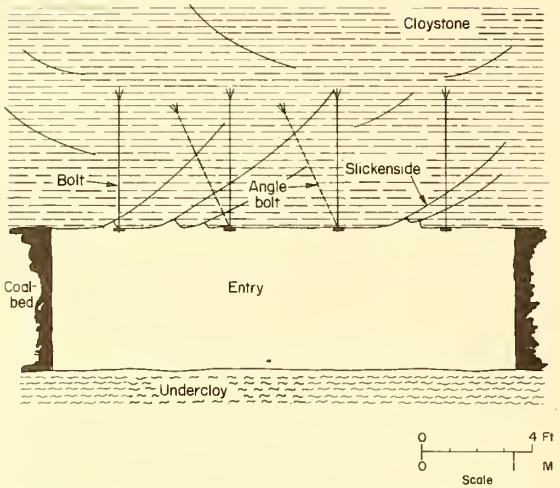

FIGURE 4. - Angle bolting of slickensides.

Clay veins are particularly prevalent and troublesome in the Pittsburgh Coalbed near Wheeling, WV, but occur sporadically throughout the Appalachian region and in many other coalbeds. Sometimes the term "spar" is applied to a very narrow clay vein or one that extends downward only a few inches into the coalbed. Other types of clay veins, such as steeply dipping, narrow, claystone-filled joints, occur in the Appalachian region but are rare and seldom cause roof problems.

The weakening effect of clay veins on mine roof extends from 3 to 12 ft (0.9 to 3.7 m) above the top of the coalbed. In addition to forming a discontinuity in the coalbed and immediate roof, the clay vein and surrounding rock are heavily slickensided and therefore likely to

FIGURE 5. - Clay vein in entry rib and roof.

break away from the roof in thick blocks as the supporting coalbed is mined. The slickensides along the margins and interior of a clay vein tend to be aligned parallel to the trend of the vein and to dip toward the center of the vein forming a troughlike effect. While prompt bolting, blocking, and strapping prevent immediate falls of roof, the clay-rich rock is susceptible to slaking and gradually disintegrates, falling little by little.

Clay veins have rarely, if ever, been detected by exploratory core drilling because of their narrow width. They are particularly abundant and should be expected where the immediate roof consists of a thick, clay-rich rock, although they also occur immediately beneath limestone, sandstone, or shale strata where perhaps the clay-rich layer has been removed by erosion. Clay veins can be predicted in advance of mining only by underground mapping and judicial projection along their trends.

CREVASSE SPLAYS

The term "crevasse splay," used here in a nongenetic, descriptive sense, designates a lithologic unit consisting of sandstone thinly interbedded with shale, or thin-bedded, laminated, micaceous sandstone (miners' "stackrock"). The unit ranges from 6 to 30 ft (2 to 9.3 m) in thickness and persists laterally as a sheetlike or lenticular body (fig. 6).

The splay type deposit that is significant in mine roof stability is the so-called low splay, where a predominantly flat-bedded, laminated sandstone unit lies within 6 to 10 ft (2 to 3 m) of the top of the coalbed and, therefore,

FIGURE 6. - Example of splay type deposit in mine roof.

constitutes part of the immediate roof. Splays that occur much higher than 10 ft (3 m) above the coalbed do not directly affect roof conditions.

Roof falls in splay sequences generally occur when a separation along a bedding plane occurs at or above the horizon of the roof bolt anchors, and the bolts are unable to maintain the integrity of the rock spanning the immediate roof. Sometimes failure can be prevented by staggering the lengths of roof bolts so that tensile stresses are not concentrated along one horizon. Full-column resin bolts resist slippage along bedding planes and thus reduce roof sag, which might reduce the possibility of failure.

A low splay deposit can be inferred from exploratory drill core information that indicates a laterally widespread, thin-bedded, laminated sandstone unit lying within 6 to 10 ft (2 to 3 m) of the coalbed, generally with some shale above and below and situated adjacent to channel-fill formations.

JOINTS

The term "joint" is used here to designate a nearly vertical, planar fracture in rock, as distinguished from a slickenside, which is a curved, grooved surface along which some movement has occurred. In the central Appalachian coal region, where coal measure strata are nearly horizontal, joints in mine roof rock usually are intraformational and of minor significance to roof stability. Roof falls may be terminated by a joint plane, but there is little evidence to indicate that the presence of a joint directly contributes to a fall.

The situation is different along the eastern margin of the Appalachian coal region where the strata have been deformed by folding and faulting. Here, jointing is more highly developed and significantly contributes to roof problems, especially where the joints occur as closely spaced, parallel sets or groups. Highly jointed roof sometimes can be supported satisfactorily with bolted channel, strap, or mat, but severe conditions may require trusses or posts and crossbars.

Highly jointed roof is very difficult to predict with certainty, but some degree of success has been claimed by people in the field, using interpretation of high-altitude aerial photography and satellite imagery, with the assumption that at least some photolineaments represent rock joint sets. It is essential to conduct a critical field check to determine the so-called ground truth of a linear lest cultural features be mistaken for geologic structures. In some areas, linear images are quite abundant, and the problem is to distinguish between those that are significantly related to roof conditions or joints and those that are not--a task for the most skilled and conscientious technologist.

DISCUSSION

There are a number of reasons for mine roof failure. The cause of some roof falls is obscure and attributed to poorly understood stress concentrations surrounding the mine openings; these falls occur most commonly when mine entries are located beneath stream valleys, and topographic relief is 200 ft (62 m) or more. Some falls result from poor roof-bolting techniques or improper installation of bolts. However, most roof falls associated with the geologic hazards referred to in this paper occur because unusual geologic structures are not recognized or anticipated and adequate support is not provided.

The identification of the structures that have been described here will require either the services of a geologist with underground experience or the training of miners (especially mining machine and roof-bolting machine operators) and their supervisors. Through regular examination of mine roof structures and conditions and the recording of this

information systematically on the operating maps, and through some trial and error, it should become possible to determine the trends of hazardous structures so that mine entry projections can be revised or potentially troublesome zones anticipated, roof support practices upgraded, and roof falls reduced.

The task will not be easy because of the complex character of many geologic features, but, in consideration of both the high priority of accident prevention and the immense financial investment at stake in developing a mine, the attempt to identify and properly support roof structures should become an integral part of every underground operation.

The application of geologic methods to other mines and quarry operations should provide similar opportunities to improve ground control practices and thus reduce the inherent safety hazards.

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COAL AND ROCK PROPERTIES FOR PREMINE PLANNING AND MINE DESIGN

By Richard E. Thill[1]

ABSTRACT

Nearly all phases of the mining operation require input on the engineering properties of rock. Engineering properties are particularly useful in the premine investigation and mine design phases of the mining operation.

Because of the general scarcity of engineering property data for coal measure strata in major U.S. coal districts and the lack of organized data bases to house and disseminate such information, the Bureau of Mines undertook a wide-ranging testing program to determine engineering properties of coal measure rocks in several U.S. coal basins. The Bureau also initiated the development of a data base for the housing, sorting, manipulation, and retrieval of property data by computer. Typical results are given for both field and laboratory determinations of geotechnical, mechanical, and geophysical properties.

INTRODUCTION

Engineering properties are important in nearly every phase of mining, but particularly in mine design, layout, and ground control. Reviews of current literature revealed that (1) not many data exist for the engineering properties of strata associated with U.S. coal deposits, (2) data available are scattered in the literature and difficult to retrieve, and (3) very few data exist in data sets that permit correlations to be made between properties for predictive purposes.

Because of the general lack of data in major U.S. coal-producing districts, the Bureau of Mines undertook a testing program to determine a variety of engineering properties, including sets of geological, geophysical, physical, and mechanical properties, in several major U.S. coalfields. In addition, data from earlier Bureau studies on all types of crystalline and sedimentary rock were summarized in data tables and incorporated into a rock property data base. The testing program and typical examples of data and analyses for the coal measure rocks, and a brief description of the mechanical properties data base and data tables are presented in this paper. The property test results are being published in Bureau and outside publications to assist mine designers and planners in the design of openings and support structures, and in the selection of appropriate excavation and support equipment.

COAL MEASURE ROCK TEST PROGRAM

The testing program consisted of both laboratory and field investigations on rock core taken at several field sites (fig. 1). In the field, lithologic and geotechnical descriptions of core were made and downhole geophysical logging was performed. Core then was boxed and shipped to the Bureau for property testing. Portions of the tests in Illinois rock were conducted under contract.

[1]Supervisory geophysicist, Twin Cities Research Center, Bureau of Mines, Minneapolis, MN.

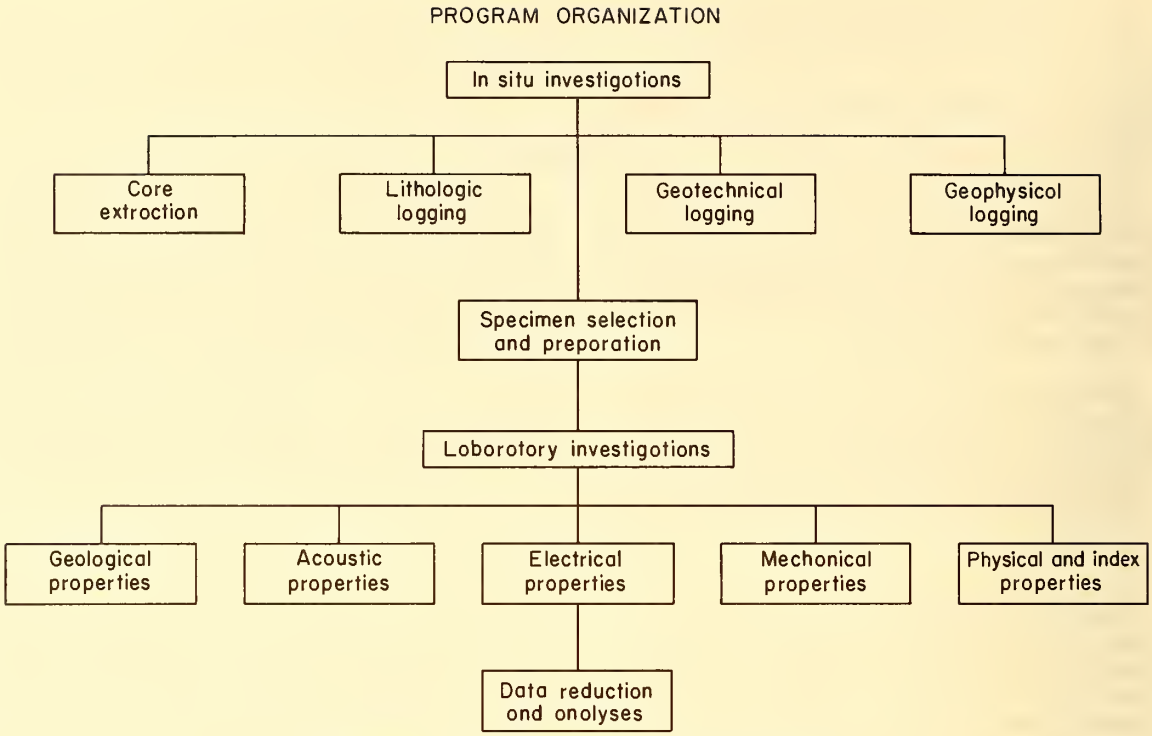

FIGURE 1. - Coal measure rock testing program.

In situ properties determined included compressional (P) and shear (S) wave velocities, formation density, derived elastic moduli, and other geotechnical properties. Laboratory determinations included mechanical properties such as uniaxial, triaxial, and tensile strength, and static elastic properties; acoustic properties such as P- and S-wave velocities, dynamic elastic moduli, and acoustic impedance and attenuation; petrographic properties; electrical properties such as dielectric constant, dissipation factor, conductivity, and skin depth; and physical or index properties such as bulk density, porosity, permeability, shore hardness, slake durability, and point-load strength index.

MINESITES

The several minesites investigated included the Gateway Mine in Pennsylvania, the York Canyon Mine in New Mexico, and three sites in the Illinois Basin. Table 1 indicates the types of studies undertaken at each site. The Gateway and York Canyon studies were more extensive and comprehensive than those in the Illinois Basin.

TABLE 1. - Properties determined at different sites

Property test or characteristic	Gateway Mine, southwestern Pennsylvania	York Canyon Mine, northern New Mexico	Illinois Basin	
			East-central Illinois	Southern Illinois and Indiana
In situ:				
Lithologic.....	X	X	X	X
Geotechnical.....	X	X	X	
Geophysical.....	X	X	X	
Laboratory:				
Petrographic.....	X	X	X	
Mechanical.....	X	X	X	X
Acoustical.....	X	X	X	X
Electrical.....	X			
Physical and index.	X	X	X	X
Environmental effects:				
Stress.....	X	X	X	X
Moisture.....	X	X	X	X

The Gateway Mine is located in the northern part of the Eastern Coal Province (fig. 2). Core was taken with a conventional drill rig equipped with a 20-ft-long, wire line retrievable, inner core barrel (fig. 3). Roughly 500 ft of

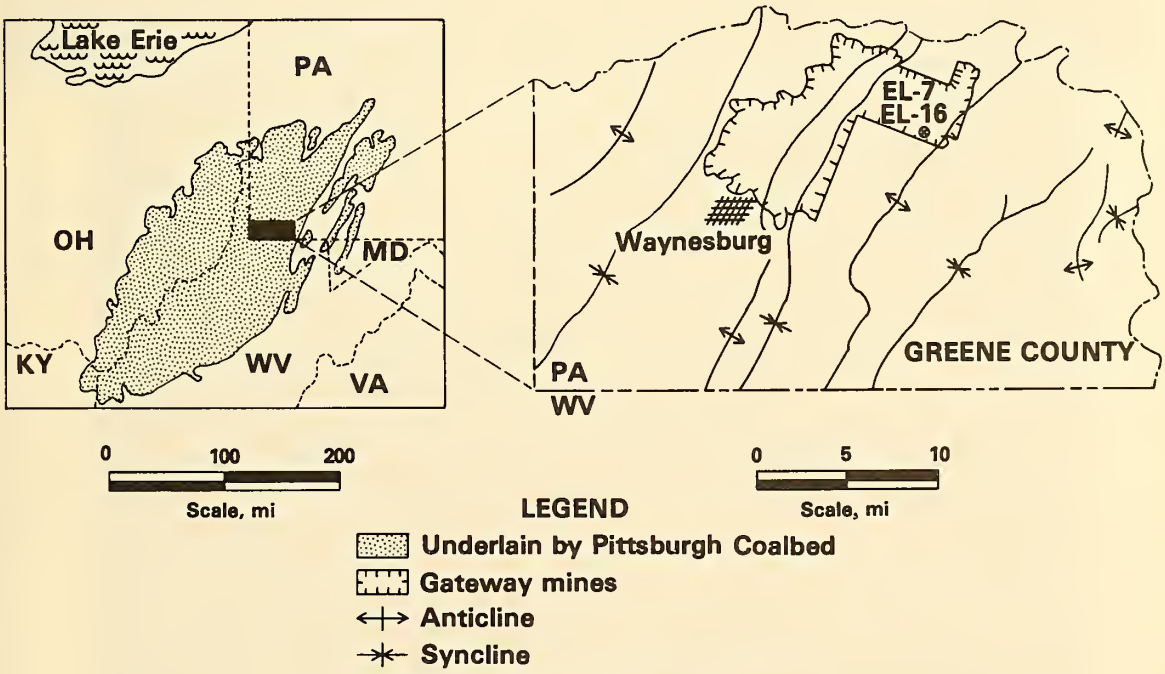

FIGURE 2. - Location of Gateway minesite.

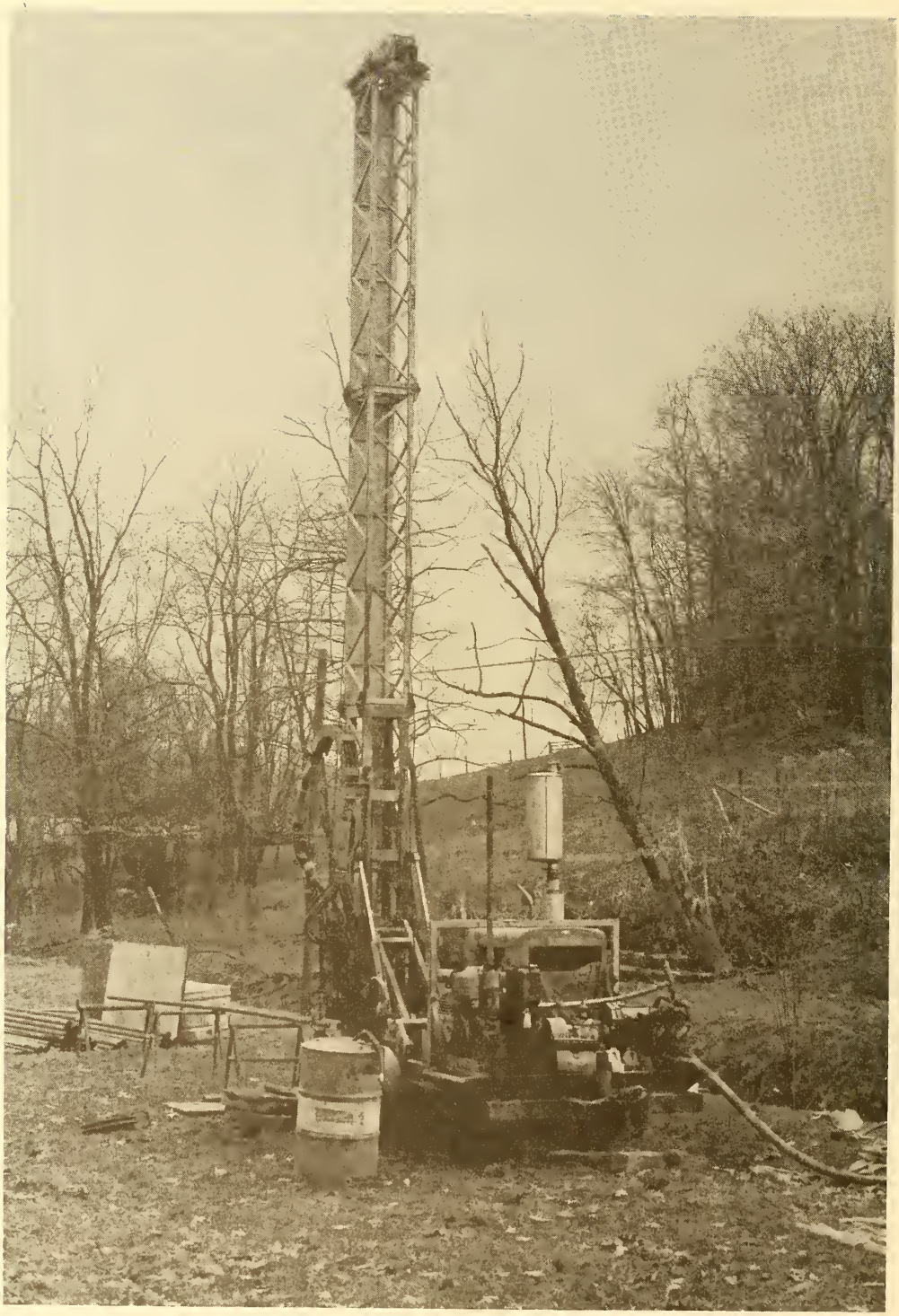

FIGURE 3. - Drill rig at the Gateway site.

FIGURE 4. - Core extraction at the Gateway site.

core was taken at the site (fig. 4) and logged for lithology, percent recovery and rock quality (RQD). Three-dimensional acoustic and gamma-gamma density surveys also were conducted in the borehole.

The York Canyon site is in the northeastern portion of the Southwest Coal Province (fig. 5). Figure 6 shows the drill rig used at the site. As core was taken from the hole, natural moisture content was preserved by wrapping the core in tinfoil and sealing it with wax (fig. 7).

The localities of the Illinois Coal Basin sites are shown in figure 8. The Danville, IL, site is located in the east-central portion and the other two sites are located in the southeast portion of the Interior Coal Province.

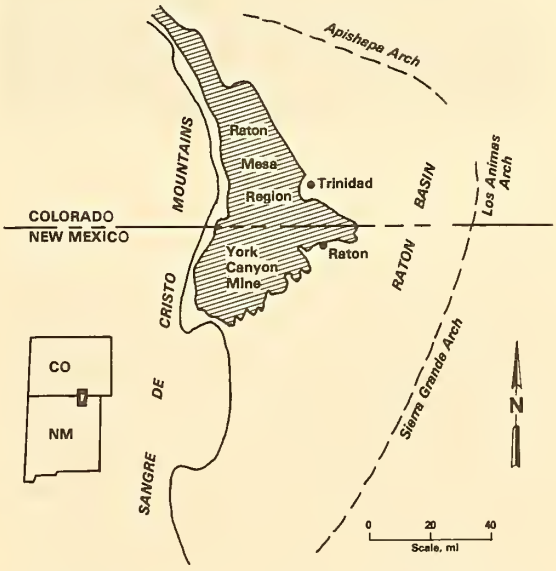

FIGURE 5. - Location of York Canyon Mine.

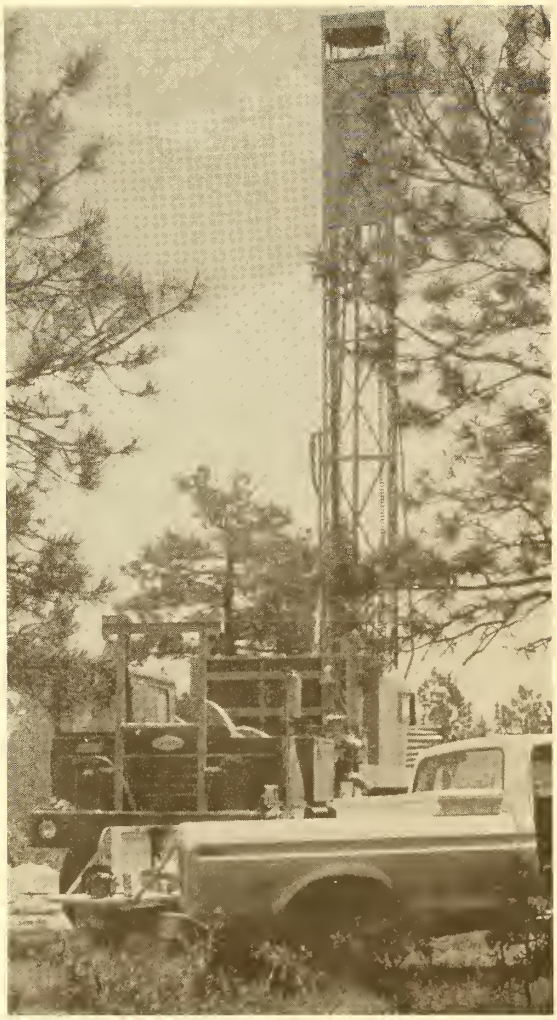

FIGURE 6. - Drill rig at the York Canyon site.

FIGURE 7. - Sealing moisture content in the York Canyon core.

FIGURE 8. - Location of Illinois sites.

LABORATORY TESTING

Test specimens were prepared from the core arriving from the field. Laboratory property determinations included petrographic, acoustic, electrical, mechanical, physical, and index properties. Specimen preparation and testing was conducted in accordance with standardized procedures described in the Bureau of Mines Test Procedures for Rocks (1)[2] and/or as recommended by the International

[2]Underlined numbers in parentheses refer to items in the list of references at the end of this paper.

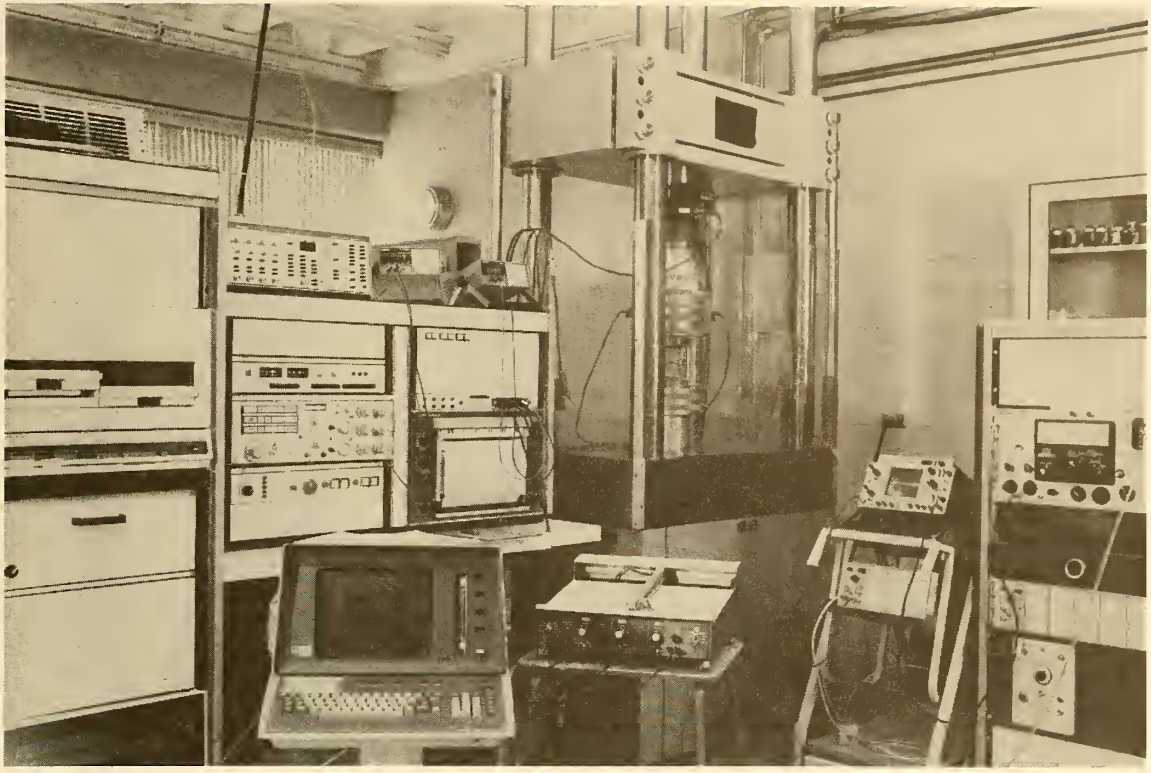

FIGURE 9. - Mechanical property testing apparatus.

Society of Rock Mechanics (2). The mechanical property testing facilities include two hydraulic, servo-controlled testing machines, one of 200,000-lb and the other of 500,000-lb load capacity (fig. 9). For triaxial confinement testing, separate servo controls are used for applying the uniaxial and confining pressure.

RESULTS

FIELD GEOTECHNICAL AND GEOPHYSICAL PROPERTIES

As core was taken in the field, lithologic descriptions were made to construct stratigraphic columns for the vicinity of the working coal seam and the overburden. As indicators of rock quality, percent recovery, RQD, and fracture frequency were measured. Figure 10 represents results obtained at the Gateway minesite. Poor quality rock is designated by zones having low RQD, low core recovery, or high fracture frequency.

Borehole geophysical logs were run at the Gateway, York Canyon, and Danville sites. The logs obtained at the Gateway site (fig. 11), for instance, permit comparison between elastic wave velocity and density response. Coal seams and fracture zones are readily detected by anomalously low-velocity, low-density response. From the full waveform velocity log, both P- and S-wave velocities can be determined, and when combined with density, these permit calculation of the dynamic elastic moduli, indicating stress-strain response and, by correlation, the

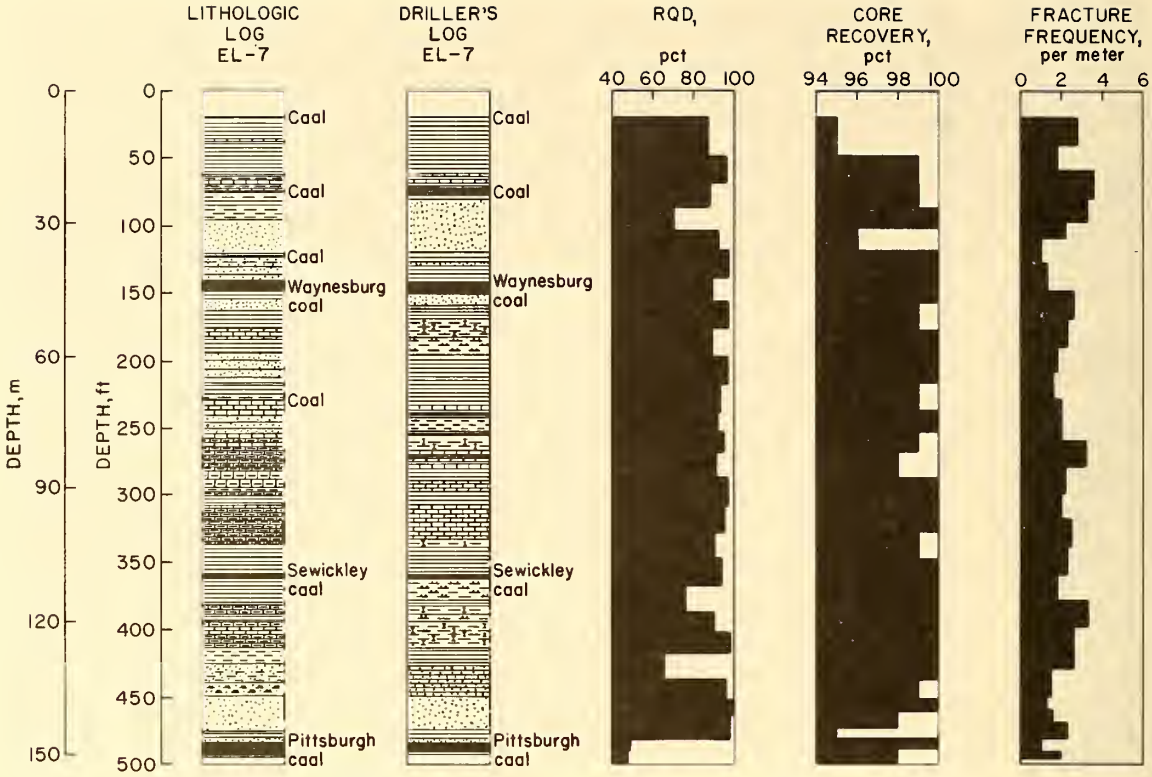

FIGURE 10. - Geotechnical logs for the Gateway site.

strength of various strata in the overburden. The geophysical data, in addition, are useful for interpreting stratigraphic boundaries and changes (3).

LABORATORY GEOPHYSICAL PROPERTIES

Geophysical properties of coal measure strata, such as elastic wave velocities and electrical properties, are not well documented in the literature but are required for the design of geophysical probes and interpretation of field geophysical data. Hence, laboratory tests were conducted for these properties. Acoustic core logging was conducted on the core as it arrived from the field, before specimens were prepared for other laboratory tests. Measurements of P-wave traveltimes were made in different directions parallel to the bedding at 0.5- to 1-ft depth increments along the core. Velocity averages for the plane and velocity difference expressed in terms of anisotropy were then plotted as functions of depth and compared with the density and lithologic logs (fig. 12). As in field results, combinations of low velocity and density normally signify coal seams and fractured zones. High values of velocity anisotropy usually suggest fracturing and poor quality of rock (4).

Although electrical properties were determined only in the Gateway strata for the dry condition, hundreds of tests were conducted in various categories of coal measure rocks over frequency ranges from 1 kHz to 100 MHz. Predicted propagation distance (skin depth) for electromagnetic energy over the frequency range from 1 kHz to 20 MHz is indicated for the composite rock types in figure 13.

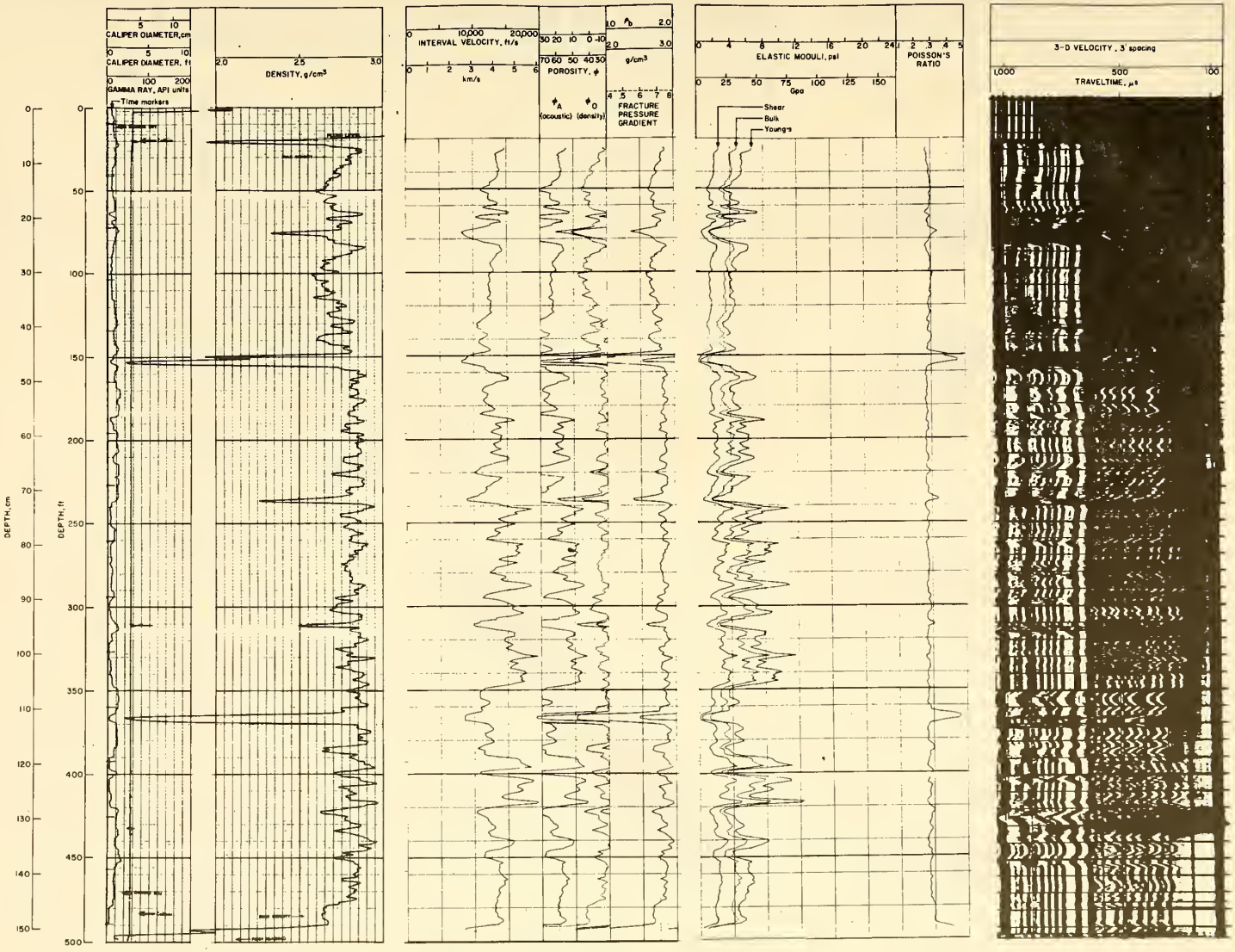

FIGURE 11. - Geophysical logs at the Gateway site.

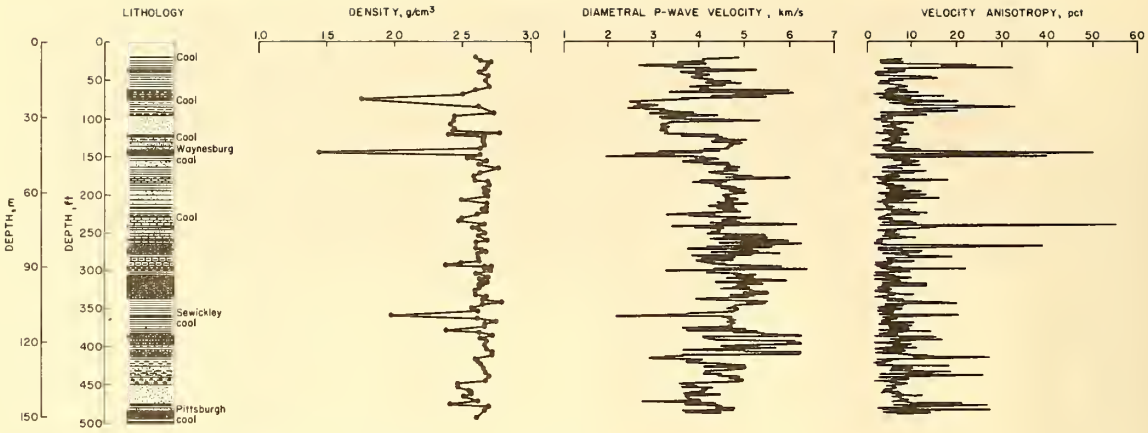

FIGURE 12. - Laboratory wave velocity and density profiles.

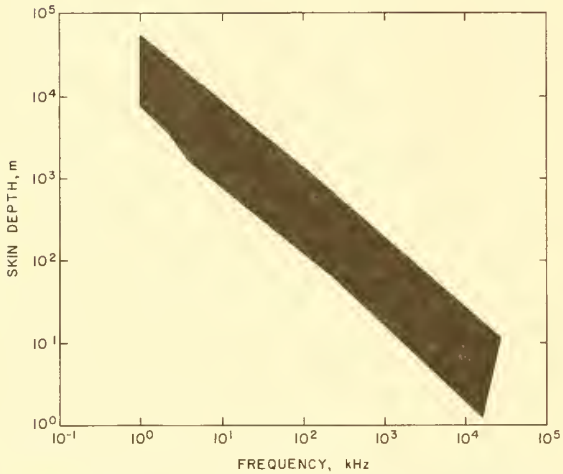

FIGURE 13. - Skin depth as a function of frequency.

MECHANICAL PROPERTIES

Mechanical properties were determined in uniaxial and triaxial compression tests using the closed-loop, servo-controlled testing machines and confining pressure cell. Resultant data were plotted as axial, circumferential, and volumetric stress-strain curves (fig. 14). Apparent Poisson's ratio, that is, the ratio of lateral to axial strain, was also determined throughout the range of stress. Concurrent with uniaxial compression testing, the change in P-wave velocity as a function of stress was determined (fig. 15). The effect of confinement on shear strength was determined using conventional Mohr-Coulomb plots and analysis (fig. 16). Although stress-strain behavior in most of the coal measure rocks exhibited nonlinear behavior, the linear Coulomb failure criteria appeared to provide a good first approximation to shear strength for most of the rock types tested. Confining pressures ranged up to 2,500 psi (17.25 MPa), simulating expected lithostatic pressures to depths of roughly 2,750 ft. Typical triaxial test results are given in table 2. Enhancement of compressive strength with confinement is indicated in figure 17. More detailed analyses, predicting failure by more complicated failure criteria, are still underway. Indirect tensile strength also was determined using the Brazilian test method.

TABLE 2. - Summary of triaxial test results

Lithologic description	Depth interval, ft	Bed thickness, ft	Number of samples tested[1]	Cohesion coefficient, MPa	Coefficient of internal friction	Angle of internal friction, deg	Correlation coefficient	Mean fracture angle, deg	Average moisture content, pct
Sandstone.....	95.3-120.0	24.7	59	12.5	0.42	23	0.996	30	0.08
Do.....	153.3-159.5	6.3	11	18.5	.58	30	.999	31	.15
Do.....	241.7-253.9	12.2	13	16.0	.50	27	.976	32	.60
Do.....	448.8-473.5	24.7	31	17.0	.64	33	.999	28	.11
Sandy shale.....	127.9-142.2	14.3	24	17.0	.60	31	.991	28	.43
Interbedded sandstone and shale.	189.0-216.4	27.4	47	14.0	.44	24	.987	27	.49
Sandy siltstone...	438.7-448.8	10.1	13	18.5	.83	40	.953	30	.42
Shale.....	366.5-378.5	12.0	17	17.0	.78	38	.939	28	.70
Shaley limestone..	310.6-336.7	26.3	58	17.0	.92	43	.977	24	.33
Limestone.....	394.0-405.6	11.6	30	11.5	.77	38	.992	27	.14

[1]Comprises uniaxial and triaxial compression, and Brazilian tensile test data used to derive Mohr circle plots.

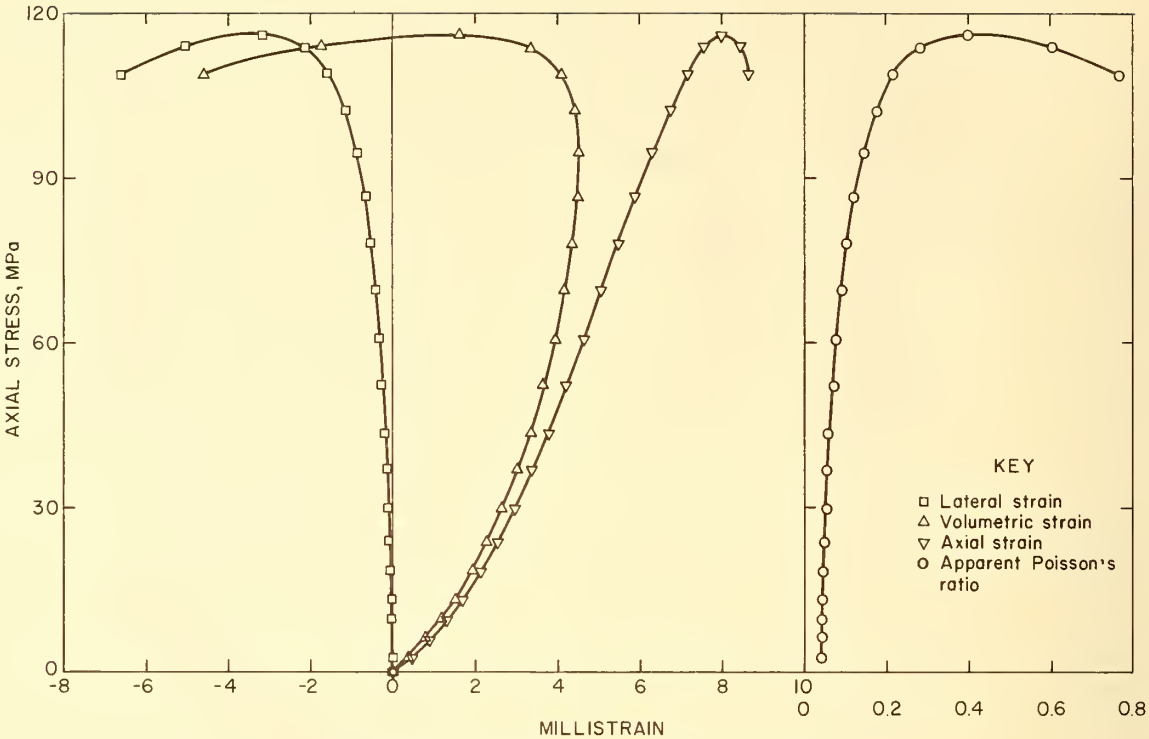

FIGURE 14. - Volumetric stress-strain curve.

PHYSICAL AND INDEX PROPERTIES

Physical and index properties determined in the laboratory included shore hardness, porosity, permeability, bulk density, point-load strength index, and slake durability. Moisture content was determined in the specimens tested for uniaxial and triaxial compression strength. In certain cases, studies also were made of the effect of moisture on elastic and strength properties and on the elastic wave velocities.

STATISTICAL ANALYSES

All rock property data were input into a computer for further analyses. Statistical analyses were conducted to obtain the mean, standard deviation, and coefficient of variability for all measurements in a particular rock type at each site. Bar charts were constructed providing the mean and range (one standard deviation) of property values for each property (figs. 18-19). Because of lithologic variability within each rock category, standard deviations frequently were quite large.

Crossplots were constructed to determine correlations between rock properties by regression analyses for each rock type (fig. 20). Correlation coefficient matrices were obtained to indicate the degree of correlation between any two properties. Those properties that exhibit significant correlation with elastic moduli and strength properties and that could be easily and inexpensively obtained during exploration investigations might, in some cases, be used as index properties to predict modulus or strength (fig. 21).

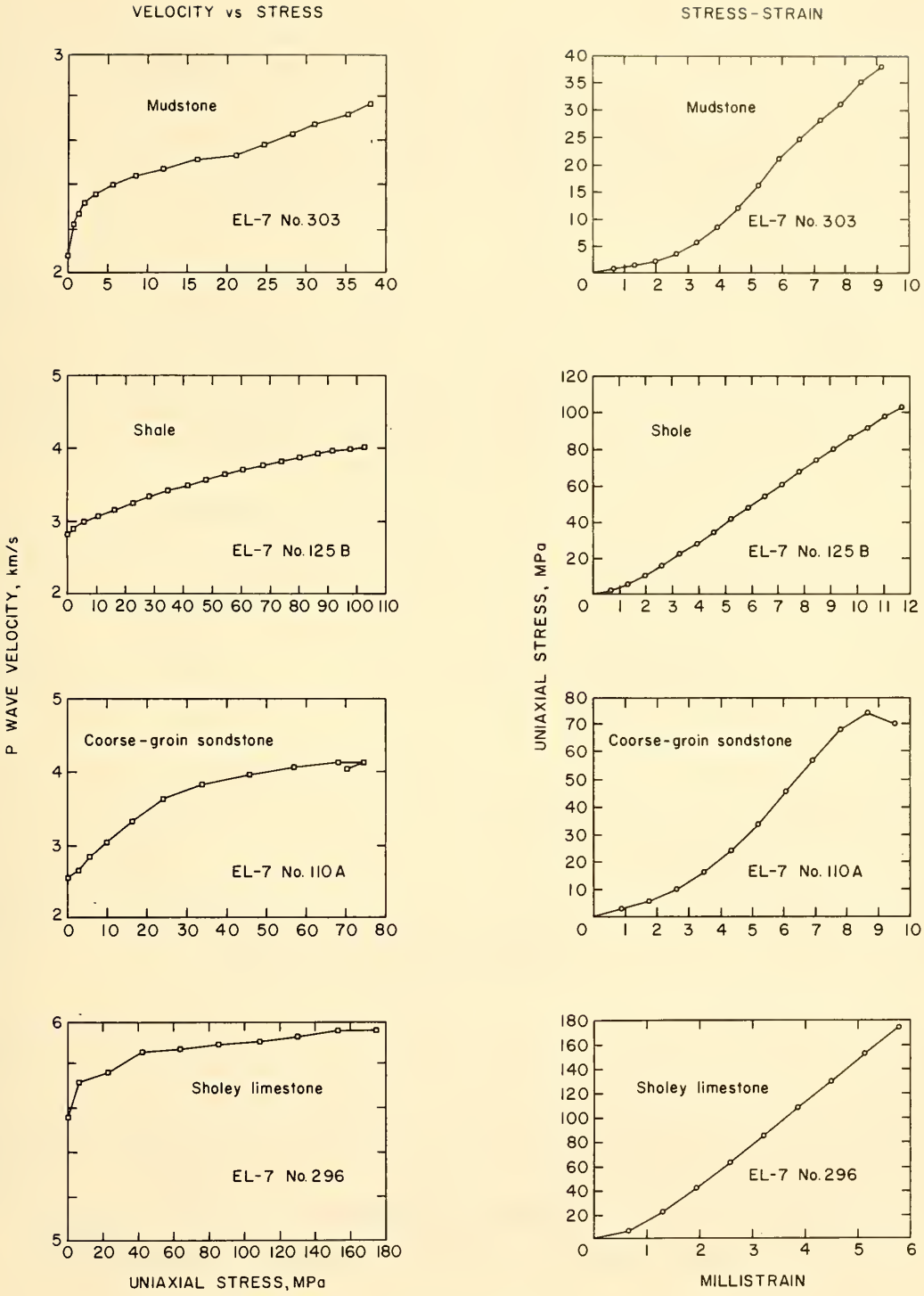

FIGURE 15. - P-wave velocity as a function of uniaxial stress.

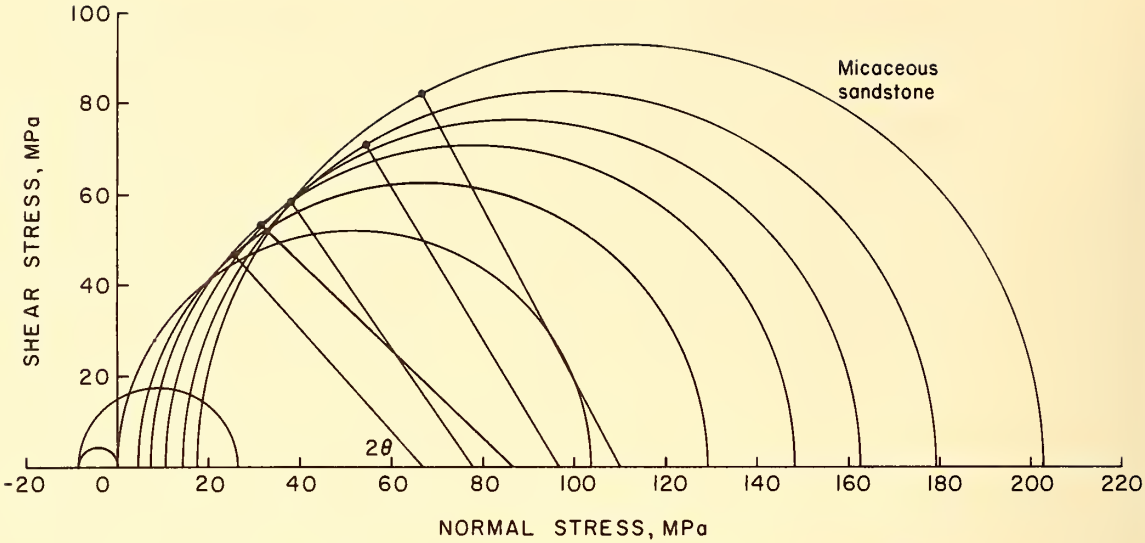

FIGURE 16. - Mohr-Coulomb plot for the Pittsburgh Sandstone.

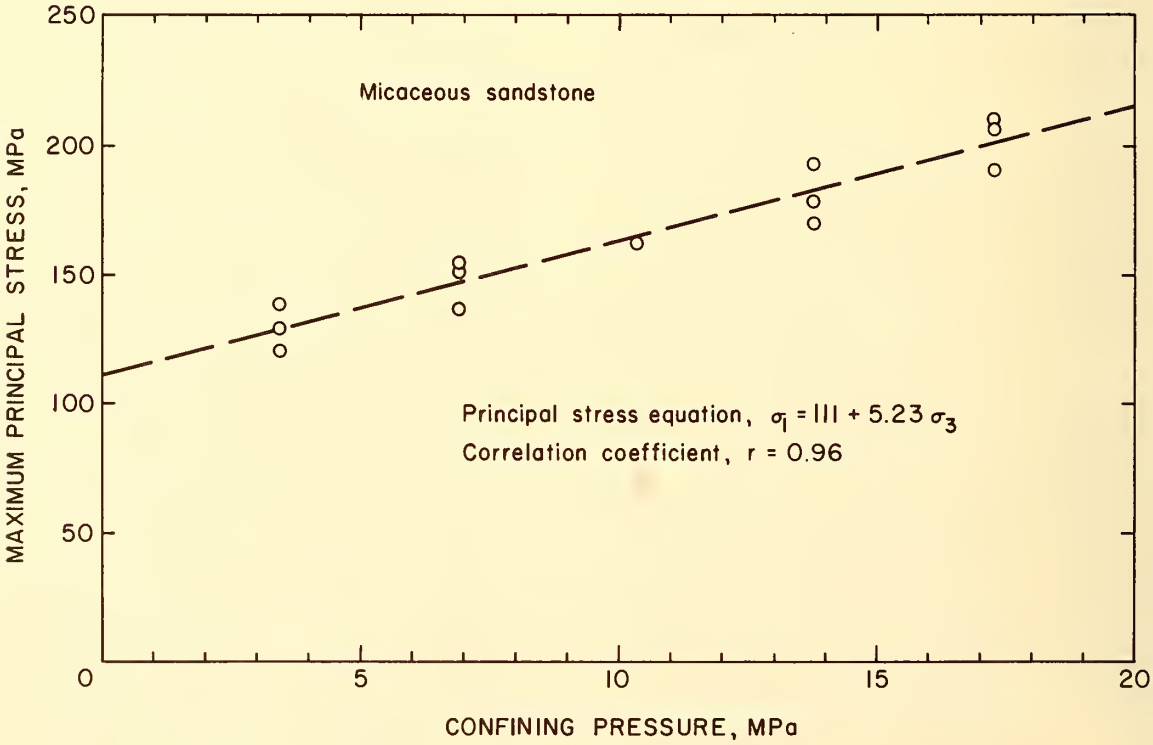

FIGURE 17. - Increase in compressive strength with confinement–Pittsburgh Sandstone.

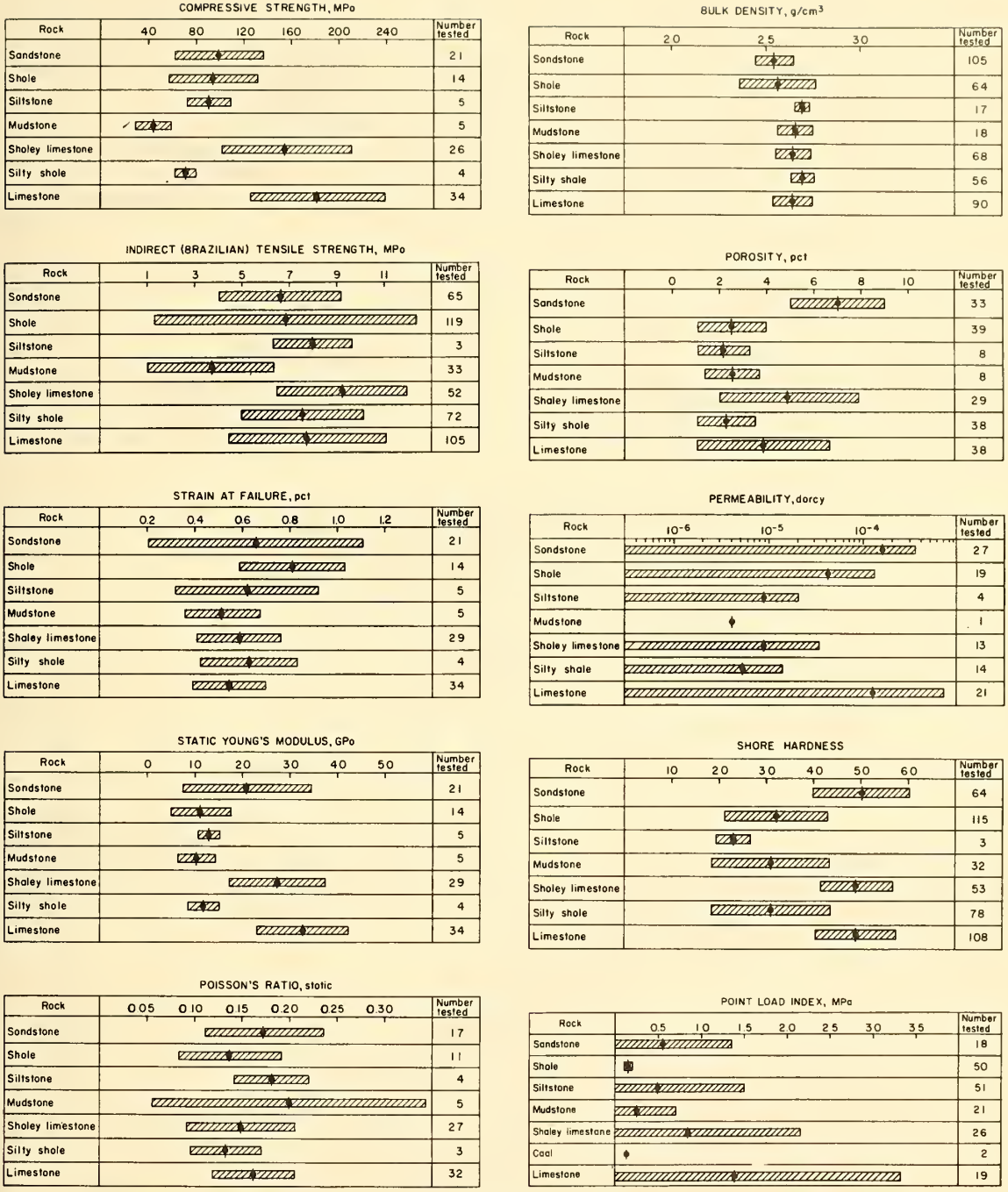

FIGURE 18. - Physical-mechanical property mean and standard deviation by rock type.

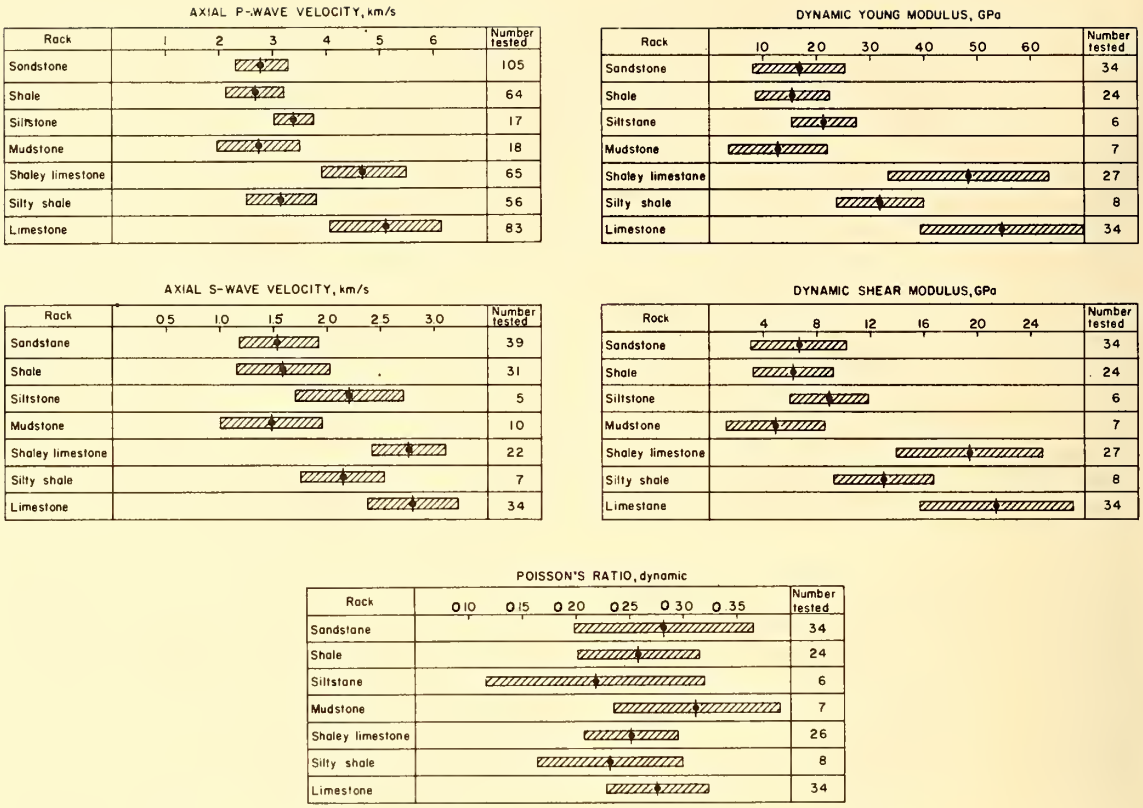

FIGURE 19. - Acoustical property mean and standard deviation by rock type.

ROCK CLASSIFICATION FOR ENGINEERING PURPOSES

INTACT ROCK

The strength and the modulus ratio were plotted according to the Deere (5) intact-rock classification scheme that is widely used in civil engineering and mining (fig. 22). The scheme describes the rock in terms of five categories of compressive strength, and three categories, high, medium, and low, of the ratio of modulus to strength.

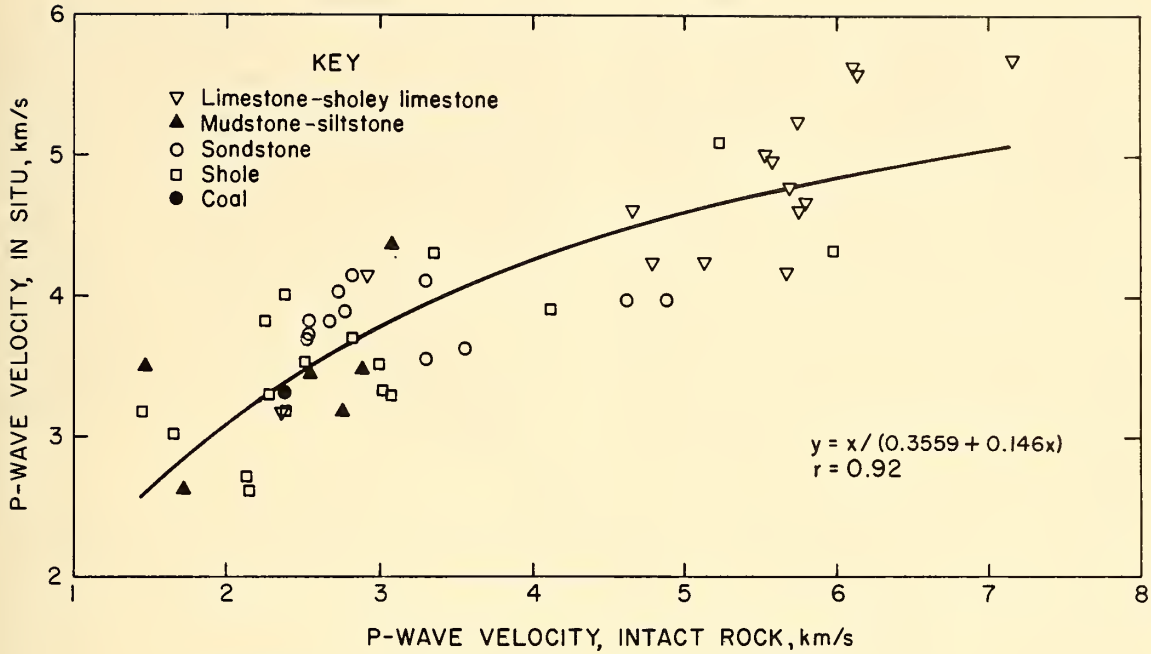

FIGURE 20. - Crossplot of in situ versus laboratory P-wave velocity.

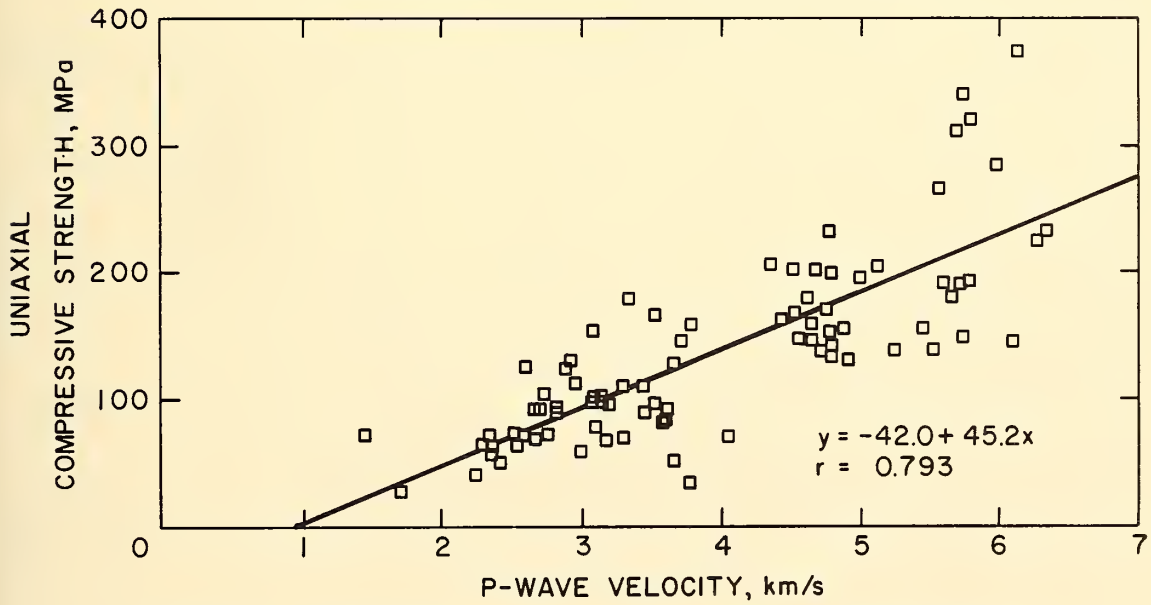

FIGURE 21. - Crossplot of uniaxial compressive strength versus P-wave velocity.

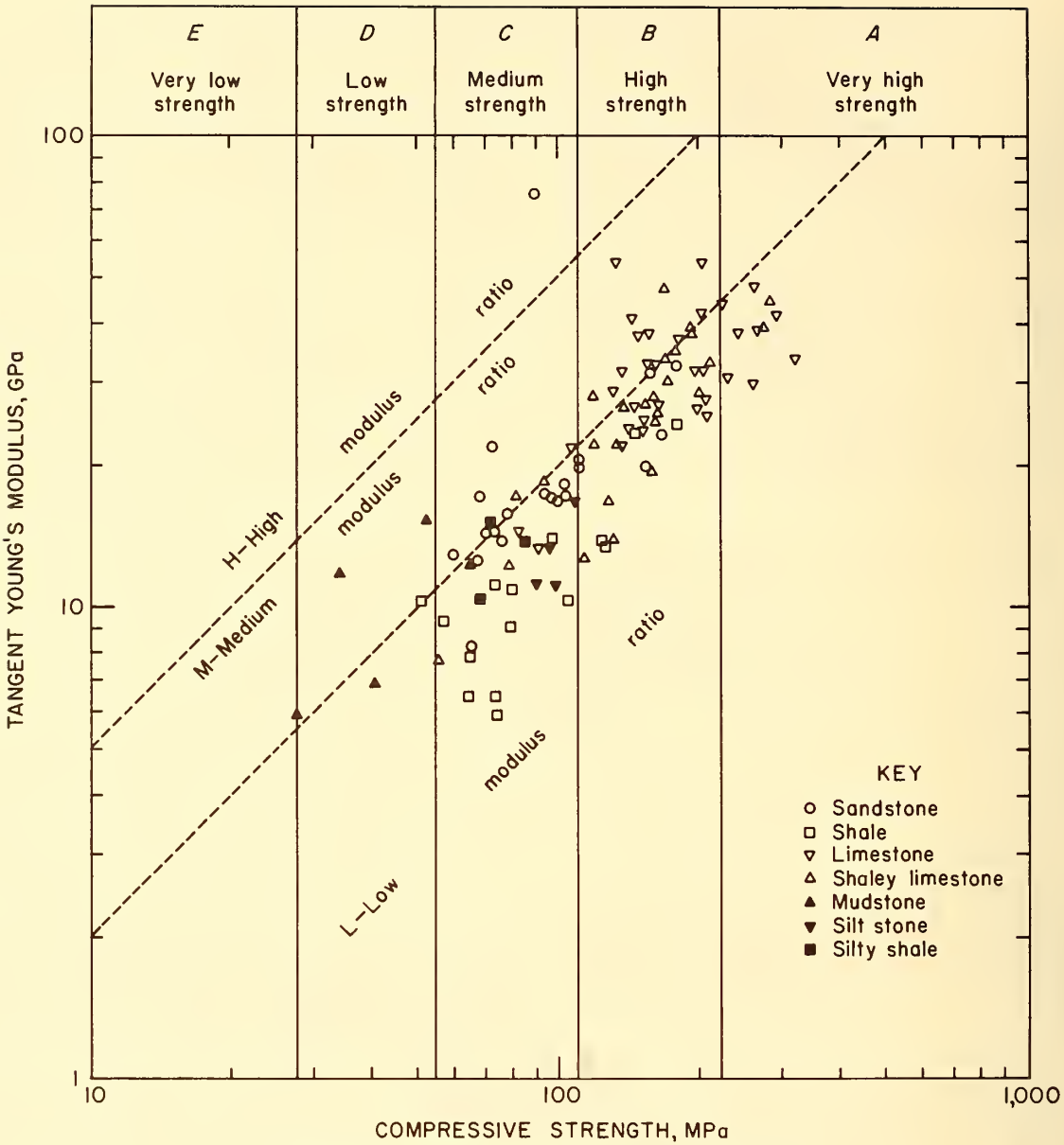

FIGURE 22. - Intact-rock engineering classification.

Figure 22 shows that for the coal measures at the Gateway Mine, the limestones and shaley limestones are classified in the high- to very-high-strength, and medium- to low-modulus-ratio categories. Siltstone and silty shales are predominantly medium strength with low modulus ratio, and mudstone is low strength with medium to low modulus ratio. Such classification schemes for intact rock are relevant to drilling, blasting, and fragmentation on a smaller scale, and for massive rock without joints. These schemes also aid in selection of appropriate mining excavation and fragmentation equipment.

ROCK MASS

Rock mass classification schemes take into account the influence of discontinuities and often, directly or indirectly, in situ environmental factors such as stress and moisture for estimating strength and deformational behavior of rock masses. The geomechanics classification proposed by Bieniawski (6) appears adaptable to coal mining applications, and has been used to a limited extent in classifying roof conditions. The classification is based on uniaxial compressive strength, RQD, the spacing, orientation and condition of joints, ground water conditions, and sometimes other modifying parameters. Significant parameters in the geomechanics classification for determining roof conditions were estimated for 50 ft of strata overlying the Pittsburgh Coalbed. Roof strata at the Gateway Mine were divided into three distinct lithologic units, and the rock mass rating was applied to each lithologic member (table 3). The lowermost member was classified as poor rock, the middle member as good rock, and the uppermost member as fair rock. Such determinations permit speculation on maximum unsupported roof span and standup time and assist in selection of appropriate support. For application in coal measure roof rocks, however, modification and improvement of the classification scheme is needed to obtain a higher degree of predictability of standup time and support requirements.

TABLE 3. - Rock mass classification of Gateway roof rock

Lithologic member and thickness...ft..	I, 10.8	II, 24.7	III, 19.8
Strength of intact rock:			
Uniaxial compressive strength..MPa..	73.....	106.....	81.
Rating.....	(7).....	(12).....	(7).
Point-load index[1].....MPa..	0.08.....	0.37-0.66.....	0.09.
RQD.....pct..	49.....	98.....	97.
Rating.....	(8).....	(20).....	(20).
Spacing of discontinuities.....m..	0.1-0.5.....	0.5.....	0.45-0.48.
Rating.....	(10).....	(20).....	(20).
Condition of discontinuities.....	Slickensided	Slightly rough, separation <1 mm.	Slightly rough, separation 1-5 mm.
Rating.....	(6).....	(12).....	(6).
Ground water general conditions.....	Moist.....	Moist.....	Moist.
Rating.....	(7).....	(7).....	(7).
Total rating.....	38.....	71.....	60.
Class No.....	IV.....	II.....	III.
Description.....	Poor rock...	Good rock.....	Fair rock.
Potential modifier: slake durability..... pct..	75.5 (siltstone). 91.1 (shale)	99.3.....	87.

[1]No rating is given because uniaxial compressive stength is preferred in the low range.

ROCK PROPERTIES DATA BASE

Although the rock properties determined in these investigations will be published, there is need for the establishment of a computerized data base where rock property data can be obtained through search and retrieval operations from remote terminal. The Bureau is working toward this goal by inputting the property data into computerized files. The numerical data base management system, in addition, needs to be able to sort and retrieve coded numerical information so that the data can be manipulated and various mathematical and statistical analyses performed. As a start in this direction, property data tables from Bureau task files were organized into a data base with standardized format for a wide variety of rock types (table 4). These property tables and a detailed description of the data base structure are being compiled into a Bureau Information Circular.

TABLE 4. - Mechanical property tables for mine rock

ID	Rock type and modifier	Location and description	Source	Young's modulus, GPa	Poisson's ratio	Compressive strength, MPa	Tensile strength, MPa	Brazilian line-load strength, MPa
1103	Shale, calcareous kerogen.	CO.....	TCRC	5.9 (N=20)		85.0 (N=20)	7.6 (N=32)	
1104	...do.....	Garfield Co., CO.	TCRC	7.0 (N=20)	0.358 (N=20)	79.6 (N=20)		
1105	...do.....	...do.....	TCRC	3.4 (N=2)	0.370 (N=2)	62.0 (N=2)		
1106	...do.....	...do.....	TCRC	8.0 (N=56)	0.183 (N=55)	90.1 (N=57)	13.2 (N=54)	
1107	Shale.....	PA.....	TCRC	16.1 (N=22)		74.4 (N=22)		6.4 (N=17)
1108	...do.....	PA.....	TCRC	13.7 (N=24)		75.0 (N=24)		6.1 (N=24)
1109	...do.....	Rice Co., KS.	TCRC	15.2 (N=7)		72.5 (N=7)		
1110	...do.....	...do.....	TCRC	21.0 (N=5)		80.5 (N=5)		

N Number of samples tested.
TCRC Twin Cities Research Center.

MINING APPLICATIONS

The physical and mechanical properties of coal measure and other mine rocks have application in most aspects of premine planning and mine design. Elastic and strength properties are especially needed in evaluating and modeling rock mass behavior and structural stability in mines. Geologic data from both surface and underground surveys are required to determine the continuity of coal seams and ore reserves, identify lithologic changes and trends, and delineate geological hazards in the proximity of mine workings. Geophysical properties are necessary for interpreting rock mass and coal or ore characteristics in advance of mining and in inaccessible zones between exploration boreholes or adjacent to mine workings. Acoustic and electrical property data are particularly beneficial in the design of geophysical probes that identify and delineate rock mass conditions during the exploration phase of mining or in advance of the working face during mine development. Index properties are needed to infer other useful engineering properties, when formalized testing procedures and facilities are unavailable or too complicated and costly for a mining operation to utilize on a cost-effective basis. Index properties can be particularly advantageous in coping with the frequent and common lithologic changes prevalent in coal measure rocks. Moreover, the index properties determined in exploration or in-mine geotechnical programs can be utilized in engineering classification schemes that infer rock mass response, indicate stability and support requirements, and permit experience gained at one site to be transferred to other sites where similar conditions exist.

Eventually, an interactive, computerized data base of engineering properties of rock can be established as an information retrieval system for use by mine operators, planners, and research or regulatory agencies. Such a comprehensive engineering properties data base will require input from numerous sources and full cooperation of the industry.

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PILLAR DESIGN EQUATIONS FOR COAL EXTRACTION

By Clarence O. Babcock[1]

ABSTRACT

Coal mine pillar design equations developed during the period 1833 to 1980 are reviewed. These equations suggest that mine pillars of different sizes are required for safety. Two widely used design equations, those of Wilson and Wardell, give pillar areas that vary by an average factor of 2.08. The pillar width and height alone are not the primary parameters in the design problem. The Mohn-Coulomb stress-failure criteria can be used to explain the difference in the estimated pillar sizes.

INTRODUCTION

Coulomb (1)[2] was the first person to publish a rational theory of earth pressures (1773), and this theory is in widespread use today for both soil and rock mechanics applications. His theory was also the first to show that the strength of a solid is related in part to the material properties and in part to the amount of constraint provided during the testing. Since that time, nearly every theory of failure of solids has been in terms of the combined stress, strain, or strain energy state. The role of constraint in strength has only recently been emphasized in the design of mine pillars. Too often, the investigator has failed to realize that the constraint and not the material strength is responsible for pillar behavior when combined states of stress or strain are involved.

STATE OF THE ART IN PILLAR DESIGN

Wilson (2) gave the equation for pillar size, $\overline{W}$, in feet, for a safety factor (S.F.) of 1.0 as

$$W = (R/3 + 2HD \times 10^{-3}) + [R/3 + 2HD \times 10^{-3})^2 + (R^2/3 - 4H^2D^2 \times 10^{-6})]^{1/2}, \quad (1)$$

where R, H, and D are the entry width, entry height, and the depth of the coal seam below the surface, respectively, all in feet.

Wardell (3) proposed an equation of the form

$$S = a/\sqrt{H} + b\,(W/H)^2 = 1{,}000 / \sqrt{H} + 20\,(W/H)^2 \quad (2)$$

for the strength of actual mine pillars. Here the variable S is the strength of the pillar in pounds per square inch; a and b are coefficients; and W and H are the width and height of the pillar in feet, respectively. Wardell gives tables of minimum pillar widths for coal seams (pillar heights) of 4, 5, 6, 7, 8, 9, 10, and 12 ft, for a safety factor of 1.5. The pillar sizes are determined from a tributary area relationship that he does not define mathematically. A relationship that generates those tabulated values was derived by Babcock and Hooker (4) as

$$\frac{(W + R)^2 \times 1.5\,D}{W^2} = \frac{1{,}000}{\sqrt{H}} + \frac{20\,(W^2)}{H^2}, \quad (3)$$

where D is the depth below surface, R is the room width, and W and H are the width and height of the coal pillar, all in feet.

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[1]Mining engineer, Denver Research Center, Bureau of Mines, Denver, CO.

[2]Underlined numbers in parentheses refer to items in the list of references at the end of this paper.

Panek (5) proposed an equation based on a theory of similitude for the design of mine pillars in which the physical properties of the roof, floor, and coal are obtained from laboratory compressive testing of model pillars of the same geometry as the mine pillars with steel platens. His equation is

$$\frac{S_{predicted}}{S_{known}} = \left(\frac{E_r}{E_s}\right)^{C_4} \left(\frac{E_f}{E_s}\right)^{C_5} \left(\frac{u_{cr}}{u_{cs}}\right)^{C_6} \left(\frac{u_{cf}}{u_{cs}}\right)^{C_7} \left(\frac{\nu_s}{\nu_r}\right)^{C_8} \left(\frac{\nu_s}{\nu_f}\right)^{C_9}, \qquad (4)$$

where $S_{predicted}$ is the expected pillar strength, S_{known} is the measured model strength, and E is the Young's modulus, all in pounds per square inch; u is the coefficient of friction; and ν is the Poisson's ratio. The subscripts c, f, r, and s denote the coal, floor, roof, and steel, respectively. The double subscripts cf, cr, and cs denote the coefficient of friction between the coal and floor, coal and roof, and coal and steel, respectively. The coefficients C_4, C_5, ... C_9 are constants to be determined by best fit statistical methods. This relationship assumes no bonding between the coal and roof and floor rock. A more complicated relationship with three additional terms associated with C_1, C_2, and C_3 is given by Panek (5) for the case when the model has a different geometric shape than the mine pillar has.

COMPARISON OF LABORATORY AND IN SITU SPECIMEN TESTING WITH OBSERVED MINE PILLAR BEHAVIOR

In general, the pillar failure prediction equations are of the form

$$\frac{\sigma_p}{\sigma_c} = A + B\,\frac{W^{\alpha}}{H^{\beta}}, \qquad (5)$$

where σ_p and σ_c are the stresses at failure in the pillar and the specimen; A, B, α, and β are constants; and W and H are width and height of the pillar, respectively. Numerous equations of this form and the results given in the literature were summarized by Babcock, Morgan, and Haramy (6) and are shown in table 1 and figure 1. In figure 1, the zone between the failed and unfailed pillars was given by Wardell (3) for S.F. = 1.0. An empirical curve for S.F. = 1.5 is also shown. The theoretical result from reference 2 (table 1) is so marked.

It is apparent that the equations in the literature predicted as safe many pillars that were unsafe. While these equations may be correct for a given coal, none of them, other than Wardell's, predicted correctly the large-scale or full-sized mine pillar behavior.

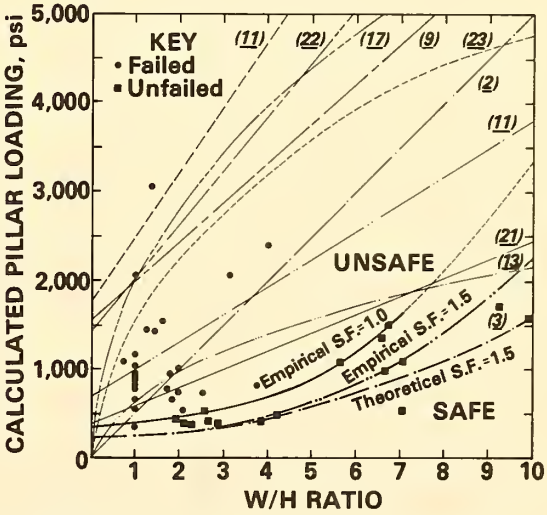


FIGURE 1. - Comparison of pillar strengths predicted by published equations and experimental strengths of mine pillars and large test specimens. Source: Reference 6. (Underlined numbers in parentheses refer to items in the list of references at the end of this paper.)

TABLE 1. - Pillar design equations of the form $\sigma_p/\sigma_c = A + B\ (W^{\alpha}/H^{\beta})$

Year	Investigator	Constants				Type of test	Seam tested	Country	Reference
		A	B	α	B				
1833	Vicat.....	B	1	2	1	S	Limestone.....	Not known.....	7
1876	Bauschinger.	a	1	1	1	S	Sandstone.....	Switzerland.....	8
1897	Johnson.....	0.778	.222	1	1	S	...do.....	United States.....	9
1901	Carpenter...	0	K	0	.5	S	Anthracite.....	...do.....	10
1911	Bunting.....	1,750	750	1	1	S	...do.....	...do.....	11
		700	300	1	1	S	...do.....	...do.....	11
1912	Griffith....	0	C	0	.5	L	Pittsburgh.....	...do.....	12
1939	Greenwald...	0	700	.5	.5	L	...do.....	...do.....	13
1941	...do.....	0	2,800	.5	.85	L	...do.....	...do.....	14
1954	Steart[1].....	0	1.04	1	1	S	Natal.....	South Africa.....	15
1956	Gaddy.....	0	C	.5	0	S	Beckley and Pittsburgh	United States.....	16
1964	Holland.....	0	C	.5	.5	S	...do.....	...do.....	17
1966	Evans.....	0	C	-.32	0	S	Deep Duffryn.....	United Kingdom.....	18
		0	C	-.17	0	S	Barnsley Hards.....	...do.....	18
1967	Salamon.....	0	1,322	.46	.66	L	Bituminous.....	South Africa.....	19
1968	Bieniawski..	0	1,100	.16	.55	L	Witbank.....	...do.....	20
1969	...do.....	400	220	1	1	L	...do.....	...do.....	21
1972	Wilson.....	0	488	1	1	T	NAp.....	United Kingdom.....	2
1973	Van Heerden.	10	4.2	1	1	L	New Largo.....	South Africa.....	22
1974	Wagner.....	1,000	580	1	1	L	Usta.....	...do.....	23
		0	1,600	.5	.5	L	...do.....	...do.....	23
1975	Hustrulid...	0	C	1	.5	S	Existing data.....	United States.....	24
1976	Wardell.....	$\frac{1,000}{\sqrt{H}}$	20	2	2	L	Newcastle.....	United Kingdom.....	3
1977	Skelly[1].....	.78	.22	1	1	L	Pocahontas.....	United States.....	25

C Constant is different for each seam.
L Large test sample.
NAp Not applicable.
S Small test sample.
T Theory.
[1]σ_c as curbical strength, while all others used $\sigma_c = 1$.

Source: Reference 6.

COMPARISON OF PILLAR SIZES DETERMINED WITH THE EQUATIONS OF WARDELL AND WILSON

The equations of Wardell (2) and Wilson (1) were used to calculate the minimum pillar sizes required, in feet, for seam thicknesses of 6 and 10 ft, entry widths of 16, 20, and 24 ft, and depths of 200 to 3,000 ft (table 2). The average pillar stresses corresponding to the pillar sizes in table 2 are given in table 3. Note that the stresses increase with depth. These results can all be represented by straight line relationships as indicated by the parameters given in table 4. In this table, σ, m, and r are the unconfined or uniaxial strength at the surface, the slope of the stress (ordinate) versus depth (abscissa), and the correlation coefficient. If these results were a perfect fit to a straight line, r would have a value of 1.0. The values of 0.992 or better indicate that a straight line assumption is a reasonable one. Notice that by changing the intercept σ and the slope, entry widths from 16 to 24 ft can be fitted. All these results could therefore be described by a Mohr-Coulomb model with reasonable success.

TABLE 2. - Comparison of pillar widths, in feet, calculated with the equations of Wardell and Wilson

Depth, ft	16-ft room width		20-ft room width		24-ft room width	
	Wardell	Wilson	Wardell	Wilson	Wardell	Wilson
6-ft-THICK COAL SEAM						
200	21	20	23	24	26	28
400	31	23	33	27	36	31
600	39	27	41	31	44	35
800	45	30	47	34	50	38
1,000	50	33	53	37	55	42
1,200	54	36	57	41	60	45
1,400	59	40	61	44	64	48
1,600	62	43	65	47	68	51
1,800	66	46	69	50	72	55
2,000	69	49	72	53	75	58
2,500	75	57	80	61	83	66
3,000	84	64	87	69	90	73
10-ft-THICK COAL SEAM						
200	29	22	33	26	36	30
400	46	28	49	32	52	36
600	58	33	61	37	64	42
800	68	39	71	43	74	47
1,000	76	44	80	48	83	52
1,200	84	49	87	53	90	58
1,400	91	54	94	59	97	63
1,600	97	59	100	64	103	68
1,800	103	64	106	69	109	73
2,000	108	69	112	74	115	78
2,500	121	81	124	86	127	91
3,000	132	93	135	98	139	103

TABLE 3. - Comparison of average pillar stress, in pounds per square inch, calculated for the equations of Wardell and Wilson

Depth, ft	16-ft room width		20-ft room width		24-ft room width	
	Wardell	Wilson	Wardell	Wilson	Wardell	Wilson
6-ft-THICK COAL SEAM						
200	745	778	839	807	888	705
400	1,103	1,380	1,238	1,454	1,333	1,299
600	1,432	1,826	1,594	1,949	1,720	1,778
800	1,764	2,257	1,951	2,422	2,103	2,236
1,000	2,091	2,646	2,277	2,848	2,476	2,615
1,200	2,420	3,004	2,628	3,188	2,822	3,004
1,400	2,715	3,293	2,962	3,554	3,176	3,372
1,600	3,039	3,615	3,283	3,902	3,514	3,721
1,800	3,334	3,924	3,594	4,234	3,840	4,016
2,000	3,642	4,223	3,919	4,553	4,182	4,341
2,500	4,417	4,921	4,688	5,290	4,986	5,094
3,000	5,102	5,625	5,445	5,989	5,776	5,843
10-ft-THICK COAL SEAM						
200	578	716	619	751	667	778
400	872	1,185	952	1,268	1,025	1,333
600	1,172	1,587	1,270	1,709	1,361	1,778
800	1,465	1,909	1,577	2,061	1,684	2,191
1,000	1,758	2,231	1,875	2,408	1,994	2,563
1,200	2,041	2,534	2,178	2,732	2,310	2,878
1,400	2,323	2,823	2,471	3,012	2,614	3,204
1,600	2,606	3,103	2,765	3,308	2,919	3,514
1,800	2,883	3,375	3,052	3,594	3,216	3,814
2,000	3,164	3,642	3,334	3,873	3,506	4,104
2,500	3,846	4,302	4,046	4,558	4,241	4,791
3,000	4,526	4,945	4,746	5,219	4,950	5,473

TABLE 4. - Parameters for straight line fit to table 3 data

Parameter	16-ft room width		20-ft room width		24-ft room width	
	Wardell	Wilson	Wardell	Wilson	Wardell	Wilson
6-ft-THICK COAL SEAM						
σ	505	812	615	862	684	699
m	1.560	1.682	1.639	1.809	1.734	1.796
r	.9995	.9931	.9993	.9920	.9989	.9940
10-ft-THICK COAL SEAM						
σ	329	667	387	739	445	786
m	1.411	1.476	1.469	1.553	1.524	1.636
r	.9999	.9963	.9997	.9950	.9995	.9939

The results given in tables 5 and 6 show the need for horizontal confinement for pillar strength. The value d* is defined as $d^* = d/(1-r_e)$, where d is the depth below surface and r_e is the fractional recovery by mining (i.e., 50 pct = 0.50). In table 5, the cohesive strength in unconfined shear is 100 psi. The small white area at the top of the table indicates that only small depths can be mined without some horizontal pillar confinement. For angles of internal friction of 30° or larger, a horizontal stress of less than 30 pct of the vertical is required for pillar stability away from the pillar edges. For large angles of friction, for instance, 50° or more, the horizontal stress required even for low 'a' values is only 11 pct or less of the vertical stress. This means that practically no horizontal confinement is required, and most of the confinement comes from the vertical stress loading across the failure surface.

TABLE 5. - Ratio of horizontal pillar stress to vertical pillar stress needed for pillar stability for a small value of cohesive strength; 'a' = 100 psi

d*, ft	Friction angle (ϕ), deg 0	10	20	30	40	50	60
200	0.17						$\frac{\sigma h}{\sigma v} = 0$
400	.58	0.36	0.20	0.09	0.03		
600	.72	.47	.30	.17	.09	0.03	
800	.79	.53	.34	.21	.12	.06	0.02
1,000	.83	.56	.37	.24	.14	.07	.03
1,200	.86	.59	.39	.25	.15	.08	.03
1,400	.88	.60	.41	.26	.16	.09	.04
1,600	.90	.62	.42	.27	.17	.10	.04
1,800	.91	.63	.43	.28	.18	.10	.05
2,000	.92	.64	.43	.29	.18	.10	.05
2,500	.93	.65	.44	.30	.19	.11	.05
3,000	.94	.66	.45	.30	.19	.11	.06
∞	1.0	.70	.49	.33	.22	.13	.07

TABLE 6. - Ratio of horizontal pillar stress to vertical pillar stress needed for pillar stability for a large value of cohesive strength; 'a' = 800 psi

d*, ft	Friction angle (ϕ), deg 0	10	20	30	40	50	60
200							
400			$\frac{\sigma h}{\sigma v} = 0$				
600							
800							
1,000							
1,200							
1,400	0.05						
1,600	.17	0.01					
1,800	.26	.08					
2,000	.33	.15	0.02				
2,500	.47	.26	.12	0.3			
3,000	.56	.33	.18	.08	0.01		
∞	.00	.70	.49	.33	.22	0.13	0.07

Next consider table 6 for a cohesive strength in shear of 800 psi. While this appears large, cohesion values up to 1,240 psi for coal have been observed in the laboratory. Notice the large blank areas indicating that no horizontal confinement is necessary. For angles of internal friction as small as 30°, a horizontal stress that is only 8 pct of the vertical is required for pillar stability, away from the pillar edge where the opening stress concentration exists. For example, if a recovery of 50 pct is used, the d* value would be 3,000 for a depth of 1,500 ft. The ∞ symbol at the bottom of tables 5 and 6 indicates the limits of horizontal constraint required at great depth assuming elastic behavior still applies.

THE ROLE OF CONSTRAINT IN PILLAR DESIGN

WHAT IS FAILURE?

What passes for coal or rock strength is often not material strength at all but constraint behavior. Consider the behavior of the coalbed in figure 2 at some depth D below the surface. Regardless of the coal strength, the coalbed will remain intact as long as it is confined by the rock strata above and below it. This will be true for any depth and any material. In other words, when completely confined, any coal or rock appears to have an 'infinite' strength. If a piece of this coal or rock could be tested in the laboratory, it would be readily apparent that the constraint and not the coal or rock strength was responsible for this behavior.

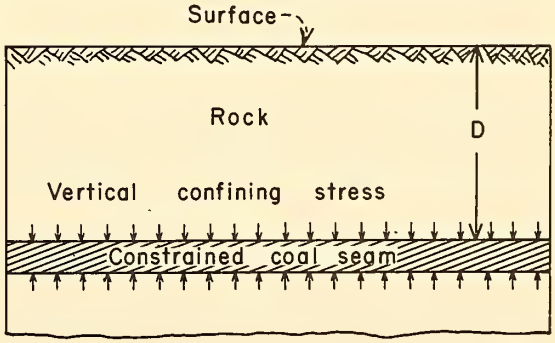


FIGURE 2. - Confined coalbed. Any coal seam of any strength at any depth constrained as shown is "infinitely" strong for a uniform vertical stress σ_y.

Another way of showing the effect of constraint is to consider a single entry in the coal layer as shown in figure 3. If D is large enough and the coal in the rib is unconfined, it will break, adjacent to the opening, for a distance that is some function of the coal seam height to point B. This broken zone will progressively provide more constraint away from the rib until the combination of coal strength and constraint halts the breaking process. The constrained coal at A is unbroken.

Next, consider several entries as shown in figure 4. Each entry will have two unconfined ribs in coal and an unconfined roof and floor. Because of less constraint, a relatively greater volume of coal will break until the combined effects of constraint and strength halt the breaking process. As the pillars become smaller, constraint plays a decreasing role in pillar stability and the coal strength without constraint a more important role.

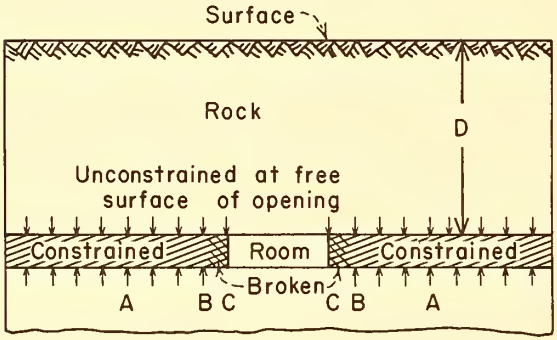


FIGURE 3. - Single entry in coal layer. Constrained coal seam is "infinitely" strong except at the free surfaces and adjacent to them.

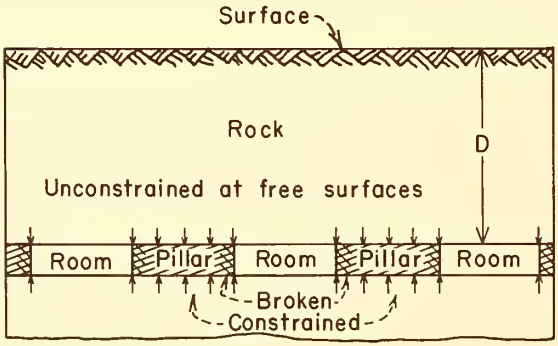


FIGURE 4. - Several entries in coal layer. As the number of free surfaces increases, the overall constraint decreases, and the overall coal strength decreases as well.

The many attempts to define pillar strength in terms of the width-to-height ratio (W/H) for the pillar imply that constraint is taken as necessary to ensure pillar survival. If the coal alone is strong enough to support the applied load, constraint will be unnecessary and the value of W/H does not enter the problem. What the W/H ratio represents is the amount and importance of the constraint to pillar survival. This is given a physical meaning if expressed in Mohr-Coulomb behavior.

A common model for failure prediction is the Coulomb (1) behavior of soil mechanics or the Mohr-Coulomb behavior of the theory of elasticity. Figure 5 shows the case of Coulomb failure in soil mechanics. The change in shearing strength with normal stress across the shear surface results from frictional effects, defined by the angle of internal friction ϕ. That is, the shearing strength is the cohesive strength C in shear plus the frictional strength, $\sigma_n \tan \phi$, that results from confinement of the shear surface by the normal stress σ_n. If the normal stress is removed at any time, the cohesive strength in shear

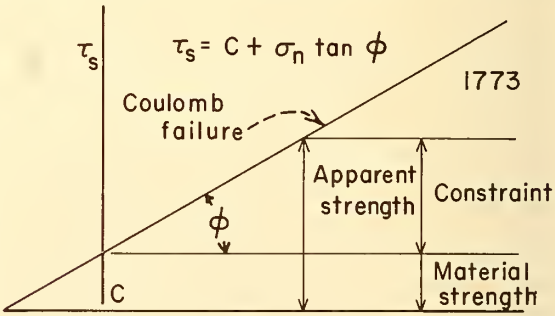
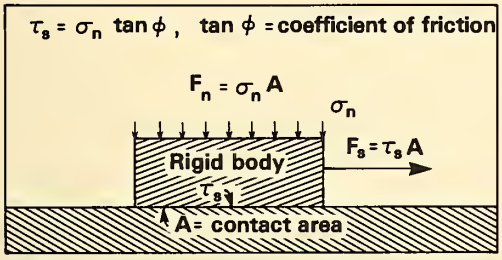


FIGURE 5. - Coulomb failure. Apparent strength as defined by the Coulomb σ_n failure condition is partly material strength and partly the effects of constraint across the failure surface in shear.

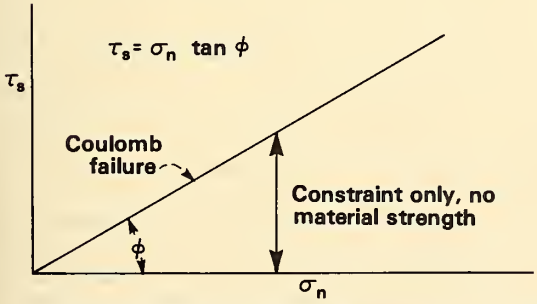
is still C. In other words, the apparent increase in strength is only the effect of one stress, the shearing stress, acting against the confining stress.

Another way of showing that the increased strength is not strength at all is shown in figure 6, which shows the frictional effect alone of one rigid solid surface being pressed against a second rigid solid surface. The resulting curve is the same as that for a Coulomb failure when the cohesive strength is zero. That is, the total apparent strength is only the result of constraint. Since the bodies are rigid they do not deform, and material strength is not considered.

Next, consider the Mohr-Coulomb stress-failure relationship shown in figure 7. It is assumed in this theory that the



A **Shearing stresses produced when two rigid bodies are displaced parallel to the contact area.**



B **The Coulomb relationship between the normal and shearing stresses of *A*.**

FIGURE 6. - Shearing stresses and Coulomb failure. The Coulomb failure relationship describes the effect of a shearing stress acting against a normal stress across the shear zone.

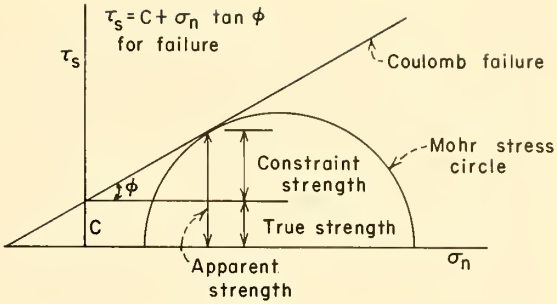


FIGURE 7. - Mohr-Coulomb stress-failure relationship. This is of the same form as the relationship shown in figure 6 but with a cohesive shear strength added.

Mohr's stress circle will produce failure if it touches the Coulomb failure line in any way. The Coulomb condition is a straight line as shown or a concave downward line when defined by the Mohr's stress envelope, with increasing displacement of the circle to the right on the normal stress axis. The strength of a solid is generally taken to be the value of the shear stress at the point the circle touches the Coulomb surface. However, in view of the fact that this is part shear strength and part confinement, this is an incorrect interpretation of pillar behavior. Since the solid can be broken with a shear stress C at any time when unconstrained, this much of the shear value is material strength and the remainder is constraint strength. The notion that the combined state of stress somehow measures strength has lead to confusion in the evaluation of pillar behavior, particularly with regard to laboratory testing of rock and coal samples, where unusually high constraint strength is provided by very stiff steel platens or triaxial confinement.

The stress states at points A, B, and C in a coal seam as shown in figure 3 are defined in figure 8. If the coal has a shear strength C_2, the Mohr's stress circles A, B, and C will not reach the failure condition, the rib will stand intact, and no constraint will be necessary for pillar stability. If the strength is C_1, the coal will break at the rib until the broken coal provides enough constraint or

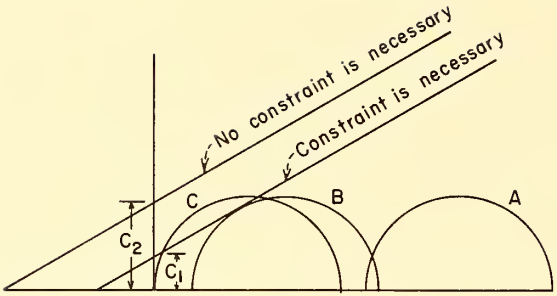


FIGURE 8. - The Mohr's stress circles for points A, B, and C shown in figure 3. The coal with shear strength C_1 has failed at the rib, and constraint is necessary to pillar survival. The coal with shear strength C_2 does not fail at the rib, and constraint is unnecessary to pillar survival.

the stress concentration disappears so that the Mohr's stress circle B will just touch the Coulomb failure condition. Constraint will be necessary for pillar survival.

DOES A LARGE W/H RATIO ALWAYS ENSURE PILLAR STRENGTH?

At least one implicit assumption inherent in the use of the W/H ratio as a measure of pillar stability is that this ratio provides constraint to most of the pillar volume away from the coal rib. In part, this belief is the result of testing flat rock or coal samples in the laboratory between steel platens that provide frictional constraint to the sample, making it almost incompressible.

Consider the relationships shown in figure 9 where steel cubes load a rock or coal cube. For 30 psi of vertical stress in the steel, a vertical strain of 1 μin/in results. If the Poisson's ratio is 0.25, the horizontal strain is 0.25 μin/in. This is an upper bound for the horizontal strain because the steel block is not constrained horizontally as would be the case for a steel platen that is much larger than the specimen. The cube of coal with a Young's modulus of

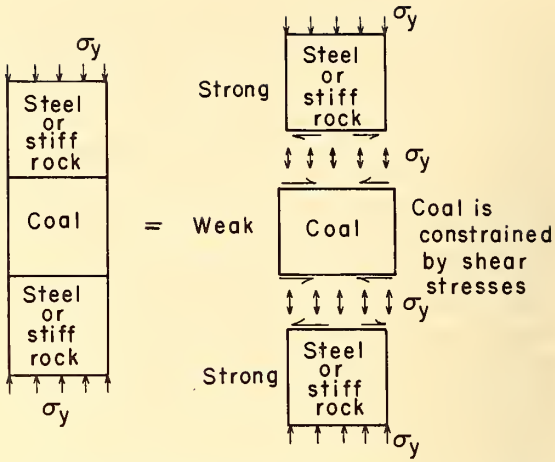


FIGURE 9. - Steel cubes loading a coal cube. If the coal is confined between two more rigid materials such as steel platens or strong roof and floor rock, the apparent strength of the coal increases from constraint.

1×10^6 psi or less would have a vertical strain of 30 μin/in and a horizontal strain of 7.5 μin/in for a Poisson's ratio of 0.25. The strains in the coal would be 30 times those in the steel. The net effect is that the steel constrains the horizontal expansion of the coal and gives it an apparent strength that is much greater than the strength of the coal when unconfined.

Next consider the relationships shown in figure 10 where cubes of rock with low Young's moduli, for example, 0.1×10^6 psi, are used to load the cube of coal with Young's modulus of 1×10^6 psi or less. A vertical stress of 30 psi will produce a vertical strain of 300 μin/in in the rock cubes. If the soft roof and floor are a claylike material, the Poisson's ratio may be very high, approaching 0.5. To be conservative, let the Poisson's ratio be 0.25. This would result in a horizontal strain in the roof and floor of 75 μin/in. This is an upper bound because the roof and floor are constrained laterally in the mine. For this

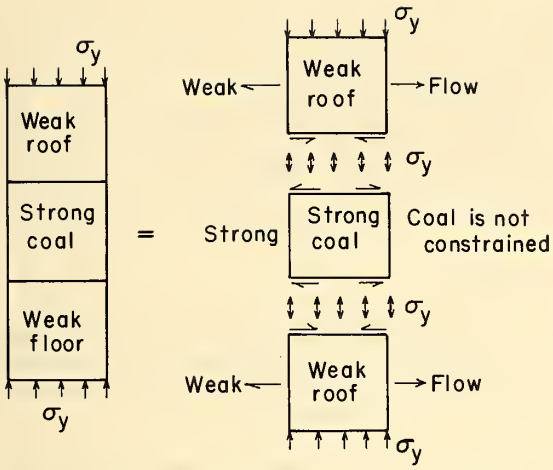


FIGURE 10. - Strong coal between weaker materials. If the coal is stronger than the roof and floor rock or the material used to load the coal in the laboratory, the coal is not constrained. In the mine the roof and floor must flow for the constraint to disappear.

horizontal strain to occur, the floor must heave and the roof must sag. These conditions are not uncommon in actual mining operations. The vertical strain in the coal would be 30 μin/in. The horizontal strain in the coal for a Poisson's ratio of 0.25 would be 7.5 μin/in. The horizontal strains in the roof and floor would be 10 times that for the coal. The net effect is that, instead of confining the coal seam, the roof and floor would actually pull the coal seam apart. In this case the W/H ratio does not indicate increased strength.

Future work on the effects of W/H ratio on pillar strength should evaluate these two important factors in terms of Mohr-Coulomb behavior. First, the strength of the coal itself, when unconfined, must be determined; and second, the role of constraint or lack of it must be evaluated. One way of doing this, in situ, would be to use the drilling yield method developed in Europe.

POTENTIAL FOR USING THE DRILLING YIELD METHOD FOR DESIGNING MINE PILLARS

Kidybinski and Stranz (26) reported on the drilling yield method to the U.S. Department of the Interior. In this method a hole drilled into the coal seam is used to study the conditions of stress in the seam. The volume of drill cuttings is expected to be 2 to 3 L/m of drill-hole length when the hole diameter is 42 mm. If the hole squeezes shut during drilling, indicating that the coal is not strong enough to handle the stress, the volume of cuttings will increase. If the volume is 6 L or more, a serious coal bump condition exists. The method has been in field use since the 1960's (27) in the Federal Republic of Germany, since the 1950's in the U.S.S.R., since 1965 in Poland, and since 1968 in Yugoslavia. A 2.5-hp motor was used, and the stalling behavior of the motor was also a part of the analysis.

The use of the drilling yield method could be helpful in the design of mine pillars by establishing the broken or squeezing zone locations or depths. The concept is shown in figure 11. In part A, a point in the coal seam denoted by P is confined and stable. If a hole is drilled through this location (part B), the constraint is removed on the hole surface. The tangential stress

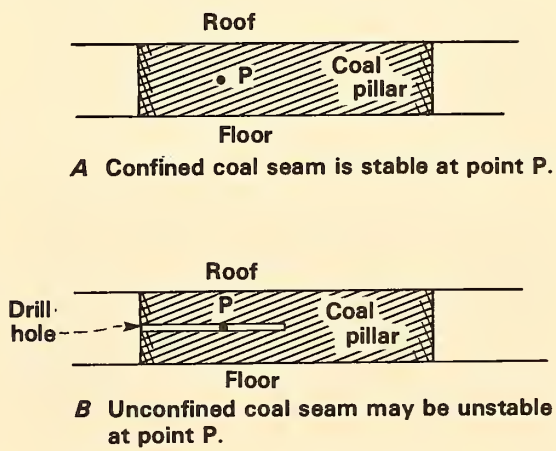


FIGURE 11. - Use of drilling yield method to establish pillar dependence on constraint for survival.

concentration on the hole boundary would be in the range of 2 to 3 times the vertical stress for an elastic condition. If the coal breaks around the hole, it does not mean that the pillar is unstable but that the coal strength is less than 2 to 3 times the most compressive principal stress. With breaking, the stress concentration will be reduced and in the limit approach the coal seam stress. The breaking should then stop. If it does not, this indicates that the coal strength is inadequate to support the stresses when unconfined. The volume of cuttings is therefore an indicator of the pillar stability versus depth from the rib.

SUMMARY AND CONCLUSIONS

The number of studies of pillar behavior based on testing of samples has increased rapidly during the last 25 yr. The concepts on which many, if not most, of these tests were based often date back to 1912 or before. That is, the number of testing procedures or methods of analysis used has not kept pace with the number of studies. Recent experimental testing has emphasized the effects of sample size and conditions of testing, including constraint. One reason for the slow progress in pillar design is that the separate effects of coal strength and the apparent increase in strength resulting from confinement have not been appreciated.

In pillar design, if the pillar must have constraint from the roof and floor to survive, the pillar becomes unstable if such constraint disappears with time or change in mine geometry. If the coal can survive in pillar form without constraint, it is more likely to remain stable when constraint conditions are changed.

The practice of assuming that a flat coal pillar will be "infinitely" strong because this is the case for laboratory testing of coal between steel platens should be examined carefully. In such tests, the strength of the platens and not of the coal is determined, and there is no correlation to mining conditions with roof and floor of rock, sometimes rock that is not very strong or structurally stable.

It is commonly considered that the recent emphasis on constraint is new, but in fact, any equation using W/H relationships implies this condition.

Most of the pillar strength results from constraint across the failure surface by normal compressive stresses according to the Mohr-Coulomb stress-failure criteria. This constraint is supplied for the most part by the vertical component of stress from the overlying rock. This is particularly true if the angle of internal friction is 30° or more. For angles as large as 50° or 60° only a very small horizontal stress component is needed for pillar stability. In addition, the small value of the unconfined shear strength often assumed for in situ behavior is largely responsible for the very different results obtained in the laboratory and in the mine.

Nearly all the theories with width-to-height relationships can be represented equally well using the Mohr-Coulomb stress-failure models. There is no magic in the width-to-height ratios for pillar design. These occur naturally when the strength increases with depth and a given entry size is used.

The cores of pillars are confined for the most part not by the horizontal pillar stresses, as in the constrained core concept, but through the action of the vertical stress away from the pillar edges. The effect that exists with respect to horizontal stress is one of a confined edge rather than a core.

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UNDERHAND CUT-AND-FILL STOPING FOR ROCK BURST CONTROL

By F. Michael Jenkins[1] and K. Robert Dorman[2]

ABSTRACT

The occurrence of rock bursts in deep metal and nonmetal mines presents a major hazard to their safe and economical operation. As part of a Bureau of Mines program to reduce rock bursts in deep vein mining, a project was conducted in the Coeur d'Alene mining district of Idaho to evaluate and demonstrate the mining of a destressed sill pillar using underhand cut-and-fill methods. A 50-ft sill pillar in a burst-prone area was preconditioned by drilling and blasting vertical holes in the ore. After raises were driven through the destressed pillar to the level above, it was mined by the underhand cut-and-fill method. The conclusion from this demonstration is that the combination of ore preconditioning and underhand mining resulted in greatly improved rock burst and ground control, allowing safe and efficient mining of a potentially hazardous, burst-prone pillar.

INTRODUCTION

The occurrence of rock bursts in deep metal and nonmetal mines presents a major hazard to their safe and economical operation. A rock burst is the sudden violent release of strain energy stored in the rock as a result of mining. The causes of rock bursting can be traced to the geometry of mine openings and rock characteristics. Strong, brittle rock in high stressed areas of the mine has a high potential for bursting. Bursting associated with stoping usually occurs in sill pillars, converging stopes, initial or subdrifts, and raises.

As part of a research program to reduce rock bursts in deep vein mining, a project was conducted under a Bureau contract to evaluate and demonstrate the mining of a destressed sill pillar using underhand cut-and-fill methods.[3] The project was carried out in two phases: first, to study the feasibility and cost effectiveness of the method and present design recommendations, and second, to enlist the cooperation of a mine and demonstrate that the method could be used to reduce rock bursts. This report presents some background information from phase I and the findings and conclusions of phase II as demonstrated by the test stopes, including cost, production, and safety studies.

BACKGROUND

In the Coeur d'Alene mining district of northern Idaho, where lead-zinc silver veins are being mined to 8,000 ft below the surface, rock bursting continues to be a severe operational problem. Finding a means of controlling rock bursting has been the goal of research for almost 80 yr. However, the problem still persists because of its complexity and the difficulty of finding practical solutions that can be applied economically.

The predominant mining method in the Coeur d'Alene mining district is overhand

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[3]Bush, D. D., W. Blake, and M. P. Board. Evaluation and Demonstration of Underhand Stoping To Control Rock Bursts. BuMines contract H0292013; for information, contact F. M. Jenkins, TPO, Spokane Research Center.

cut and fill. The general plan is to develop horizontal levels from vertical shafts, produce ore from stopes that are mined upwards toward the mined-out level above, and backfill each stope cut with mill tailings. The result is an ever-decreasing pillar of ore between the level above and the miners working in the stope, with lateral stress concentrated in the pillar. Structural failure of the sill pillar often produces rock bursts.

Experience indicates that sill-pillar bursting usually begins when the pillar height is reduced to about 80 ft, with the greatest frequency and severity occurring at pillar heights of 40 to 30 ft. These bursts often inflict great damage to the stope as well as to haulage drifts and crosscuts above and below. A large burst can displace more than 1,000 tons of rock, heave the drift floor, and break numerous caps and posts. The results are extensive repair costs and loss of production. Statistics indicate that failure associated with sill pillars accounts for more than 60 pct of all rock bursts.[4]

Ore preconditioning, the destress blasting of a pillar prior to mining, has been shown to effectively reduce rock bursts.[5] Blasting changes the characteristics of the ore from a strong, brittle material to a yielding material incapable of storing strain energy. To be effective, however, the preconditioning must thoroughly fragment the ore. This creates a hazard to those working in overhand stopes because they are working beneath fragmented rock. The underhand cut-and-fill mining method was developed in Canada to mine pillars of ore similar to those created by preconditioning.[6] A combination of the two techniques (preconditioning and underhand cut and fill) seemed an attractive solution to recovering burst-prone sill pillars.

PHASE I--EXAMINATION AND DESIGN RECOMMENDATIONS

During phase I of this project, the underhand cut-and-fill practice was examined at two mining operations in Canada and two in the United States where the method was being used for primary production and for pillar recovery under difficult ground conditions. An artist's conception of a typical underhand cut-and-fill stope is shown in figure 1.

Stress analyses were made of mining by overhand cut and fill and by underhand cut and fill to compare their effects on ground control in the Coeur d'Alene District.

To examine the bursting potential of underhand cut-and-fill stoping, the initial simulation assumes five end-to-end stopes mined in a flat-back arrangement toward an unmined level below. This simulation ignores active mining on multiple levels; thus, no sill pillar is created. The maximum stress (30,000 to 50,000 psi) occurs at the stoping horizon. With no sill pillar being created, the stress does not change as mining progresses. Because of the constant rate of energy release, little bursting should be encountered with a flat-back underhand method if no pillars are created.

Because mining is not normally confined to one level, more realistic, multilevel mining sequences were examined. A series of simulations with simultaneous mining on three levels was made. The potential for bursting approaches that of the overhand method soon after multilevel mining occurs. In this case, the pillar stress is between 55,000 and 60,000 psi or nearly that amount induced in the sill pillar

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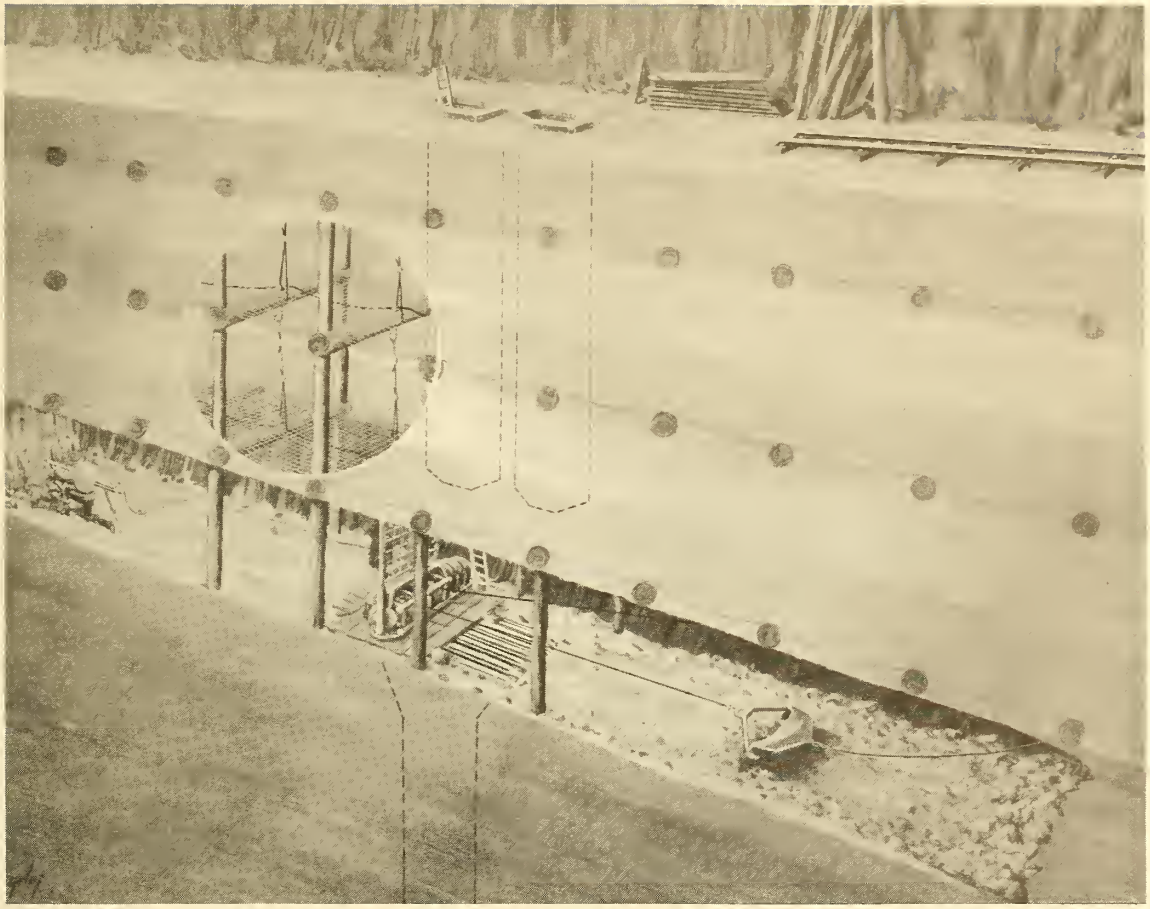

FIGURE 1. - Artist's conception of an underhand cut-and-fill stope.

for the overhand case. Though energy release rates are lower, bursting will not be eliminated by underhand mining alone. However, underhand cut and fill offers distinct advantages where destressing is used in rock burst control: (1) Preconditioning techniques are easily applied since destress holes can be drilled vertically in the ore body, (2) damage during preconditioning will generally be less severe and will not produce the massive caving that often occurs during conventional destress blasting, and (3) blast-induced caving and timber damage during mining are eliminated.

Following the mine examinations and stress analysis studies, three alternate methods of underhand cut-and-fill stoping were suggested for the Coeur d'Alene. A cost-and-production estimate was made for the three methods as well as for typical timbered cut and fill. They indicated that efficiencies (tons per worker-shift) for the underhand cut-and-fill method were comparable to practices in use, and that considerable cost savings could be obtained over conventional timbered cut and fill, owing to the reduction of timber required for support. The cost estimate found that underhand cut and fill would not, however, be an economical alternative for primary production at all mines.

PHASE II--FIELD TEST

The phase I study indicated underhand cut-and-fill stoping could be a cost-effective method of rock burst control when used in conjunction with ore preconditioning. During phase II, the method was tested in an operating mine, and productivity, relative cost, and ground control were evaluated. This section discusses the field test, from site selection through mining of the final cut.

SITE SELECTION AND MINING PLAN

The results of the phase I study were presented to the major mining companies in the Coeur d'Alene District. The response was generally positive, although most of the companies questioned the economics of the system. One company was interested enough to cooperate with the Bureau and Terra Tek, Inc., in a field test. An ideal test area was selected in an unusual manner. Miners were preparing a relatively stable area for the test when a major burst occurred in a 50-ft sill pillar in another part of the mine. Based on past experience, management predicted that additional pillar bursts of possibly greater magnitude were likely to occur during mining of this sill pillar. They chose this site for testing the underhand cut and fill combined with ore preconditioning.

A mining plan, illustrated in figure 2, was prepared by Terra Tek and submitted to the Bureau and the mine for approval. The plan called for the following sequence of events:

1. Complete repair of the rock burst and complete the present stope cut. Three of the stopes in the pillar were to be brought to the same elevation, thus creating a flat back. One stope had been dropped because of poor-grade ore.

2. Clean out and repair the haulage lateral above the pillar. Timbersets were to be repaired or replaced, slabs were to be removed, and track was to be repaired where possible.

3. Prepare the haulage drift for cemented fill after cleanup and repair. A floor mat, consisting of 12- by 12-in stringers, lagging, 4- by 4-in No. 8 wire mesh, and a layer of woven polyethylene cloth (Fabrene),[7] would be laid over the entire 600-ft length of drift. At 100-ft intervals, fill fences would be constructed from light timber, wire mesh, and Fabrene to limit the extent of any individual pour.

4. Drill the pillar with vertical destress holes and blast using ammonium nitrate-fuel oil (ANFO) or water gel.

5. Perform pre- and post-preconditioning seismic velocity surveys through the pillar to evaluate effectiveness of preconditioning.

6. Fill the haulage drift with cemented sandfill. An 8:1 sand-to-cement dry weight ratio was chosen, based on experience in other mining districts.

7. Drive three-cap raises (manway and timber slide, plus two joker chutes) through to the haulage drift. Raise through cemented fill to provide secondary access to the stopes. Provide airtight raise covers to avoid upsetting the ventilation flow.

8. Leave the present overhand mining floor open for access way between stopes.

9. Begin underhand cut-and-fill mining by breasting beneath the haulage-level cemented fill. Post or timber beneath the haulage floor mat if necessary, making the stope as narrow as possible. Leave the raises open for secondary escape as mining progresses downward.

10. Prepare the floor of each cut with caps on 6-ft centers, lagging, wire mesh, and Fabrene. Pour cemented fill with 8:1

[7]Reference to specific products does not imply endorsement by the Bureau of Mines.

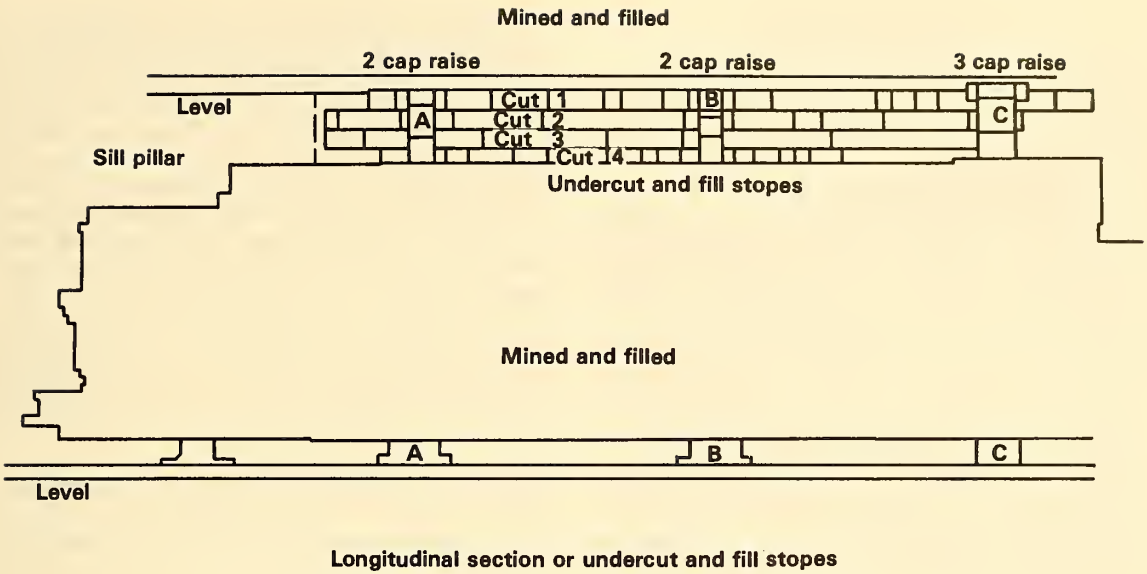

FIGURE 2. - General layout of pillar recovery plan.

sand-to-cement ratio (by weight) to a depth of 3 to 5 ft, followed by uncemented tailings for the remainder of each 10-ft cut.

11. Instrument each cut with closure extensometers and fill pressure cells to monitor load-displacement behavior of the fill.

12. Take the last cut by end slicing.

Preparations for the underhand cut-and-fill experiment began in November 1980 based on the mining plan.

INSTRUMENTATION

The instrumentation program was aimed at quantifying the stress and displacement behavior of the wall rock, ore body, and fill before and after the destress blasting as well as during subsequent mining.

Initial closure and stress change instruments were installed on the upper level during cleanup and repair of the September 1980 rock burst damage. These were to supply baseline data during leveling of the stopes and during the period prior to blasting the destress round. A second group of instruments was installed during preparation of the destress round.

Instrumentation performance was mixed. The most consistent and useful data were obtained from the closure points and stope closure extensometers. This closure data nicely illustrated the effects of preconditioning and pillar behavior before and during mining. The stress meters provided qualitative data on stress conditions in the walls surrounding the test stopes, which could be of possible use in a modeling study. However, there were too few gauges for a quantitative evaluation of the stress conditions in the pillar. The soil pressure cells placed in the fill proved disappointing. The gauges are thought to have suffered from electronic, environmental, and, possibly, instrument-construction problems. Little quantitative data were produced by the pressure cells prior to failure. One point is evident from this test: Hand-measurable instruments, such as the tape extensometer, provide the most reliable data in a harsh mining environment.

SITE PREPARATIONS

Because the mine had no facilities for adding cement to the sandfill (mill tailings), a small cementing system was designed and constructed on-site. The mine's existing sandfill system consisted of two major parts (fig. 3): the surface sand-pumping facilities located within the mill and underground sand storage and distribution system. At the mill, a duplex piston pump was used to pump the sandfill slurry (40 to 50 wt pct solids) through 11,000 ft of 3-in line to the 2000 level of the mine. At the 2000 level, the slurry was discharged to a cyclone where the underflow was approximately 53 to 60 pct pulp density and contained a high percentage of fines. The high percentage of fines bears directly on the ability to achieve a good sand-cement set.

After passing through the cyclone, the underflow was dumped to a storage tank (the storage tank was blasted out of country rock and lined with shotcrete) and air-agitated. The sandfill was gravity-drained from the tank and distributed to the required stope through 3-in black pipe with victualic-style couplings.

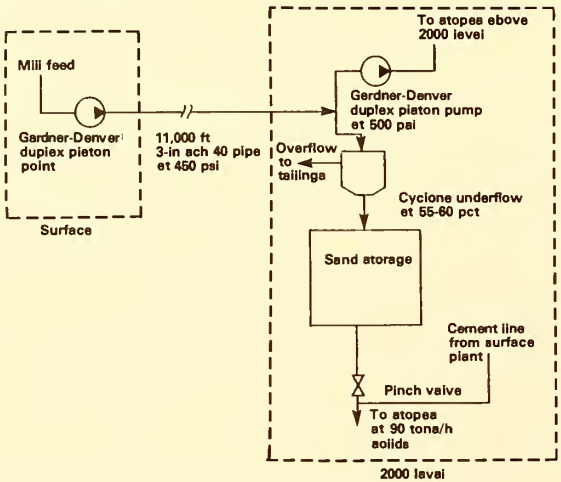

FIGURE 3. - Schematic of existing sandfill system.

A study was conducted to determine the most cost-effective method of introducing cement into the sandfill for distribution to the stopes below the 2000 level. Because of operational problems (tramming and inability to mix cement in the underground storage tank), as well as the cost of handling and preparing large tonnages of cement underground, the decision was made to place the cement slurrying and pumping facilities outside the mine.

The cementing system can be divided into two component parts: cement handling and cement pumping. The components are diagramed in figure 4.

The cement-handling system is that portion of the cementing system that stores bulk cement, delivers it at a desired rate, and slurries it to the desired bulk density. The functions of the handling system are--

1. Bulk storage of up to 60 dry tons of cement.

2. Delivery of from 0 to 5.5 cfm bulk cement for slurrying, with adjustment for any range in between.

3. Capacity for slurrying up to 3,000 gal of a 50-pct-solids cement slurry.

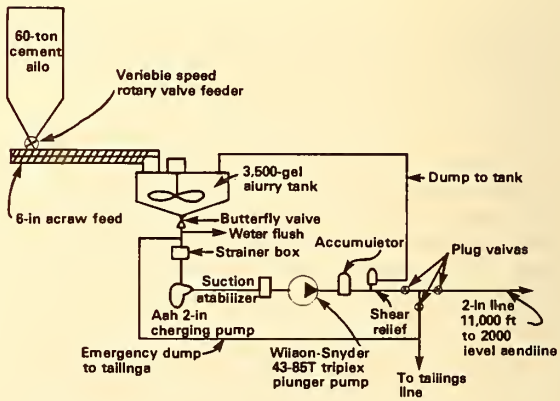

FIGURE 4. - Schematic of cement-handling system.

The pumping system referred to here includes all pumps, valves, pipelines, and accessory equipment for delivering the cement slurry to the 2000 level. The governing functions of the system are--

1. A system capable of pumping at a rate of 30 to 60 gpm at pressures to 600 psi during normal operation.

2. In the event of a need to stop pumping into the sandline for short periods of time (for example, sandline breaks in shaft), the pump should be capable of displacing less than 5 gpm of slurry.

3. The underground line must be capable of being flushed both from outside with fresh water, or from inside the mine in the event of pump failure.

The design of the cementing system was completed in late July of 1980, and orders were initiated for components in August. The initial testing of the system was completed in January 1981. After extensive use of the system, the normal operating conditions for a slurry pulp density of 50 pct were a flow rate of 50 gpm at a discharge pressure of 450 psi.

During construction of the cementing system, preparation of the floormat in the sill drift was completed. When this drift was originally driven, 12- by 12-in stringers were placed at each rib-floor intersection and run parallel to the drift axis for its entire length. The posts for the drift caps were footed on these stringers. The floormat was constructed by placing lagging across the stringers and covering the lagging with a single layer of 4- by 4-in wire mesh (fig. 5) followed by a single layer of Fabrene. As no rail track existed, all timber was hand-trammed into the drift. The rock burst had reduced the drift section to less than 6 ft in height and width in certain areas and eliminated air and water services. Slabs pulled loose from the walls were broken and removed by hand. Consequently, repairs and mat preparation required approximately 1 month.

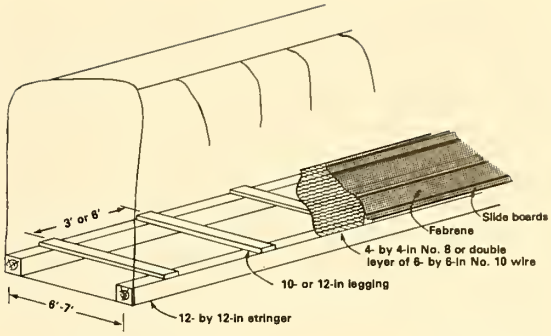

FIGURE 5. - Floormat preparation in overlying drift.

Shortly after completion of the cementing system, the drift was filled in a series of five individual pours, each 100 to 200 ft in length. Owing to poor availability of sandfill, this process required approximately 1 month for completion.

By end of January 1981, stope preparations were completed. The three stopes had been brought to the same elevation and were sandfilled (with the exception of each raise area) to within 4 ft of the back. It was decided to leave this 4-ft access way between stopes to provide ease of movement during mining and simplify preconditioning of the pillar. Each raise area was timbered in preparation for raise driving from the stope below.

The original preconditioning design called for a series of 35-ft-long, 2-1/4-in-diam vertical blastholes drilled upward on 6-ft centers the entire length of the stope block. Based upon the underground crew's recommendation, 10-ft spacing was adopted, and the holes were drilled with jacklegs and stopers using 1-in rope-thread steel with 2-in and 2-1/4-in cross bits. Two to four drillers, working on a single-shift basis, required less than 2 weeks to drill the 29 holes. No holes were drilled at each future raise location for fear that blasting would create a slabby back.

Prior to shooting the destress round, a seismic velocity survey was performed.

The data showed P-wave velocities averaging about 14,000 ft/s. Higher velocities, indicating either more competent rock, stress concentrations, or both, were recorded at the ends of the stope. The waves passed through the west area at significantly lower velocities. This agrees well with the highly broken nature of the ore body observed in the stope in the vicinity of the September 1980 rock burst. In general, the survey velocities indicated that both ends of the sill pillar (not affected by the prior burst) had the highest probability of future bursting. The destress round was loaded and shot on March 3, 1981.

Following preconditioning, the mining crews were returned to the three stopes and began raising through the sill pillar. The mining plan called for the driving of three-cap raises consisting of manway, timber slide, and two joker chutes above each previous raise. The underground crew, based upon past experience, felt that driving three-cap raises might open too much ground, increase the risk of rock bursting, and create a slabby back. A compromise was reached to drive two-cap raises above the two west stopes and a three-cap raise in the east stope, where conditions were generally better. The west stopes would be serviced from above, and the east stope from below. The geometry of the raises and the stopes and the preconditioning drill pattern are shown in figure 6.

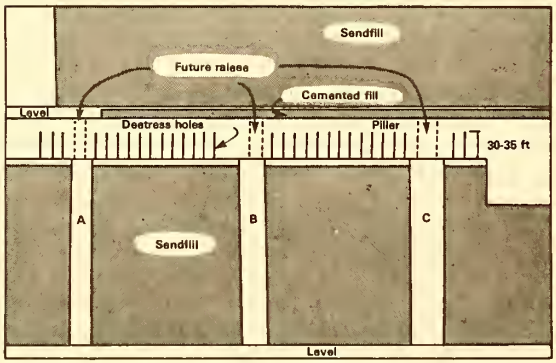

FIGURE 6. - Drilling pattern for pillar preconditioning.

The driving of the raises proved to be fairly difficult in B and C stopes, but was accomplished with little difficulty in A. The rock burst in the B-raise area had left a badly broken ore body and a slabby back. C-raise was located in fairly competent ground, but an inexperienced crew overloaded the raise rounds, shattering the back and creating a problem. Raising required approximately 1 month in A stope and 1-1/2 months in B and C.

For procedure evaluation, it is noted that the miners had some work reservations, and absenteeism was a problem. Also, the mining method conventionally employed at the mine and the particular site geometry necessitated some of the above-mentioned special preparations. The haulage drift above the sill pillar had been driven in the vein and, therefore, had to be filled before underhand stoping could begin. In addition, raises had to be driven through the sill pillar because the mine used blind stoping. This would not be required where raises are developed from level to level before stoping commences.

MINING

Stoping beneath the sandfill of the sill drift began in earnest in May 1981. Drifting was accomplished by breasting from the raise in each direction using a modified V-cut (fig. 7). During the initial cut, timber sets were stood beneath the floormat because of slimes, old track, wood, and loose rock beneath the mat. Raise timber required excessive repairs because of high closure rates. This proved to be a continuous problem throughout the project and resulted in much lost time. Mining the first cut required approximately 1-1/2 months in A and 2 months in B and C stopes. During mining of the first cut, a moderate rock burst occurred at some distance out in the wall below A and B stopes. Only minor damage was seen in the stopes, primarily loose slabs shaken down in the access way at the bottom of the pillar.

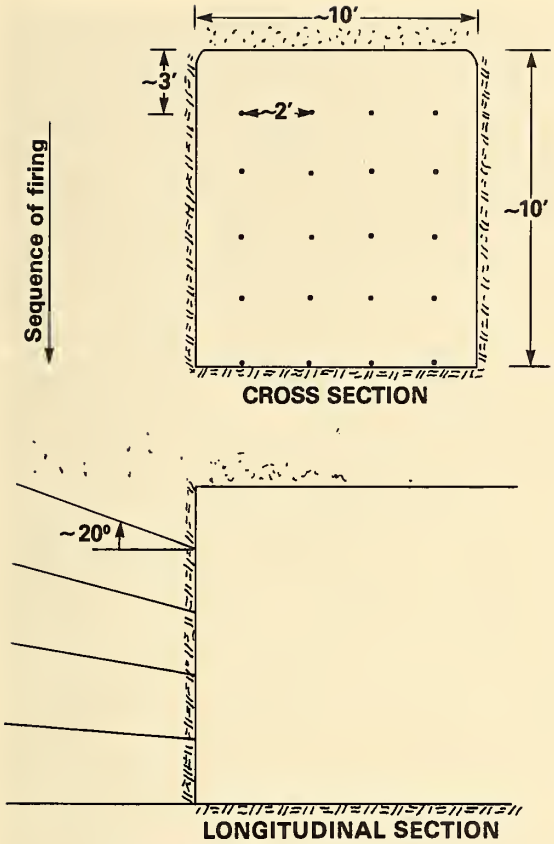

FIGURE 7. - Modified V-cut.

The stopes were prepared for sandfilling of the first cut. A nominal 1-ft layer of broken muck was leveled on the floor as a blast cushion, followed by heading in of caps on 6-ft centers on top of the muck floor. Two rows of lagging were nailed down from cap to cap, followed by a layer of 4- by 4-in No. 8 wire mesh and Fabrene cloth. The floor caps were cabled to the roof caps to prevent their slippage and to eliminate the need for posting on the cut below. A conventional sand wall, seen in figure 8, was constructed. An initial 8:1 sand-to-cement mix was poured to a thickness of 3 to 4 ft, followed by uncemented sandfill in the upper portion of the cut. The high fines content and low pulp density of the fill, as well as poor drainage qualities, resulted in a large loss of cement down the chutes to the level below. Most of the cement and fines remaining in the stope were concentrated at the front of the stope near the sand wall, leaving an inconsistently cemented sandfill over the length of the stope.

Mining in stope A progressed well during the second cut despite a poorly cemented sandfill. As shown in figure 9, there was no need for additional timber in this stope.

Mining in B and C stopes was combined on a double-shift basis, with all broken ore slushed to raise B. Inexperienced mining crews occasionally blasted down some of the overlying caps and fill. Raise closure continued at a nearly constant rate of 1.5 in per month. Raise A experienced nearly 18 in of total closure during 12 months of mining. The stope downtime for replacement of broken raise caps was significant, requiring nearly 3 weeks between cuts. The second cut required approximately 1 month to complete in stope A and nearly 3 months in B.

At this time, a complete report of the mining and the problem with the cemented sandfill was made to the mine management. It was agreed that the sandfill crew would make a concerted attempt to increase the pulp density of the fill and eliminate the high slimes content.

Preparation of the floormat was the same as in the previous cut, with two exceptions: an 18-inch layer of broken muck was left on the stope floor as a blast cushion, and the caps were tied with wire rope to split-set rock bolts placed in the wall, as shown in figure 10, rather than to the overlying caps. Upon filling, however, problems with low pulp density and high slimes content were again encountered, and poor sandwall and stope drainage technique caused a high cement loss down the chutes.

The mining of the third cut in stope A went quite well. Experience, gained in the previous cuts, eliminated basic operational problems. Once mining had progressed two rounds on either side from

FIGURE 8. - Sand wall construction.

the raise, it was possible to cycle each stope face almost nightly. Tying the caps to split-set bolts with wire rope eliminated the need for additional timber. The marginal cemented fill in this cut presented no significant problems due to the narrow (6 to 7 ft) width of the stope. Mining of this third cut required approximately 1 month, including 1 week of raise repair prior to mining. Productivity and cost were greatly improved.

Closure of the stope continued at approximately the same rate as before. Low microseismic activity was likely related to frictional sliding along fractures in the crushing pillar. The data indicated the pillar was failing in a nonviolent manner with little likelihood of blasting.

During this time, the second cut in stope B was completed and prepared for fill. During filling, greater attention was directed to proper drainage, resulting in less spillage of cement to the level below. This produced the best fill to date, as was observed when mining began in the next cut (fig. 11). Though still not of the quality desired, the fill was strong enough to support itself. Both stopes were experiencing a continual problem with much removal. Prior cement spills, which had reached the pockets on the lower level, cemented the broken ore. The result was inadequate storage capacity and muckbound stopes.

FIGURE 9. - Completion of mining in second cut.

FIGURE 10. - Stope prepared for filling.

Preparation of the third-floor mat in stope A required approximately 1 week. The mining crew handled the sandfilling duties themselves. They made an initial 35-min pour, then held the water behind the sand wall, allowing the cement to settle out. The water was then decanted off, and the remainder of the stope was filled. A fairly well-cemented fill was obtained. By the end of December 1981 mining was completed on the third cut of stope B without major mining problems. The hung ore pocket continued to cause a muck removal problem.

The fourth and final cut, which was 14 ft in height, was mined by end slicing, but with conventional breasting-down rounds rather than the horizontal V-cut used thus far. The mining in stope A progressed rapidly and was completed by mid-March. This crew had become quite proficient in underhand mining and had few problems over the last two cuts.

In this final cut, a decrease in closure rate was reflected by fill pressure of approximately 100 psi (before erratic readings developed). Slow progress was made in stope B because of many boulders, which restricted the access way at the bottom of the pillar. Poor ore prices caused layoffs at the mine, and the crews were changed in B, further slowing

FIGURE 11. - Cement fill above third cut.

progress. A general mine closure eventually halted mining at the end of May 1982, with an approximate 50-ft length of pillar remaining. Mining of the pillar following initial raising had required 12 months in stope A and some 14 months in B.

COST COMPARISON

For cost analysis during the demonstration phase, the actual production costs are given in table 1. Data comparison is made to mining of a 35-ft-thick sill pillar in a nearby area of the mine by the overhand method.

It is estimated that underhand cut-and-fill costs would be $48.81 per ton if the method was adopted as a standard procedure for mining sill pillars. This estimate argues that experienced crews in well-prepared stopes would reduce costs in the areas where unscheduled maintenance proved costly in terms of lost production, as well as extra labor and materials.

TABLE 1. - Comparison of sill pillar recovery costs, per ton

Item	Overhand cut and fill	Underhand cut and fill
Labor	$19.94	$22.56
Explosives	1.54	1.15
Rock bolts	1.81	.25
Sandfill	1.25	1.42
Drill repair and drills	1.01	.95
Bits and rods	.77	.91
Timber	1.97	2.09
General-conventional-miscellaneous, underhand-Fabrene, cement	1.99	6.68
Total direct costs per ton	30.28	36.01
Total indirect costs per ton	27.08	27.41
Total costs (excluding milling and general and administrative)	57.36	63.42
Productivity....tons per worker shift	12.04	7.96

CONCLUSIONS

Little difficulty was experienced in applying underhand cut-and-fill mining procedures to the Coeur d'Alene mines. After initial problems were resolved, rounds were cycled daily in the later cuts. Two noteworthy problems were encountered, however. Excessive closure in the raises resulted in a great deal of raise repair. Carefully designed and constructed raises would save considerable time and expense in light of the amount of closure that can be expected in a preconditioned pillar. The second problem was getting a well-consolidated cemented fill. Particle size of the mill tailings was very fine and detrimental to setting of the sand-cement mix. Also, the weight percent solids of the mix was too low. Excess water prevented cement from adhering to the sand particles, allowing it to be carried off with the slimes. Control of the pulp density is critical and should be kept at 65 pct minimum.

Regardless of operational problems, the success of the project must be measured in terms of rock burst reduction. The only rock burst occurred some 100 ft out in the wall below stopes A and B (during mining of the first cut), causing minimal damage in the undercut stopes. Five caps in stope B were broken, but it was not apparent whether this was caused by the burst or by the 4 in of closure that resulted from mining. A minor amount of rock slid off the walls. Only 0.3 in of closure was due to the burst. In the old stopes below, considerable rock was shaken from the back. There was no seismic buildup prior to the burst, and the popping and cracking accompanying mining was of a destressing nature, too small to register on the mine microseismic monitoring system.

The last three cuts were mined without the occurrence of bumps or rock bursts. The minor popping and cracking that accompanied mining of the first three cuts disappeared by the fourth (final) cut as the stopes continued to squeeze and further destress. The high closure accompanying the mining of all cuts, 2 to 4 in per cut, verified the effectiveness of destress blasting in softening the pillar.

Underhand mining beneath a cemented backfill appears to offer greatly improved ground control during sill pillar mining. Provided the cemented fill is of good quality, there is no potential roof fall problem.

A comparison shows the cost of underhand cut and fill was 11 pct higher and productivity was 34 pct lower than that associated with mining a similar pillar using conventional overhand cut and fill. This is largely due to the mine's inexperience with the mining system (including preparation) and cemented backfill. If the costs of the underhand cut and fill are adjusted to reflect an acceptable productivity level, the underhand cut-and-fill mining will reduce sill pillar mining costs by 15 pct.

The conclusion from this demonstration is that the combination of ore preconditioning and underhand mining resulted in greatly improved rock burst and ground control, allowing safe and efficient mining of a potentially hazardous, burst-prone pillar.

HAZARD DETECTION

ACOUSTIC CROSS-BOREHOLE SYSTEM FOR HAZARD DETECTION

By Karen S. Radcliffe[1] and Richard E. Thill[2]

ABSTRACT

A high-frequency (20 kHz) acoustic cross-borehole system has been devised and field tested to remotely investigate the structural conditions of a rock mass in advance of mining. Elastic properties of the rock can be determined by monitoring acoustic waves generated between boreholes, and the structural integrity of the rock mass can be interpreted by evaluation of the acoustic signal characteristics. Identification of hazardous ground conditions prior to mining can help reduce, and ultimately prevent, injuries and fatalities to miners as well as interruptions in the mining operation due to encounters with unstable ground and geologic hazards.

INTRODUCTION

Geologic anomalies can have serious effects on the safety of miners and on production when encountered unexpectedly during a mining operation. Unidentified fracture zones, voids (such as abandoned mine workings or solution cavities), lithologic facies changes, and inclusions, both within and surrounding the ore body, create potentially hazardous conditions for the miner. Fractures, joints, faults, and bedding planes are also critical in determining environmental effects from mining, controlling ground movements, and supporting excavations and mine structures.

These structural features create zones of increased permeability, often containing large quantities of ground water that can quickly inundate the mine when encountered during mining. These same structural features can also produce weakened and therefore unstable roof conditions, resulting in roof falls or collapse at the working face. Also hazardous in coal mining is the methane that may be contained in fractures or joints. Abandoned and unmapped oil and gas wells, and especially abandoned mine workings, create vulnerable conditions for inundation and mine flooding, especially in coal mines.

In addition to the structural weakness inherent in a rock mass from natural discontinuities and geologic features, damage can also be introduced into the mine's supporting structures from mine excavation and blasting operations. Even controlled blasting can result in overbreak into mine structures or weaken already unstable areas.

To help reduce, and ultimately prevent, injuries and fatalities to miners and interruptions in the mining operation from unstable ground and other geologic hazards, it is necessary to remotely investigate the structural conditions in advance of mining. This can be accomplished on a large scale during mine exploration and development, or on a small scale as mining progresses by probing the rock mass structure ahead of the working face. It is necessary to characterize the nature and structural integrity of the rock mass surrounding the material to be mined as well as the integrity of the ore body itself. By characterizing these materials and identifying any hazardous anomalies prior to rock excavation, appropriate safeguards can be taken to reduce or prevent the hazards posed by unstable conditions.

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Various techniques exist for assessing the condition of a rock mass, ranging from mechanical property tests on small-scale samples in the laboratory to large-scale testing under field conditions. Although laboratory testing provides valuable information concerning the intact elements of the rock mass, the effects of large-scale discontinuities in that rock mass are best determined in situ. Mechanical property data that are truly indicative of the behavior of the rock mass under in situ conditions of stress, moisture, and other environmental factors are also best determined in the field. In situ techniques also provide the only opportunity to identify and specifically locate the presence of geologic anomalies and hazardous conditions.

Depending upon the type and size of feature to be located, several in situ geophysical methods for evaluating the structure of a rock mass are available. Nonseismic geophysical surveying includes gravimetric, magnetometric and geoelectric, thermal, radioactive, and geochemical methods (1).[3] Application of these techniques, as well as seismic methods, is based on the presence of a measurable difference in a physical property within the earth materials under evaluation, such as acoustic velocity, density, magnetic susceptibility, or resistivity. Use of nonseismic geophysics in detecting mining hazards is limited by the range over which the methods can be applied and by the degree of variation in physical properties required within the structures of interest. Interpretation of field data is also limited because of its dependence upon the model chosen in the investigation.

Seismic techniques have been used extensively by the petroleum industry for locating geologic structures favorable for economic accumulations of oil and gas. Surface seismic exploration methods have also been used for determining subsurface geology and mapping. Of all the physical methods used in geological exploration, the seismic methods are considered to be the most direct, and when applicable, give the least ambiguous results. Most of the techniques used are surface surveys that operate over a large horizontal distance. The vertical depth to which seismic surveys are effective depends upon the stratigraphy and structure as well as the strength and frequency of the seismic signal utilized in the survey. The applicability of seismic methods is based on the relationship between the acoustic properties of rocks and their physical properties, mineral composition, and structural integrity (2).

To overcome the deficiencies of many surface geophysical methods, an acoustic cross-borehole device was developed and field tested by the Bureau of Mines (3). The apparatus is designed to operate at high frequency (20 kHz) over moderate distances between boreholes. These distances range from a few meters to tens of meters, depending upon the acoustic transmission characteristics of the rock. Applications include evaluating the elastic properties and integrity of underground structures, monitoring stress changes in and around underground workings, evaluating fragmentation and overbreak from blasting, and detecting discontinuities in advance of mining.

ACOUSTIC CROSS-BOREHOLE SYSTEM

The acoustic cross-borehole system operates at a frequency of 20 kHz. It couples under pneumatic or hydraulic pressure to the borehole wall, and can therefore function in water-saturated or dry holes. Acoustic measurements can be made in vertical holes from the surface, or in horizontal or inclined holes in the roof, rib, or floor of an underground mine.

SYSTEM COMPONENTS

The basic acoustic cross-borehole system consists of a high-frequency pulse generator, transmitting and receiving borehole transducers, and monitoring and timing electronics (fig. 1). Auxililary components include a downhole amplifier in the receiver transducer to improve the

[3]Underlined numbers in parentheses refer to items in the list of references at the end of this paper.

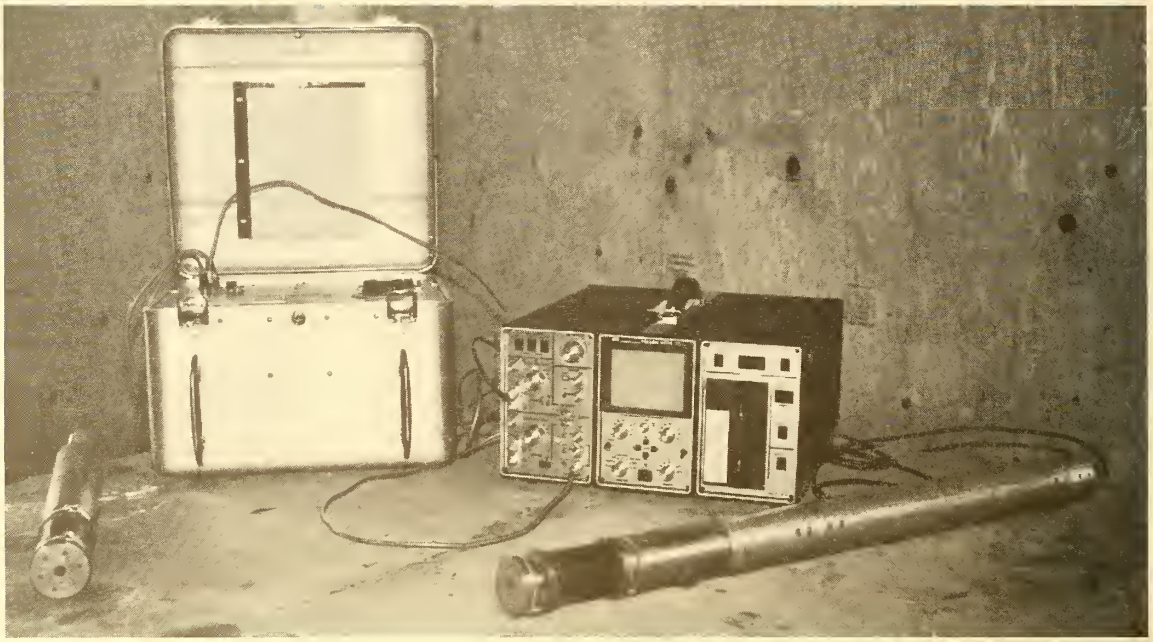

FIGURE 1. - Basic components of acoustic cross-borehole system: borehole transducers, pulse generator, and signal monitoring and timing electronics.

received acoustic signal, and high-voltage coaxial cable for driving the source.

Pulse Generator

A 5-kV electronic pulse generator drives a piezoelectric source in the downhole transmitter. A fast-rising output trigger pulse is provided for synchronizing the timing circuitry. The unit is operable from a conventional 60-Hz, 120-V line source.

Borehole Transducers

The transmitter and receiver transducers are pressure-sensitive hydrophones. Uniform lateral radiation and reception of acoustic energy is achieved through use of cylindrical lead-zirconate piezoelectric transducers operating in the radial expansion mode. A cross-sectional view of the transducer assembly is shown in figure 2. An internal inflatable bladder causes displacement of fluid in the transducer cavity and outward expansion of the outer neoprene boot to couple to the borehole wall. The transducer elements are moderately damped, giving high sensitivity but producing some reverberation ringing. The reverberation is not detrimental to picking the onset of the first arrival normally associated with the compressional wave (P-wave), but it can mask or interfere with subsequent wave arrivals. The emitter and receiver transducers are identical in construction and are 6.67 cm diam by 20 cm long. The central operating frequency of the emitter is 20 kHz. Frequency response of the receiver is essentially flat between 19 and 31 kHz. Operating voltage for the emitter is limited to 2 kV to prevent deterioration of the piezoelectric elements.

Downhole Amplifier

A solid-state amplifier was designed to fit into a small (7-cm-diam) compartment located near the receiver transducer. The amplifier provides substantial signal gain and drives a long length of cable (up to 25 m) with minimum signal loss or distortion. Frequency response of the amplifier is flat over the range 5 to 50 kHz.

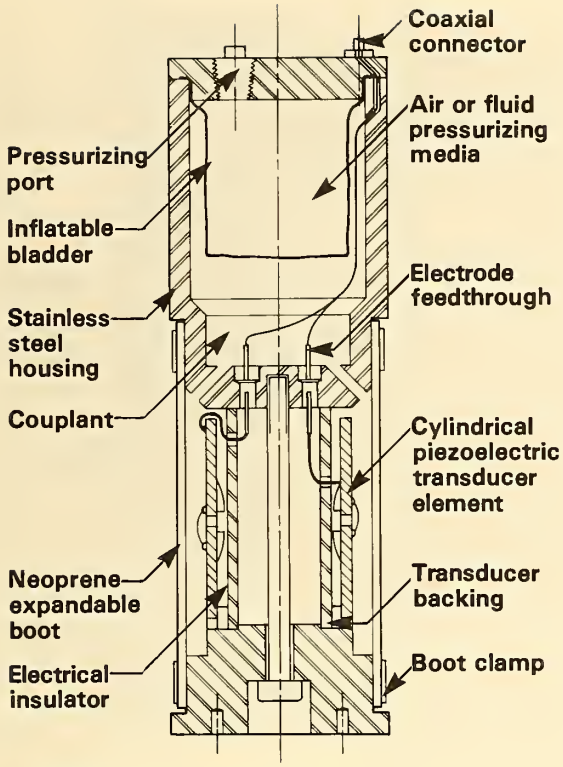

FIGURE 2. - Cross-sectional diagram of borehole transducer assembly.

Monitoring and Timing Circuitry

Conventional oscilloscope waveform display and timing circuitry are used to monitor the received signal and to obtain elastic wave transmit times. Acoustic signals are captured and digitized on a floppy disk to permit detailed waveform analysis.

PRINCIPLES OF OPERATION

The principle of the acoustic cross-borehole system is to propagate a seismic wave from an emitter in one borehole to a receiver in a second borehole (fig. 3). By measuring the wave's traveltime and the distance between boreholes, the acoustic velocity of the seismic wave through the rock mass can be determined. By moving the emitter and receiver along the lengths of their boreholes, profiles of wave velocity or amplitude in the plane including the boreholes can be made. Proper geometric arrangement of the boreholes then enables delineation of the subsurface structures. Calculation of wave velocity assumes a straight line of propagation between the emitter and receiver transducers, and for short propagation paths, the direct compressional wave is normally the first energy to arrive at the receiver. At greater distances, however, and in the presence of higher velocity zones adjacent to the path between the emitter and receiver, refracted waves may begin to reach the receiver before the direct P-wave. Typical wave traces are shown in figure 4.

Deviations from the expected characteristics of the propagated wave can be used to identify structurally hazardous features in mine strata. Characteristics of the seismic wave arriving at the receiver (such as the wave velocity, amplitude, and frequency content) are related to the physical and mechanical properties of the geologic media through which they travel. In rock masses approximating homogeneous, elastic, isotropic media of semi-infinite extent, a seismic disturbance will generate compressional (P-wave) and shear (S-wave) waves that travel with velocities related to the density and elastic constants of the rock medium according to the equations

$$V_p = \left\{ \frac{E(1-\sigma)}{\rho\ (1+\sigma)(1-2\sigma)} \right\}^{1/2} \qquad (1)$$

and

$$V_s = \left\{ \frac{G}{\rho} \right\}^{1/2} \qquad (2)$$

where V_p is the compressional wave velocity, V_s is the shear wave velocity, ρ is the density, σ is Poisson's ratio, and E and G are the Young's and shear moduli, respectively. Surface waves may also be generated at free surfaces of the rock mass by the seismic disturbance (3).

Unfortunately, rock masses are seldom homogeneous, perfectly elastic, or isotropic, but are more often complex structures containing joints, bedding, fractures, and weathered zones. Discontinuities such as these, in addition to

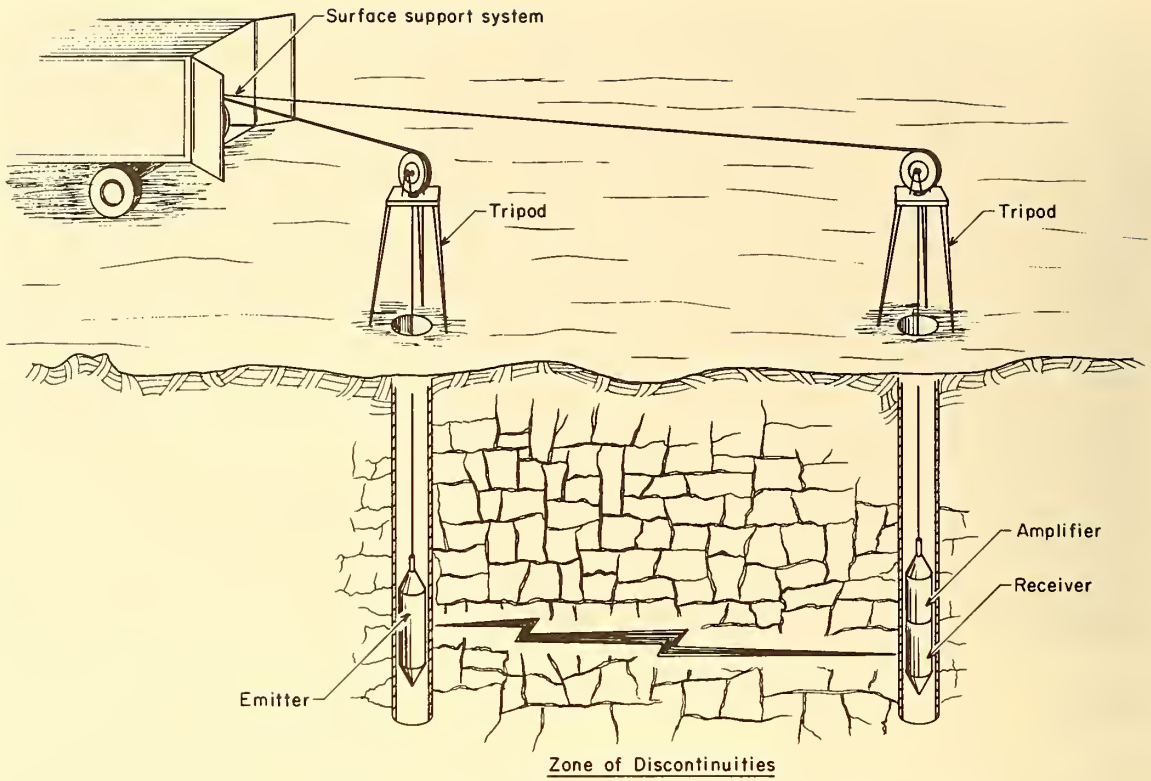

FIGURE 3. - Acoustic system for detecting discontinuities between boreholes.

cavities and voids within the rock mass, all influence the transmission of seismic waves through the rock. The presence of structural features in a rock mass can be identified by monitoring deviations in the compressional- and shear-wave velocities as well as the associated mechanical properties that can be calculated. The structural integrity of a rock mass is assessed by comparing the wave velocity, wave amplitude, other signal characteristics, and the physical and mechanical properties of the rock mass with known characteristics of the intact material.

Reliable detection of geologic hazards requires that there be sufficient acoustic contrast between the target hazard and the surrounding material so that the probing energy will be reflected, refracted, attenuated, or otherwise identifiably modified. Also important with respect to the frequency of the seismic signal generated are the target size and shape compared with the wavelength of the signal. The ability to accurately interpret information about geologic conditions from the detection signals requires prior identification of various types of geologic anomalies and what their effects on propagated seismic signals might be.

Propagating and detecting seismic waves in a discontinuous rock mass requires a tradeoff between range and resolution. Use of high-frequency seismic energy results in better resolution of small geologic features but limits the penetration distance, because of the selective attenuation and absorption of the higher frequency components of the propagating wave. Seismic resolution of geologic anomalies that may cause mining hazards requires use of a wavelength comparable to, and preferably smaller than, the dimensions of the anomaly to be observed. Table 1 describes typical applications of the cross-borehole acoustic technique relative to the seismic frequencies required.

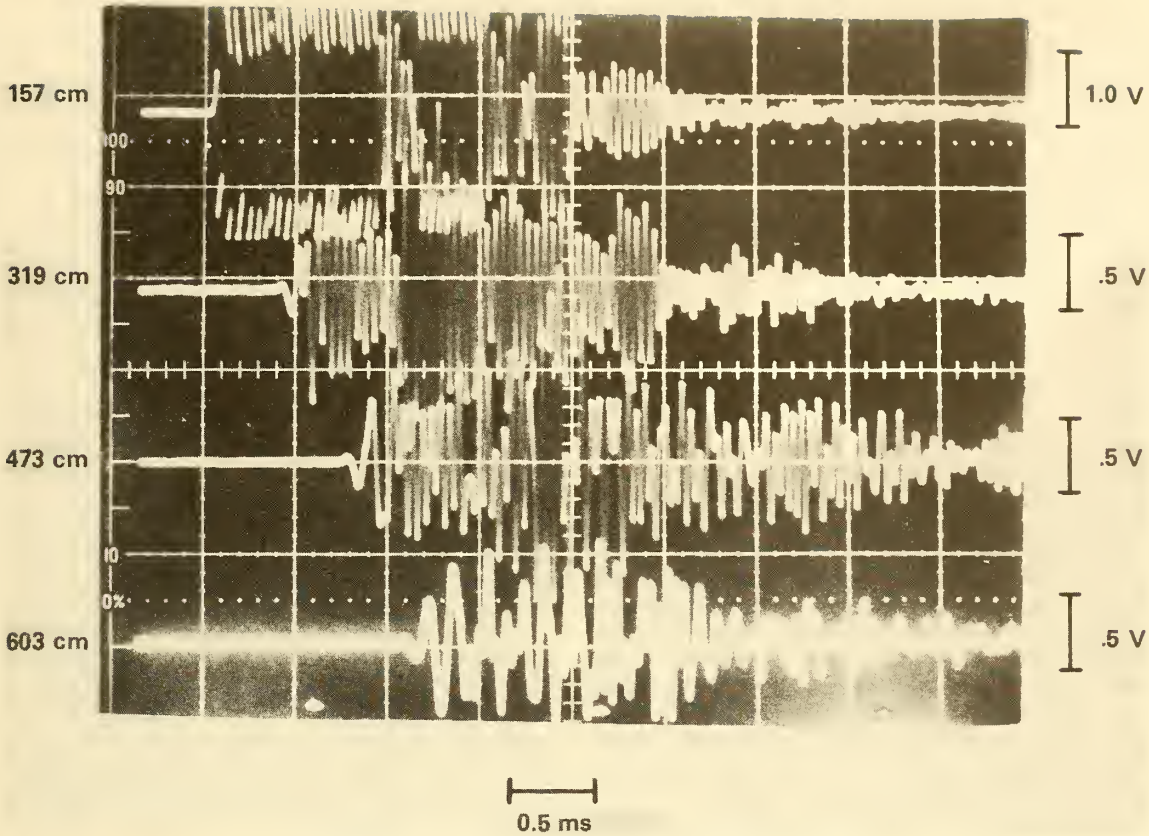

FIGURE 4. - Typical wave traces obtained in situ.

TABLE 1. - Typical applications of acoustic cross-borehole methods

Operational frequency.....	20 kHz	2 kHz
Typical wavelength in rock, m:[1]		
Sandstone.....	0.10-0.18	1.0-1.8
Limestone.....	.16- .28	1.6-2.8
Shale.....	.11	1.1
Coal.....	.06- .14	.6-1.4
Geologic hazards or structural features detected.[2]	Fractures, joints, kettlebottoms, blast damage, small-scale geologic discontinuities.	Faults, channel sands, voids, abandoned mines, large-scale geologic discontinuities.

[1]Reference 2.

[2]Rock mass properties (Young's modulus, shear modulus, bulk modulus, Poisson's ratio) can also be determined at each frequency for various rock types.

PROOF TESTING AND LABORATORY CALIBRATION

The performance capability of the acoustic cross-borehole apparatus was first tested in a block of granite in the laboratory. Tests were conducted to determine waveform characteristics, P-wave velocity, the radiation pattern from the source, the traveltime correction factor for electronic delays and pulse buildup in the transducer, and the optimum coupling pressure.

Testing determined that a minimum pressure of 10 psi was required to couple the transducer to the borehole wall and that amplitude of the first arrival gradually increased until 50 psi, after which there was little change. All subsequent tests were therefore conducted at coupling pressures between 50 and 70 psi. A nearly uniform radiation pattern was observed by monitoring the output from an accelerometer as the receiver was rotated around the source. Small irregularities were attributed to the coupling and surface conditions. Wave transmission tests produced excellent signal reception with well-developed first arrivals. Rise time of the first arrival was about 5 μs, indicating that high frequencies were retained over short transmission distances in the granite. This permitted wave velocity to be determined within 1 pct precision (3).

Mechanical properties of the block were established in advance by conventional ultrasonic and bar resonance tests (4) on core extracted from boreholes in the granite block. These properties provided a basis for comparison of the laboratory data with in situ measurements.

Laboratory testing was also performed in a concrete slab to determine the effects of propagation distance on the amplitude of the first wave arrival. The wave traces verified that the first arrival diminishes rapidly with increasing propagation distance. As demonstrated in the granite block, the high frequencies were filtered through the concrete. Although the time difference between first and second arrivals increases with propagation distance, the change is not large and again introduces an error of less than 1 pct in determining transit time at these distances.

FIELD VERIFICATION

EMERALD ISLE MINESITE

The first field tests of the high-frequency cross-borehole system were conducted at the Emerald Isle open pit mine near Kingman, AZ, where the Bureau of Mines was experimenting with blasting to fracture the rock mass for in situ leaching (5). Acoustic velocities of a copper conglomerate were determined in deep (76 to 85 m), vertical, water-saturated boreholes following blasting. The tests were designed to determine whether the cross-borehole system could be used to detect increased fracturing following blasting. Results showed a decrease in acoustic velocity with depth, corresponding to a deterioration of the rock due to blasting, and delineation of a weathered zone at the granite-gneiss interface. Wave velocities calculated from field data are shown in relation to velocities determined from laboratory tests on preshot core recovered from the boreholes (fig. 5).

SENECA MINESITE

Performance tests of the cross-borehole system were also conducted in inclined, water-filled boreholes underground at the Seneca Mine near Mohawk, MI. In

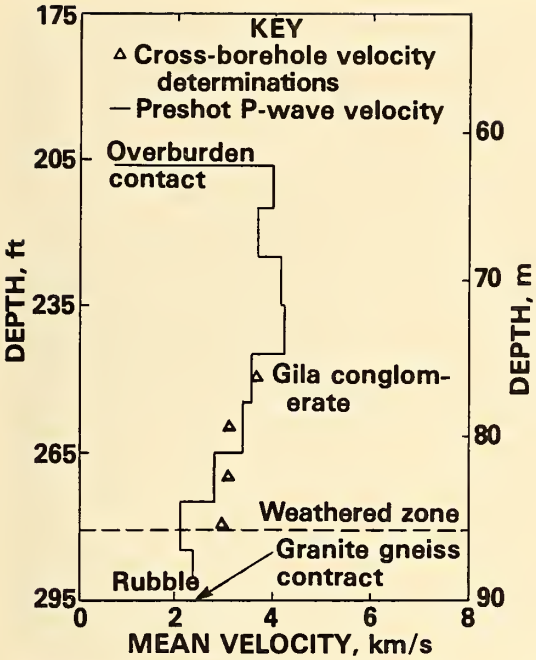

FIGURE 5. - Comparison of laboratory and field acoustic velocities at the Emerald Isle minesite.

studies similar to those at the Emerald Isle site, the Bureau of Mines was conducting research to develop methods of confined blasting to create fracture permeability for in situ leaching of native copper ores (6). Acoustic velocities were determined before and after blasting at a separation distance of 5 m between boreholes. The velocity determinations indicated little, if any, effect from blasting in the zone of the explosives column. Results from permeability and Rock Quality Designation (RQD) measurements conducted at the same time also indicated that the rock was not highly fractured by the blast and that fractures which were present were tightly closed by overburden pressure (3). The excellent quality of the signals received indicated that considerably larger distances (>15 m) could have been traversed in the amygdaloidal basalt host rock.

LOGAN WASH MINESITE

The most recent series of field tests was conducted in cooperation with Occidental Oil Shale, Inc., at the Logan Wash experimental mine near DeBeque, CO. Acoustic velocities were determined under preshot and postshot conditions as in situ oil shale retorts were rubblized using explosives. The field tests were designed to evaluate the cross-borehole system's capability to detect blast damage to the pillars separating the retorts. One set of measurements was taken in vertical holes, and two other series were made at inclinations of 30° and 45° in water-saturated boreholes.

Slight decreases in acoustic velocities were observed following retort rubblization, suggesting increased fracturing in the pillars (fig. 6). The greatest overall decrease in velocity was on the order of 4.5 pct in a zone between 12 to 15 m from the borehole collar and 14 m in from the retort wall. It was expected that there would be a minimal change in velocity before and after blasting, since the measurements were made in the pillar and the blasting was designed to contain fragmentation within the rubble bed in the retort.

Cores extracted from the pillar before and after retort rubblization were subjected to ultrasonic velocity determination in the laboratory to provide another set of data for detecting velocity changes. The observed decrease in field velocity following blasting corresponds favorably with the laboratory acoustic data, as well as with RQD and fracture frequency determined from the preshot and postshot core. This positive correlation suggests that the cross-borehole system is a reliable tool for detecting small variations in geologic structure in situ.

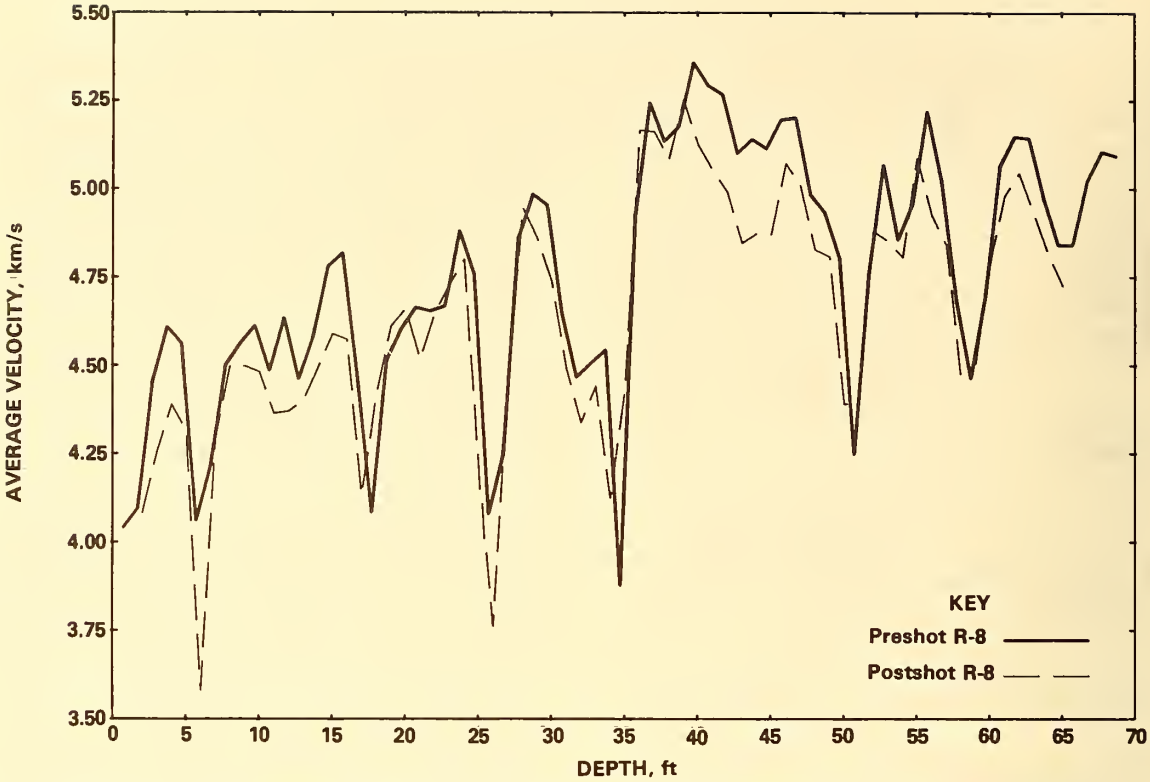

FIGURE 6. - Depth versus acoustic velocity, preshot versus postshot Retort 8, Logan Wash site.

DEVELOPMENT OF LOW-FREQUENCY CROSS-BOREHOLE SYSTEM

A second cross-borehole system is currently in development by the Bureau of Mines that operates at lower frequency (1 to 2 kHz) over wider separation distances between boreholes. The low-frequency seismic signal generated by this system will provide complementary hazard detection capabilities to the high-frequency system by detecting much larger anomalous features prior to mining, and at distances of up to 150 m. This system is especially designed to detect uncharted underground voids in coal measure rocks. It is unique in that it will operate simultaneously in combined modes of reflection and through-transmission to provide the best opportunity to detect hazards in advance of mining (fig. 7).

The acoustic principles utilized by this low-frequency system are the same as those governing the operation of the high-frequency cross-borehole system currently in use. The combined reflection, through-transmission capability, however, provides for a modified method of structure interpretation. In this case, the emitter probe located in one borehole sends out an acoustic pulse toward the receiver probe in a second borehole. A void space, or other geologic discontinuity, will be delineated by the record of transmitted and reflected energy in the acoustic waveform generated by the emitter. A sharp discontinuity between boreholes will reflect back nearly all energy to the receiver-emitter probe and transmit little energy to the distant receiver. Distance to the discontinuous surface can be calculated from the traveltime data. Conversely, in a solid, homogeneous rock mass, nearly all of the energy will be transmitted and little reflected. By profiling along the lengths

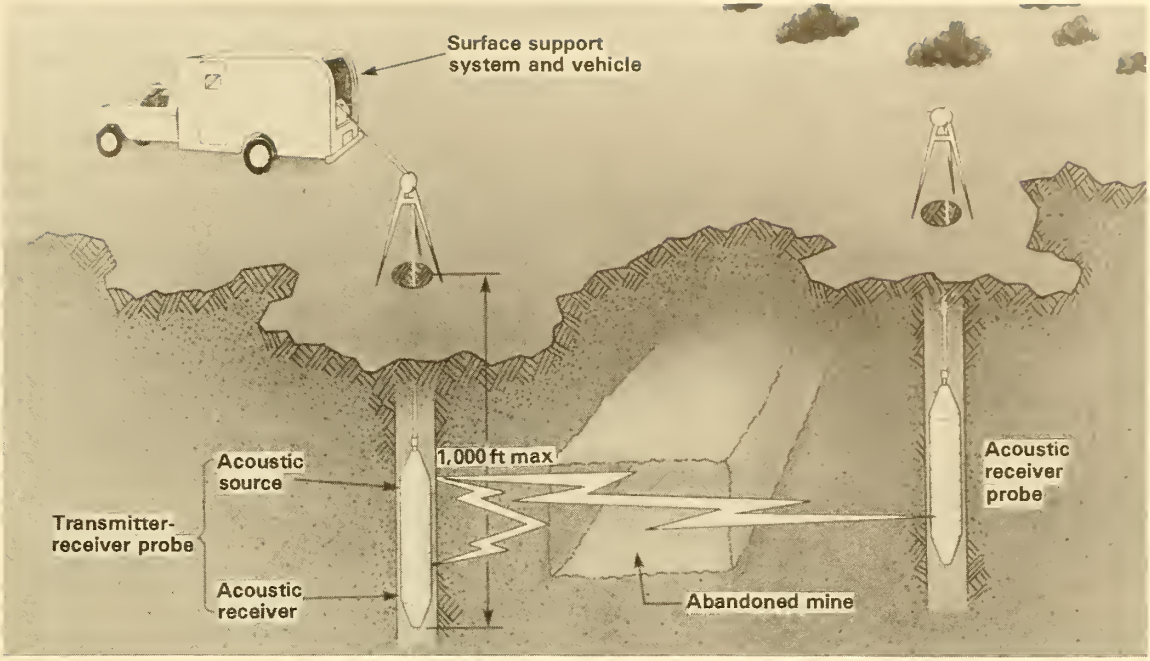

FIGURE 7. - Combined reflection and through-transmission cross-borehole system.

of the boreholes, and by arranging geometric patterns of boreholes, voids and other discontinuities could be delineated in size, shape, and location. Comparison of the transmitted and reflected energy will be useful in interpreting hazards ahead of mining. Sophisticated data processing techniques will also permit more detailed reduction of velocity and frequency data.

CONCLUSIONS

A high-frequency acoustic cross-borehole system has been developed and field tested by the Bureau of Mines for in situ investigation of rock mass properties and detection of geologic hazards in advance of mining. The system operates at a frequency of 20 kHz between boreholes separated by distances from a few meters to tens of meters. Changes in the characteristics of a seismic wave as it propagates through geologic structures can be used to detect potential hazards in the mining environment. Applications of the cross-borehole system include detection of fracturing and overbreak from mine blasting, locating underground voids or abandoned mine workings, inferring stress distributions in mine structures, and detecting geologic hazards in advance of mining.

The second cross-borehole system currently under development will propagate much lower frequency (1 to 2 kHz) seismic waves between boreholes separated by up to 150 m. This system will operate in a combined reflection, through-transmission mode to detect and define the size, shape, and location of larger geologic hazards in advance of mining. The system is specially designed to detect uncharted voids in coal measure rocks. Field testing of the low-frequency system will follow assembly of the equipment components, currently in progress.

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IN-SEAM GEOPHYSICAL TECHNIQUES FOR COAL MINE HAZARD DETECTION

By Richard J. Leckenby[1] and James J. Snodgrass[2]

ABSTRACT

The Bureau of Mines has helped to make a number of advances in improving and developing geophysical methods for detecting mining hazards from the working face in underground coal mines. This paper presents the results of some of the Bureau's recent work in four, in-seam geophysical methods:

1. A synthetic pulse radar system with initial field test results showing its potential.

2. A high-frequency prototype seismic system used to ascertain development criteria for a hand-held scanner.

3. Controlled-source piezoelectric transducers used to generate predominantly compressional or shear wave energy for better control over propagation of guided waves.

4. A dry-hole borehole sonic probe adapted to determine compressional and shear wave velocities in a coal seam from a horizontal borehole.

INTRODUCTION

Coal mines are often plagued by adverse ground conditions because of geology or previous mining or drilling in the area. Common geologic problems include faults, channel sands, split seams, and clay veins, while manmade problems include abandoned mines and wells. These problems can cause roof falls, blowouts, flooding, or inundation. It is important to determine the existence and locations of these problem areas so that precautions can be taken to correct or avoid potential hazards. In addition, timely detection can lead to increased productivity and resource recovery.

One of the most common and costly methods of hazard detection is drilling, but even extensive drilling programs can miss significant geologic and manmade conditions. The Bureau has been investigating, devising, and improving in-seam geophysical methods for accurate, reliable, economic, and practical detection and mapping of potential hazards from the working face.

There is no one proven method that can detect all hazards in all conditions and locations. It is necessary to undertake research to devise a variety of methods that can be used either singly or in combination to provide for the most efficient, reliable, and accurate hazard detection. This paper considers some of the most recent technological advances the Bureau has made in in-seam geophysical methods. It includes a new ground probing radar system called "synthetic pulse radar" and seismic methods which include developments in high-frequency, seismic, guided waves and borehole techniques.

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SYNTHETIC PULSE RADAR

Ground probing radar is a method using electromagnetic waves for the detection and mapping of anomalous conditions in the ground. A number of ground probing radar methods are either being used or being investigated for use.[3] Short pulse radar is probably the most commonly used method, but electronically it is difficult to generate a high-power, high-frequency, broadband short pulse that can be used to detect in-seam coal mine hazards up to 200 ft from the working face. Synthetic pulse radar is a method that overcomes some of the difficulties found in short pulse systems.

Synthetic pulse radar is based on a proprietary concept developed by ENSCO, Inc. A feasibility study was conducted first under a Bureau contract to determine the potential of the concept for improving radar technology.[4] Based on that first study a cost-sharing contract was awarded to XADAR Corp. (a subsidiary of ENSCO, Inc.) to design, build, and test a prototype system for use in underground coal mines.

The construction and initial testing of the synthetic system has been completed to the point where it is now feasible to use the system and to design new measuring and processing techniques to take advantage of this new radar system.

SYNTHESIZED PULSE

To understand synthetic pulse radar and how it differs from the more conventional short pulse radar, it is useful to review some of the basic principles behind Fourier analysis. According to Fourier theorem, any periodic function (wave) can be represented as a sum of a number (possibly infinite) of sine and cosine functions. That is, a periodic wave can be given by the equation:

$$Y = a_o + a_1 \sin \omega t + a_2 \sin 2\omega t + a_3 \sin 3\omega t + \ldots$$

$$a_1' \cos \omega t + a_2' \cos 2\omega t + a_3' \cos 3\omega t + \ldots \qquad (1)$$

This equation is known as a Fourier series, which is composed of a constant a_o, amplitudes a_1, a_2, a_3,... a_1', a_2', a_3' ... and angular frequencies ω, 2ω, 3ω, ... The resultant wave is regarded as being built up by a number of waves whose wavelength ratios are 1, 1/2, 1/3, 1/4, ... From the study of sound this represents the fundamental and its various harmonics.

For waves that are not periodic, but have zero displacement beyond a certain finite range, a Fourier series cannot be used. Instead, a Fourier integral is employed. The nonperiodic wave is composed of (or can be synthesized by adding) an infinite number of wave trains, each having a frequency differing only infinitesimally from the next. The process of determining the components at each frequency is done by what is known as the Fourier transform, and likewise the wave synthesis is done by the inverse Fourier transform.

The above discussion is based on continuous waves, but in actual practice, waves are measured for finite periods. This leads to a whole new discussion on discrete Fourier transforms and a computational method known as the Fast Fourier Transform (FFT) and the Inverse Fast Fourier Transform

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[3]Leckenby, R. J. Electromagnetic Ground Radar Methods. Paper in Premining Investigations for Hardrock Mines. Proceedings: Bureau of Mines Technology Transfer Seminar, Denver, CO, Sept. 25, 1981. BuMines IC 8891, 1982, pp. 36-45.

[4]Fowler, J. C., S. D. Hale, and R. T. Houck. Coal Mine Hazard Detection Using Synthetic Pulse Radar (contract H0292025, ENSCO, Inc.). BuMines OFR 79-81, 1981, 84 pp.

(IFFT),[5] which will be described by simply stating that a FFT of a discrete time series using prescribed cautions, methods, and rules will yield a Fourier series of finite components which will be a good estimate of the actual Fourier transform.

With this general background of the Fourier transform it is possible to see the difference, or probably better stated, the similarity between a short pulse and the synthetic pulse. Figure 1 is composed of three graphs. The top graph is a wave contour of a digitized transmitted pulse after traveling through about 12 m of coal. A FFT was applied to this time domain data to yield the frequency domain data plotted in the lower two graphs. The center graph is the amplitude spectrum, which is the magnitude of the sine and cosine components at each discrete frequency. The bottom graph is the phase spectrum which is the tangential angle between the two components. The wave was digitized with 1,024 samples of equal time intervals, between 0 to 500 ns. This yielded a corresponding spectrum with a fundamental frequency of 2 MHz and 512 harmonic frequencies, of which only the first 65 are plotted because the remaining frequencies are so low in amplitude that they appear almost zero on the graph.

[5]Bracewell, R. N. The Fourier Transform and Its Applications. McGraw-Hill, 1978, pp. 356-381.
Brigham, E. O. The Fast Fourier Transform. Prentice-Hall, 1974, 252 pp.

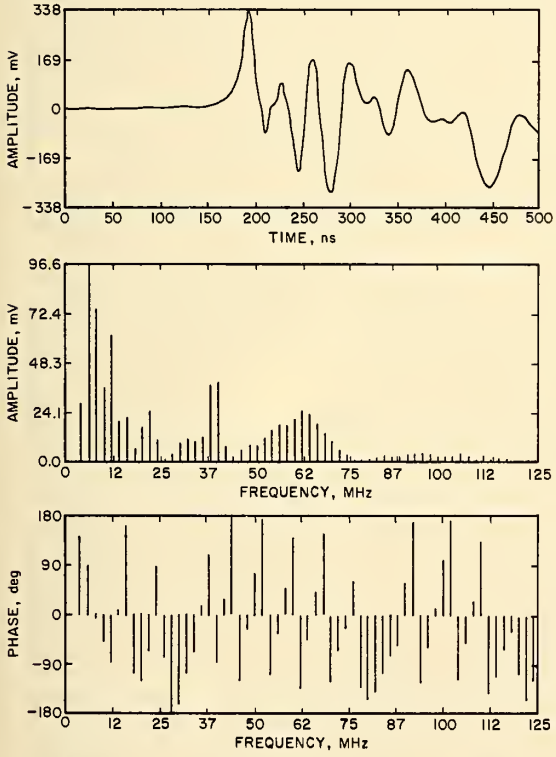

FIGURE 1. - Three graphs showing the short pulse signal in the time domain (top graph) and the frequency domain (bottom two graphs). From top to bottom the graphs are (1) wave contour, (2) amplitude spectrum, and (3) phase spectrum.

In operation each discrete frequency needed to synthesize a pulse is transmitted one at a time, and the resultant amplitude and phase at each frequency is recorded. That is, the acquired data are from the frequency domain and can be presented in a similar fashion as the lower two graphs in figure 1. To ascertain the time domain or corresponding pulse data an IFFT or equation 1 is used to synthesize a wave, which can be presented in a fashion similar to the wave contour of the upper graph in figure 1.

SHORT PULSE VERSUS SYNTHETIC PULSE

The discussion thus far has served to explain how a short pulse and synthetic pulse will basically produce the same results. The major difference is in the hardware and how it controls the energy of the transmitted and received signals. On a short pulse system the pulse is usually generated by a fast discharge of a high-voltage potential into an antenna and a complex impedance. The potential, rate of discharge, antenna, and impedance control which frequencies and their amplitudes will be transmitted into the ground. This in turn will determine the shape of the pulse, the resolution and the effective pulse power, the sum of the power at each frequency, which in turn determines the range of the system.

Figure 1 is a good example of an erratic pulse spectrum that can be ascertained from a short pulse system. On the other hand the synthetic pulse transmits each known frequency at a given power, which assures better control over the shape, resolution, and power of the transmitted signals. It is also easier to obtain higher effective pulse power than by using a broadband short pulse. That is, a small increase in power for each frequency, when summed, gives a large increase in pulse power. Likewise, pulse power may be increased by increasing the number of frequencies transmitted.

THE SYNTHETIC PULSE SYSTEM

The major components of the synthetic pulse system are shown in figure 2. The system employs a hetrodyne receiver, which requires two synthesizers that are phase locked together with a 1-mHz offset. By mixing the transmitted signal and the intermediate frequency (IF) of 1 MHz with the received signal, the detection circuits only have to operate at the IF frequency and not over the entire band of transmitted frequencies. This mixing allows for easier shielding of the input from direct feedthrough of the transmitted signal. A fiber optic link is also used between the control unit and the transmitter to eliminate direct feedthrough signals.

The transmission frequencies are in the range of 20 to 160 MHz, with a minimum frequency spacing of 100 kHz, thus providing a time window of 10,000 ns. The control unit, shown in figure 3, is housed in an aluminum case with its own batteries. The receiver electronics are contained in the receiving antenna. The transmitter, figure 4, consists of a small box containing batteries and electronics with the power amplifier being mounted in the transmitting antenna. The antennas are loaded wideband dipoles similar to those used in short pulse systems. A host computer, which is required to perform the IFFT, can access the digital tape by either a serial or parallel interface.

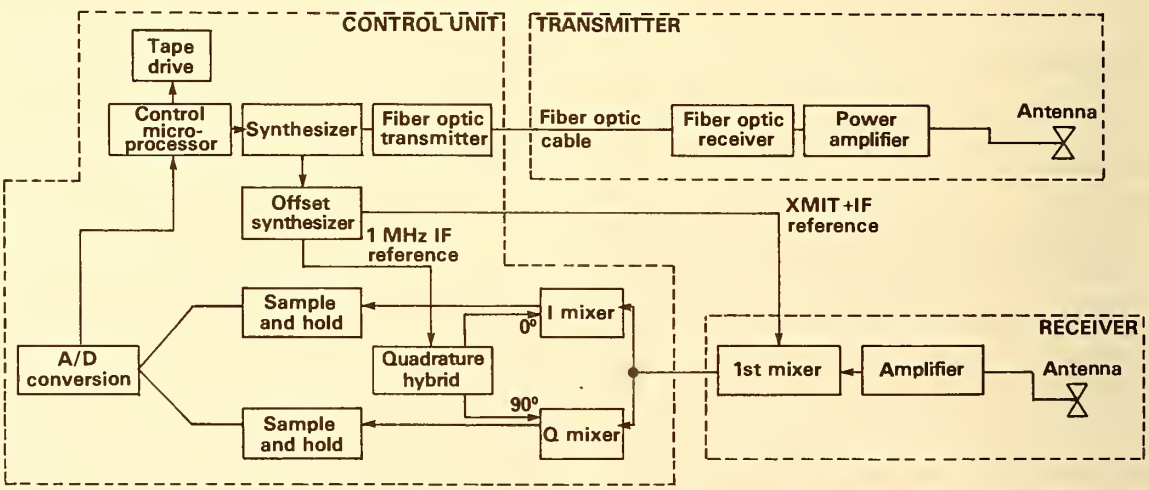

FIGURE 2. - Block diagram of the synthetic pulse radar system.

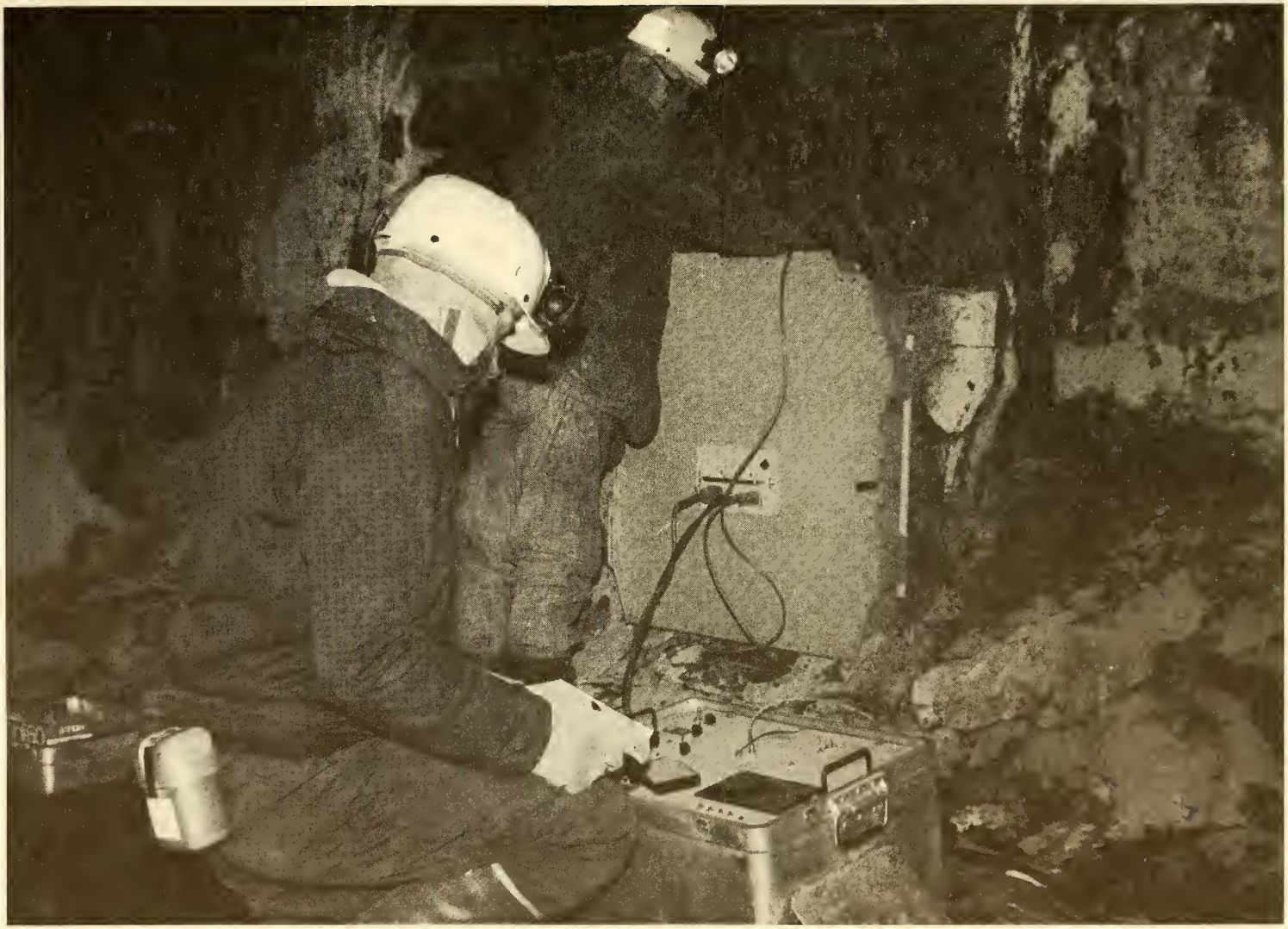

FIGURE 3. - The control unit for the synthetic system (foreground) and the receiving antenna being held next to the coal rib (background).

FIGURE 4. - The transmitter components. From left to right: (1) the transmitting antenna with the power amplifier, (2) a box containing the fiber optic receiver and batteries, and (3) the fiber optic cable and cable reel.

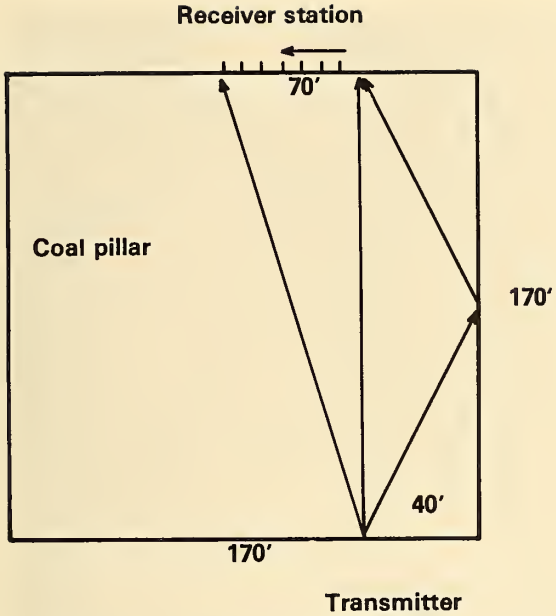

FIGURE 5. - Main travel paths for radar signals through a 170-ft coal pillar.

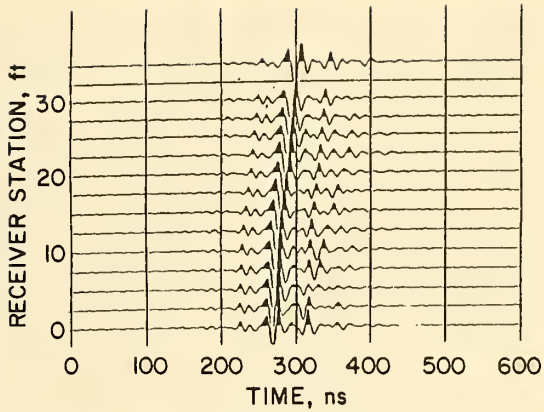

FIGURE 6. - Synthetic pulse radar signals through 170 ft of coal.

SYNTHETIC PULSE FIELD TESTS

The synthetic pulse system was tested at four separate sites to determine its capabilities of detecting hazards in a coal seam. Two tests were in Eastern coal mines and two were in Western coal mines. Figure 5 shows possible travel paths from a transmission test through 170 ft of coal at Consolidation Coal Co.'s Humphrey #7 Mine near to Morgantown, WV. These paths include direct transmission, reflection off the side rib, and refraction along the back rib. Figure 6 shows the synthesized pulses as the receiver was moved along the back rib. Additional "ghost" signals are present in this data, because at the time of this test the power amplifier was in a separate box from the antenna and ringing occurred in the connecting coaxial cable. This problem was later solved by mounting the amplifier directly on the antenna.

The field tests were very successful. The synthetic pulse system more than doubled the probing distance of previously tested short pulse systems. The system was also successfully used in determining the in situ electrical properties of coal. However, the ultimate range and limitations of the system have yet to be determined.

SEISMIC METHODS

Seismic methods in underground coal mines are divided into three categories. The first uses relatively high frequencies for near-field, high-resolution of smaller reflection targets. The second uses guided elastic waves for delineation of major anomalies in the far-field. The third, a borehole technique, provides calibration velocities for the first two techniques with the potential for mapping coal thickness from a horizontal hole.

HIGH-FREQUENCY TECHNIQUE

Operating frequencies in the range of 5,000 Hz have potential for resolving targets as small as 6-in diam within 20 to 30 ft of the rib or face. Such resolution is necessary to locate small voids (e.g., well bores) and small faults ahead of the face. An advantage of the high-frequency technique is that the source and receiver can be configured to

provide a directional beam of energy for a relatively narrow field of view. Thus the boundary effects of reflections from the roof and floor are minimized.

Figures 7 and 8 illustrate a prototype system used to demonstrate the high-frequency technique. The system was developed under a Bureau of Mines contract by the Energy and Minerals Research Co. In figure 7, the source and receiver transducers are shown mounted side-by-side on the rib of a coal pillar. The source and receiver are piezoelectric transducers tuned to the same (5,000 Hz) frequency with a relatively low-Q (broadband) response found appropriate to couple signal energy into coal media. Electronic circuitry (fig. 8) includes a variable pulse-width drive circuit for the source transducer, and signal conditioning circuits for the receiver output.

The system was calibrated by transmitting a pulse through the 18-ft-wide pillar and measuring the traveltime with a receiver mounted on the opposite rib (fig. 9). Velocity of sound through the coal was determined to be 6,922 ft/s, consistent with laboratory measurements of coal samples from this seam. With this information, and by selectively filtering the received signal on the same rib (fig. 7), an easily discernible echo from the opposite rib was observed. These tests show that it is possible to obtain a reflection from a planar interface at distances of at least 18 ft. The detection circuitry, with the velocity calibration, lends itself to numerical readout of distance to the reflector. This will provide an on-site interpretation of the results and allow an unskilled operator to scan a volume of coal ahead of the face for potentially hazardous conditions.

In the preliminary tests of the high-frequency system, transducers were bonded to the coal rib with resin grout to achieve adequate coupling of energy into the coal. Subsequent research has emphasized development of a force-insensitive mounting configuration, which will be a

FIGURE 7. - High-frequency piezoelectric transducers mounted on the rib of a coal pillar.

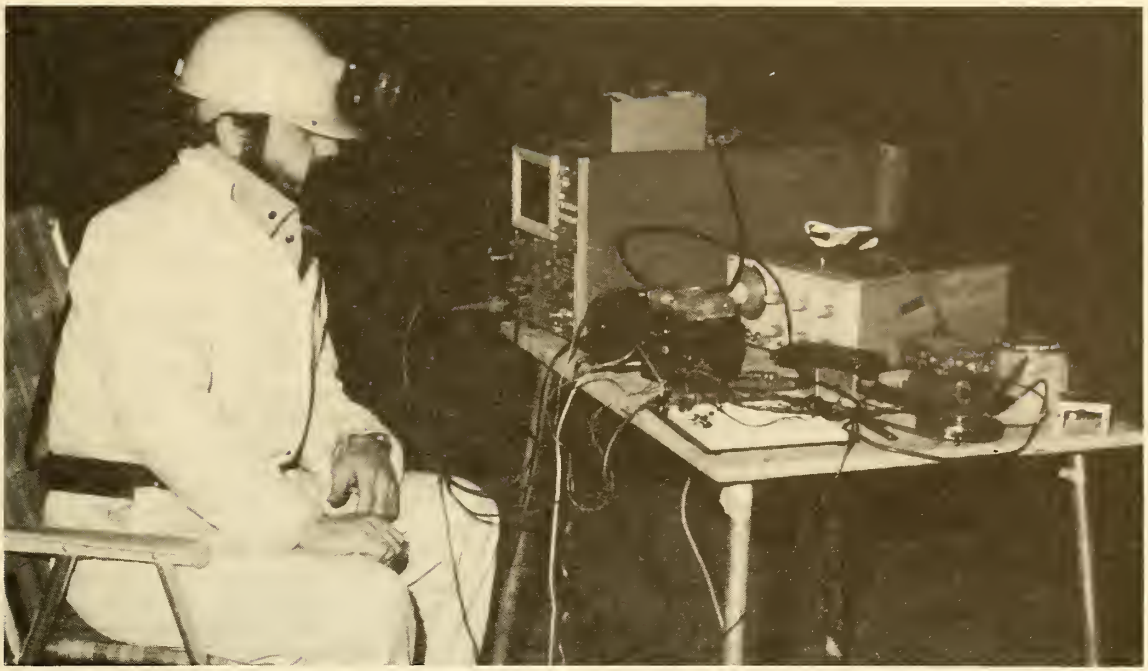

FIGURE 8. - Breadboard electronic circuitry for pulse shaping, signal conditioning, and detection of reflections.

FIGURE 9. - Small piezoelectric transducer mounted on the opposite rib for velocity calibration.

hand-held scanning unit employed at various locations to map anomalous conditions ahead of the face. It was found that an equivalent amount of transmitted energy can be achieved with reduced drive voltage on the transmitter by employing gated bursts of single frequency, rather than single pulses. With lower power levels for energy transmission, the system can readily be made intrinsically safe for operation in coal mines.

GUIDED WAVES

Under certain conditions, a coal seam may be a waveguide for long-range propagation of seismic energy. This requires a seam that is bounded by roof and floor rock that have a greater density and a higher sound velocity than the coal. When these criteria are met, as they generally are in nature (the coal seam being bounded by shale or sandstone), then the multiply reflected waves from the roof and floor will constructively interfere to produce resonant modes of propagation that undergo less attenuation than the direct body waves. These normal resonant modes are dispersive (different frequencies travel at different speeds) and are referred to as Rayleigh- or Love-type waves because of their similarity to earthquake-generated waves, which travel large distances over the earth's surface.

The dispersive nature of the normal modes and the restrictive nature of the underground environment complicate the application of the technique, but the potential benefits of far-range detection of faults and abandoned workings justify research to develop the concept.

Two procedures are used in underground mines: through-transmission and reflection surveys. Seismic sources used are generally small explosive charges placed in drill holes or hammer blows on the rib. Because explosives present an obvious safety hazard, and because hammer blows are not reliably repeatable, the Bureau developed, under a contract, controlled-source piezoelectric transducers (fig. 10) to generate predominantly compressional or shear wave energy. The most desirable type of wave from the standpoint of simplicity in interpretation is a horizontally polarized shear-wave, which will be totally internally reflected within the seam for the normal modes of the Love type. The shear wave source and receiver can be mounted on the rib of a coal pillar (fig. 11) to preferentially excite horizontal particle motion and generate the Love-type modes.

The shear wave source and matching receiver were used to demonstrate the through-transmission and reflection methods in a coal pillar at the Bureau's Safety Research Coal Mine, Bruceton, PA (fig. 12). Waveforms recorded at the receiver directly opposite from the transmitter having the shortest travel path are reproduced in figure 13. The top

FIGURE 10. - Controlled-source shear wave transducers.

FIGURE 11. - Shear wave source mounted on the rib of a coal pillar to generate horizontal vibration.

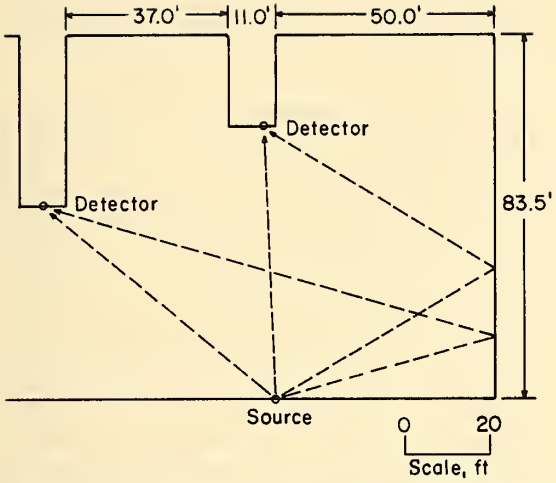

FIGURE 12. - Plan view of the test pillar at the Bruceton Mine with transducer locations and travel paths for seismic signals.

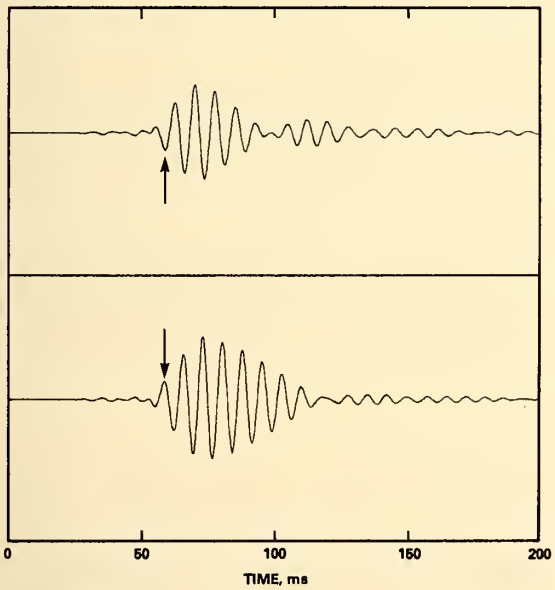

FIGURE 13. - Waveforms recorded at the nearest receiver location. The shear wave source has been rotated 180° to obtain the bottom trace, reversing the polarity of the arrival indicated by the arrow.

trace shows a low-amplitude first arrival followed by a large secondary signal, which is interpreted as the shear wave traveling in the coal seam at a speed of approximately 2,300 ft/s. Support for this interpretation is provided by the lower trace, which was obtained by reversing the pulse direction of the source transmitter. The shear wave arrival clearly shows a 180° phase shift (arrows) as expected.

In figure 14, the waveforms recorded at the two receiver locations are compared. Here the top trace is an amplified version of the previous data for the nearest receiver. The directly transmitted shear wave arrival is again indicated, and a later very similar arrival with lower amplitude and reversed polarity is clearly evident at about 100 ms. Arrival time for this event corresponds to reflection of the shear wave from the end of the coal pillar; a polarity reversal would be expected from the negative reflection coefficient at the coal-air interface. The bottom trace shows the waveform at the far receiver position of figure 12. The direct shear wave is obscured in the early portion of the trace where some apparent resonance or ringing appears; however, after the amplitudes die off, an arrival at about 150 ms corresponds to

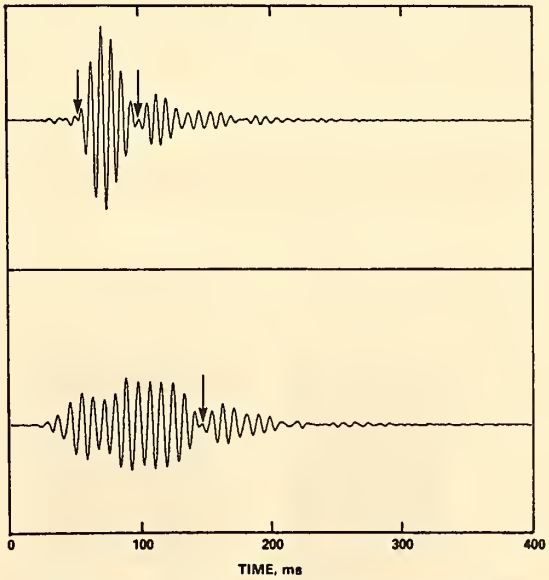

FIGURE 14. - Waveforms recorded at the near receiver (top) and far receiver (bottom).

reflection from the end of the pillar over the longer travel path for the second receiver position. It should be noted in figure 12 that the various travel paths are at different angles to the direction of particle motion excitation; for the far receiver position, proportionately more compressional wave component of energy would be detected, possibly contributing to the higher amplitude early arrival and obscuring the direct shear arrival.

BOREHOLE TESTS

When explosives or hammer blows are used in underground seismic surveys, the more complex waveforms generated require accurate determination of both the compressional and shear wave velocities for interpretation. A borehole sonic logging probe was adapted for use in a horizontal drill hole to investigate velocities near the edges of a coal panel. The probe is a dual-receiver, two-component tool designed to selectively detect particle motion parallel or radial to the borehole axis (compressional or shear waves, respectively). Hydraulic pistons clamp the transducers in rigid contact with the borehole wall, and the probe can be used in either fluid-filled or dry holes. Figure 15 illustrates the probe configuration, with typical waveforms observed on the compressional and shear wave channels at the two receivers.

The logging probe was positioned at a starting depth of 17 ft in a drill hole in a coal pillar (fig. 16). Transducers were clamped to the borehole wall, and full waveform recordings were taken on the compressional and shear wave channels for both receivers, R1 spaced at 4 ft and R2 6 ft from the transmitter. Pistons were then retracted and the process was repeated at 1-ft intervals toward the rib. Velocity determinations from these data are plotted in figures 17 and 18. Both the compressional and shear wave velocities exhibit low values near the rib, reach a shoulder a few feet into the rib, and tend to level off toward a constant value with depth.

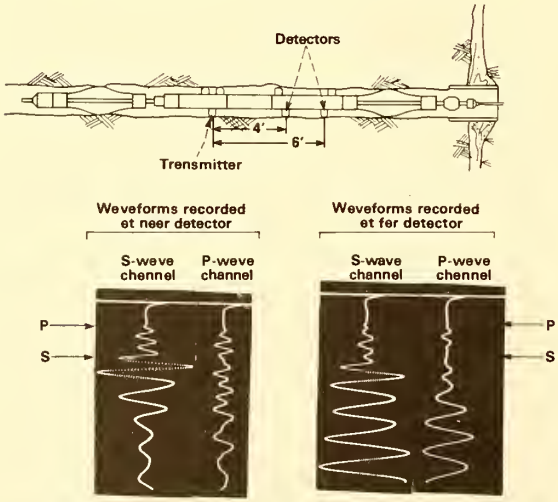

FIGURE 15. - Diagram of dry hole sonic probe and sample waveforms.

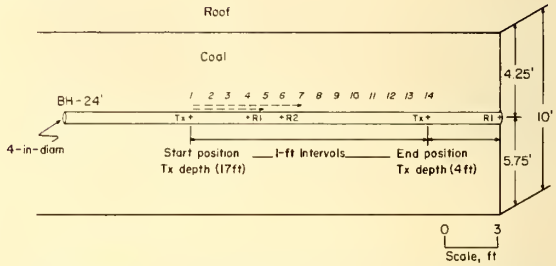

FIGURE 16. - Cross section view of horizontal drill hole in a coal pillar and locations of velocity measurements.

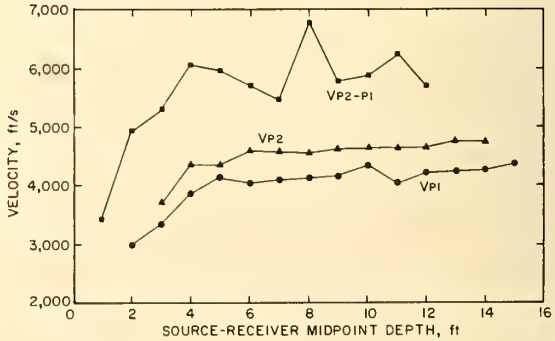

FIGURE 17. - Compressional wave velocities versus depth of measurement in a coal pillar.

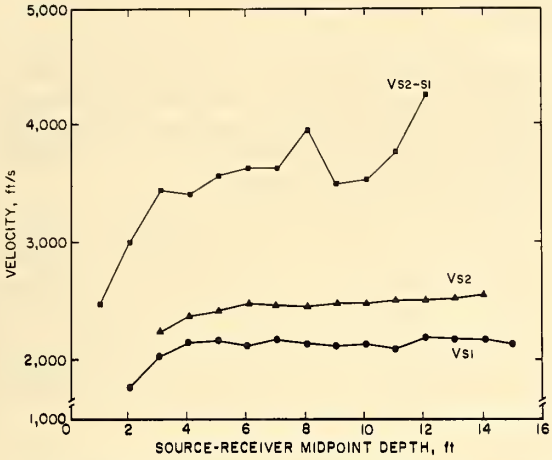

FIGURE 18. - Shear wave velocities versus depth of measurement in a coal pillar.

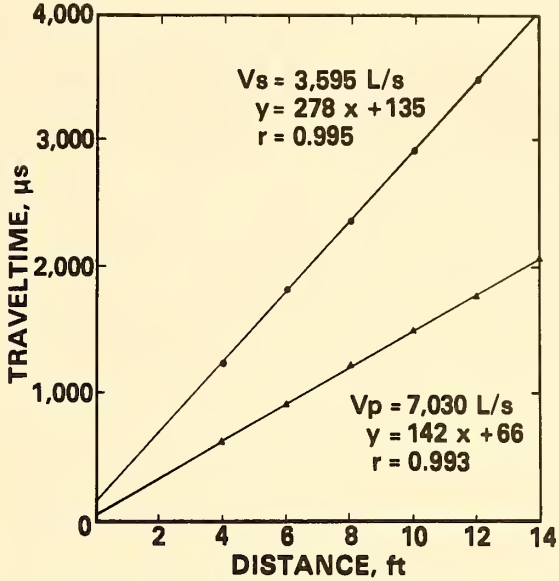

FIGURE 19. - Traveltime curves for determining average velocities within the coal pillar.

The two lower plots in figures 17 and 18 represent average velocity determinations over the 4-ft and 6-ft travel paths from transmitter to receiver, and include the effects of variation in borehole diameter and electronic delays in the probe circuitry, producing somewhat low values. The upper curves represent differential determination of velocities over the 2-ft travel path between the two receivers. They should represent more realistic values because any constant electronic delays are accounted for; however, the values are more variable because of greater sensitivity to local anomalies in the borehole wall.

Average velocities for calibration of the seismic surveys were established by plotting a summation of the differential traveltimes versus distance along the borehole (fig. 19), neglecting the lower velocity values shallower than the shoulder at about 4-ft depth in figures 17 and 18. A good correlation is achieved with this method using linear regression to provide values of shear and compressional wave velocities which are consistent with previous well-log sonic data in vertical exploration holes.

CONCLUSION

Four geophysical methods--synthetic pulse radar, high-frequency seismic, guided waves, and borehole velocity logging--were investigated by the Bureau to devise better in-seam hazard detection techniques. It is anticipated that these techniques will complement each other and will provide a valuable tool to the mining industry. The synthetic pulse radar provides high-resolution reflection capabilities in the range between 10 ft and 200 ft. The high frequency seismic method, when fully developed, should provide near-range, up to 30 ft, high-resolution reflection detection capabilities, and the lower resolution, long-range guided-wave method using the control sources will provide detection in the hundreds of feet. The borehole sonic probe will be useful for determining the seismic velocities needed for both the high-frequency seismic and guided-wave methods.

Research is continuing on all four methods. This paper demonstrates the potential of the methods. Further research, development, testing, and actual use are required to reach the expectation that is expected for each method.

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SATELLITE IMAGERY AS AN AID TO MINE HAZARD DETECTION

By Robert A. Speirer[1] and Stanton H. Moll[1]

ABSTRACT

The Bureau of Mines is involved in ongoing research to develop potential hazard evaluation maps for mine areas. These maps will be generated using computer-aided methods to analyze Landsat imagery and multivariate data sets. A means of image processing whereby lineament information is enhanced and extraneous information suppressed has been devised. Digital processing is particularly appropriate for picking lineaments from Landsat data because it is faster, less biased, more repeatable and, ultimately, less costly than manual interpretation.

INTRODUCTION

The geologic environment in mining areas directly influences the safety of mine workers. Where possible hazards exist, it is essential that their nature and location be determined before mining into them. The basic mine plan can then be modified at an early date for safety and economy.

Geologic features in coal mines, such as faults or sand channels, cause zones of weakness in the roof due to fracturing or differential compaction. These features usually require that roof bolt plans in their vicinity be modified. Roof or rib falls in underground coal mines and slides in open pits are related to these geologic features and still account for a large number of fatalities. Although fatalities were reduced since the enactment of the Federal Coal Mine Health and Safety Act of 1969, roof falls still cause some 40 pct of all underground coal mine fatalities and disabling injuries.

Detection of possible zones of weakness using data derived from Landsat satellites has been demonstrated by many researchers. Since the zones of weakness that may cause mine hazards may be reflections of discontinuities in the earth's crust, surface expressions of such discontinuities should be, and often are, visible on Landsat images to the trained interpreter. Much effort has been devoted to mapping lineaments (linear features often representing discontinuities) and demonstrating correspondences between lineaments and natural features (1-2).[2] Additional effort has been applied to the specific task of delineating lineaments in mine areas (3). The Mine Safety and Health Administration (MSHA) also conducts a program to provide technical support for lineament analysis to mine operators.

MANUAL LINEAMENT ANALYSIS

Current technology consists of a trained operator using visual techniques to plot linear features onto a base map from Landsat images. Rinkenberger (4) used this method to demonstrate the correlation of lineaments with known faults and fractures. Generally, such methods, including those used by MSHA, involve the use of analog equipment for edge enhancement and density slicing (mapping ranges of brightness to a single color or brightness) of subsets of standard Landsat scenes. The visually enhanced images are manually interpreted to obtain the

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[2]Underlined numbers in parentheses refer to items in the list of references at the end of this paper.

lineament map, which is then registered and overlaid onto a base map.

An example of a lineament map over a mine area is shown in figure 1. The lineaments have been manually interpreted from a Landsat scene. When overlaid on a mine outline, a high correlation is seen between the lineaments and known roof fall occurrences (small dots). With the aid of a lineament map, the mine operator has some warning of zones of potential hazards and can use the map as a guide in anticipating hazardous ground conditions (large dots). Verification of these hazards may lead to installing additional roof support or to modifying the mine plan to avoid the hazards.

Although this method has proven viable, and is generally accepted by mine planners, it has a number of substantial, interrelated drawbacks. These include:

1. Repeatability. Repeated interpretations of the same image are rarely, if ever, identical. Two interpreters will seldom agree about the length, azimuth, or number of lineaments. Even the same trained interpreter rarely obtains the same results on subsequent trials.

2. Bias. Each interpreter brings his or her own biases to the task. Furthermore, since these are an unconscious, and hence, of unknown quantity, they are extremely difficult to control and

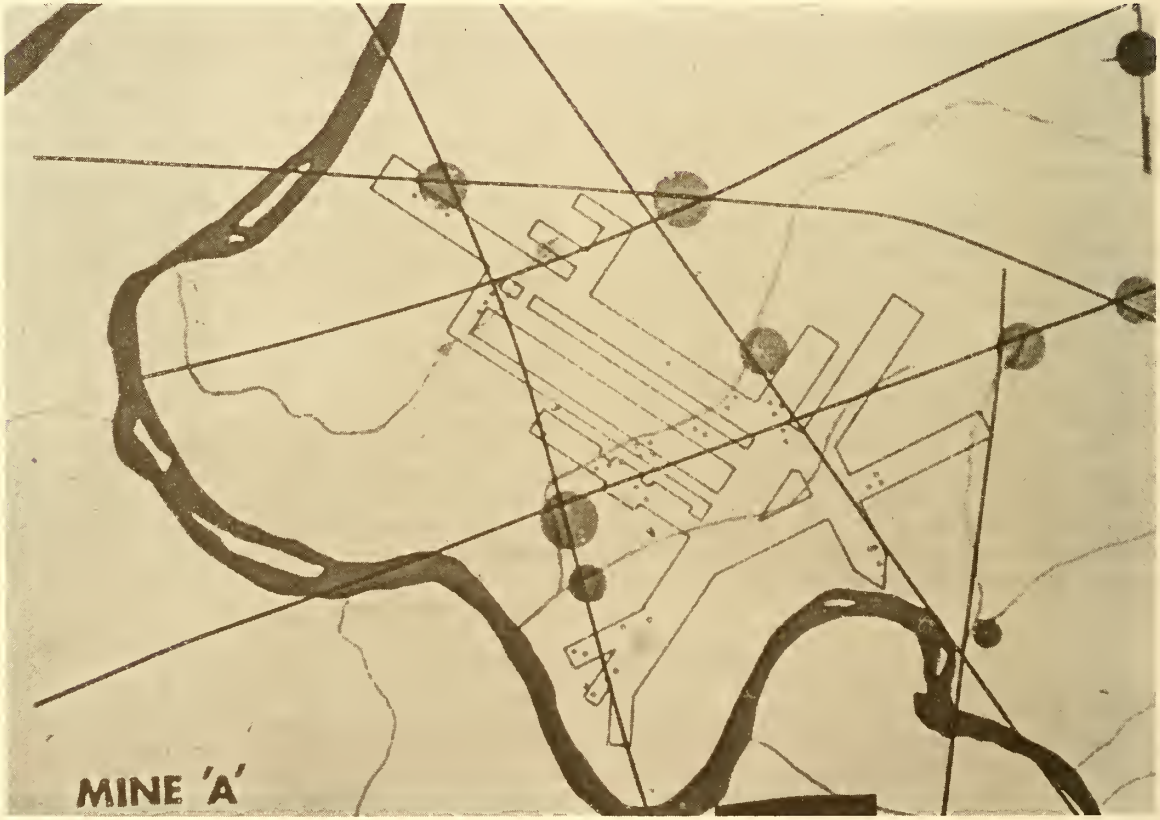

FIGURE 1. - Lineament map registered to a mine map, generated by the manual method. The areas where greater roof instability may be expected are shown in large dots.

compensate for. For example, an interpreter with some previous knowledge of the structure of an area will probably color the interpretation with those preconceptions. One interpreter may prove adept at locating "local" or short features, whereas another may, for example, preferentially locate northeast-trending features over north-trending lineaments. Again, the individual biases may change from trial to trial.

3. Experience. The interpreter must be trained in the "art" of lineament analysis. The more experience an interpreter has, the more he or she will be able to control the problems of repeatability and bias. At the same time, however, the cost of analysis will increase with the increase in experience.

4. Speed and cost. Although a Landsat scene can be analyzed by a trained interpreter using just an image and a pencil, the process is slow and tedious. The cost is low because it consists of only an interpreter's wages and the cost of the image. Enhancement devices, at added cost, can be used to speed up the analysis. Such devices do not, however, relieve the tedium of the manual interpretation process nor eliminate the other drawbacks.

COMPUTER-AIDED LINEAMENT ANALYSIS

Because of the success of the manual lineament analysis technique, the Bureau is devising a program to improve the technique with computer processing. Computer processing and advanced analytical techniques can reduce or eliminate the drawbacks of the manual method while improving throughput and making the technology more widely accessible.

The advantages of using digital methods for lineament analysis are several. These include:

1. Suitability. Landsat data is discrete digital data, and hence lends itself readily to digital processing. Images are available from EROS Data Center, Sioux Falls, SD, that have already been enhanced by digital means to improve image quality. Scenes can also be acquired in digital form as Computer Compatible Tapes (CCT's) and further processed to enhance desirable features such as lineaments.

2. Repeatability. If identical parameters are entered into a program, the computer will repeatedly find an identical crop of lineaments from the same scene.

3. Controlled bias. The computer can be directed to find features of a given size or shape and can be expected to find all features for which it has been directed to search. Different processing methods can enhance or suppress certain features. These capabilities cannot be expected from a human interpreter. Most importantly, the biases of the computer can be known, whereas those of a human interpreter cannot.

4. Flexibility. Because computers are more easily "retrained" than human interpreters, they can be directed to perform many different tasks.

5. Speed and cost. Although initial costs for the computer method are likely to be greater than for the manual technique, this disadvantage is soon overcome by greater throughput, increased reliability and repeatability, and less need for a trained interpreter. Software development costs may be high for a single application, but they can usually be amortized and shared among several uses. Less time is needed to train an operator for the image processing system than to train an interpreter for the manual technique.

An earlier study, now complete, was made to try to devise a computer technique to detect lineaments in Landsat scenes. After transferring the lineaments to a base map, they were field checked to verify the correspondence

between the plotted lineaments and surface geology. The technique proved successful in discovering some features in the mining areas where it was tested. It was apparent, however, that further refinement was needed to learn how to discriminate between manmade features such as roads, canals, fields, vapor trail shadows, etc., and natural phenomena such as faults, fractures, joints, and paleochannels. Furthermore, the technique proved to be "blind" to lineaments in certain orientations, and also to other nonlinear features that may be important, such as circular or serpentine patterns (e.g. calderas and thrust faults).

AUTOLINER PROJECT

The Bureau has recently concluded an interagency project with the U.S. Geological Survey to develop a method of automatic lineament mapping (5). The Autoliner project, although not capable of generating a lineament map per se, did result in a method of processing the Landsat scenes to enhance those natural features of interest to the image analyst while suppressing extraneous information. The resultant image is far superior to a standard image for lineament analysis.

AUTOLINER METHOD

Basically, the autoliner works as follows:

A Landsat satellite records the reflectance of an area on the ground in each of four spectral bands. The smallest area that can be resolved, called a pixel (picture element), measures about 75 m by 75 m. Each pixel, in each band, is sent back to earth as a value between 0 (for no reflectance) to 63 (for total reflectance).

On earth the data is cleaned up (preprocessed) to correct for such aberrations as differences in detector calibration, atmospheric haze, data transmission errors, and geometric distortion. When this procedure is complete, the data still comprises four bands ranging in value from 0 to 63 each, but now is geographically correct. An image made at this point would be brighter and easier to interpret than an image made before preprocessing. These images are marketed by EROS Data Center, and are the images usually used in manual interpretations.

A great deal more processing can be done, however, to enhance specific features. Since the pixel values (called density numbers, or DN) are directly proportional to reflectivity, the difference between pixel values is their contrast. Linear features will usually have a moderate contrast, which will be due to vegetation or elevation differences across the feature. Other features, such as snow-field boundaries, will have extreme contrast, whereas very low contrast is usually minor in content. Consequently, a window can be specified. Outside this window data can be ignored (or set to no contrast), whereas contrasts inside the window can be magnified to further increase the contrast.

There are many ways of establishing the contrast of a pixel with its surroundings. A pixel may be compared with a single immediate neighbor or with any number of surrounding pixels. A small area will more closely reflect the value of its central pixel than will a large area, hence the more distant pixels should be more suppressed than the proximal pixels. These methods of establishing and enhancing contrast were studied in the Autoliner project.

The results of the optimal method for enhancing lineament information is shown in figure 2. Called a "thematic linear feature map," the values within the selected window are shown in black, and all contrast values outside the window are in white. Selected lineaments are shown by pairs of arrows.

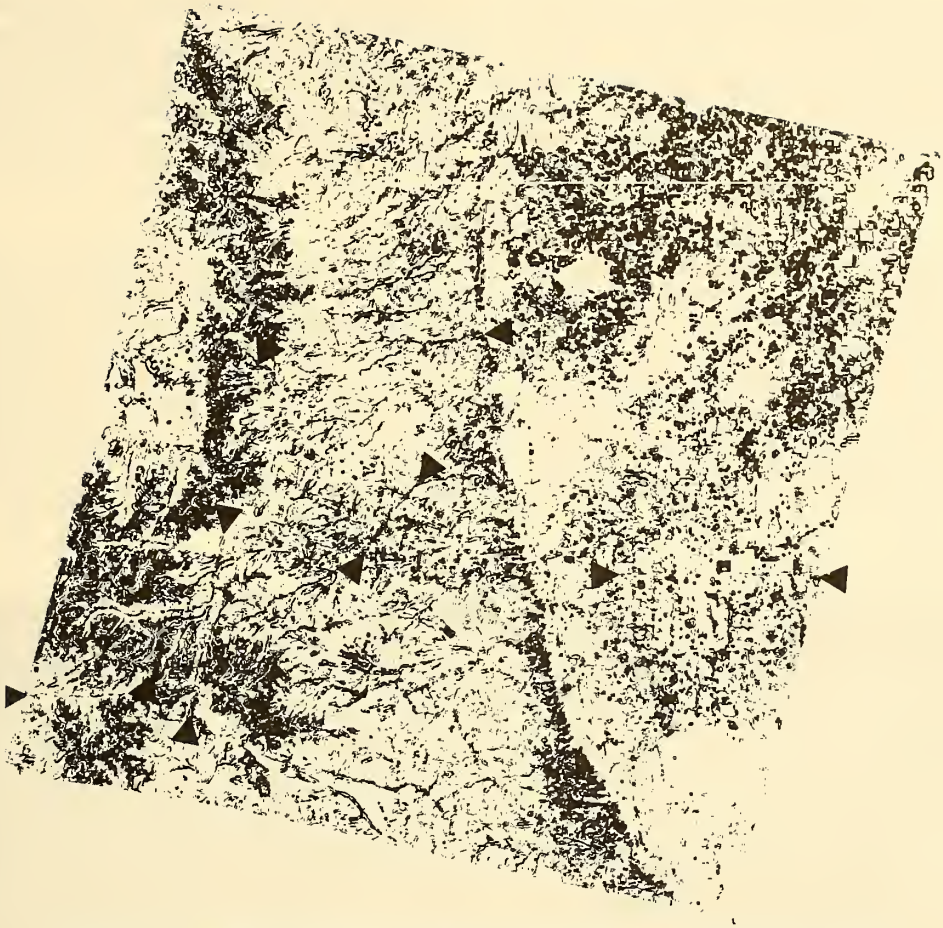

FIGURE 2. - Thematic (binary) lineament feature map of a Denver Landsat 3 image, produced using the modified-modified gradient method. Arrows show lineaments.

AUTOLINER CONCLUSIONS

The Autoliner project resulted in promising techniques of image processing and enhancement for lineament analysis. These techniques permit the highlighting of data that contain lineament information, while suppressing information extraneous to the interpretation. However, the procedures outlined so far must still be interpreted by a trained lineament analyst. That is, they are still subject to visual analysis for lineament picking, albeit with a much improved product.

The image processing software used in the Autoliner project will shortly become available to the public. MIPS (Minicomputer Image Processing System) was designed to operate on a DEC PDP-11 minicomputer, model 23 or higher, running the RSX-11M operating system. Color graphics are provided by a Grinnel model GMR 27 or 270 image processing display system, with hardcopy output via color Optronics or high-resolution Dunn camera.[3]

[3]Reference to a specific brand, equipment, or trade name in this report is made to facilitate understanding and does not imply endorsement by the Bureau of Mines.

Most of the processing routines in MIPS are written in DEC FORTRAN IV PLUS, and hence should be readily transportable to other computers. However, since image processing deals with such large data sets, the decision was made to optimize I/O (data input and output operations) and data transformation routines by writing them in machine-specific assembly language, and using low-level operating system routines. Consequently, translating these routines to another computer will involve some investment of time and probably greater program execution time as well.

FUTURE RESEARCH

The primary objective of Bureau research with the MIPS system is to develop a potential hazard map for use by mine planners. A similar product acquired by the manual method was shown in figure 1. Using the MIPS system, it is hoped that this product can be improved in two ways: (1) by incorporating the "Autoliner" methodology and its derivatives, and (2) by integrating other data sets into the hazard analysis.

Figure 3 shows a prototype of a hazard map generated using digital means. Although numbers have not, at this point, been assigned to the contours, they could represent a variety of quantities, such as degree of hazard or length of roof bolts to be installed. Assessing the parameters to incorporate into the plan, and their respective weights, is one of the objectives of the Bureau's research.

FIGURE 3. - Preliminary potential hazard map of the same area as shown in figure 1, to be generated by integration of Autoliner results and geologic, geophysical, geochemical, and miscellaneous data sets. Contours shown represent arbitrary units and values.

For example, lineament analysis alone cannot provide data about the degree of weakness each lineament represents. For this information, ancillary and complementary data sets must be used. Part of the Bureau's research will be to assess the applicability of these data sets. Data sets which are anticipated as being useful include radar, magnetics, gravity, digital elevation data, geologic mine maps, previous roof-fall data, drilling data, methane and helium concentrations, and other geophysical, geochemical, geological, and miscellaneous data.

CONCLUSIONS

Computer applications are becoming commonplace in mine planning, and we expect that they will soon be common in daily operations as well. Furthermore, given the massive amounts of mine-related data available, the only feasible means of assimilating it is by computer. Our research was undertaken to assist mine planners and operators in promoting safe and economic operating conditions.

Since much of the current work in providing lineament information to mine planners is done by MSHA, the Bureau intends to work closely with MSHA in developing the methodology. It is hoped that eventually MSHA will have image processing systems in their field offices where a semi-automatic analysis can be generated locally for mine operators.

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MICROSEISMIC TECHNIQUES APPLIED TO FAILURE WARNING IN MINES

By Fred W. Leighton[1]

ABSTRACT

Miners have long known that rock noise, or the popping and cracking of the rock commonly heard during mining, can be indicative of the stability of the mine structure. For many years, miners have "listened" to the rock talk and many times have interpreted changes in rock noise activity to be a warning of failure and have retreated from the failure area. Microseismics, or the study of rock noises, was begun in the early 1940's, partly because of this historical fact.

Microseismics uses geophysical equipment to detect and analyze rock noises on both the audible and subaudible level. Thus, these systems are much more sensitive than the human ear and "hear" even more of the rock "talk" than do miners.

Research has shown that microseismics can be used to precisely locate those portions of a working area that are generating rock noise, and that the rock noise release rate information from each area can be used to analyze its stability. In some instances, rock noise data have been analyzed to provide warning of imminent structural failure, and personnel have been removed from or prohibited from entering a failure area. This paper presents a brief history of how microseismics evolved, explains why the technique works, and describes the basic equipment used. Past results in both coal and metal and nonmetal mining systems are shown, and recent results concerning the occurrence of a failure in a coal mine advancing longwall section are presented.

INTRODUCTION

When a rock mass is subjected to changing stress conditions, such as those caused by mining, small-scale adjustments occur within the rock that release seismic energy. This energy, when in the audible range, is called rock noise. Those areas in which stress changes occur are also the areas of the structure most likely to fail. Individual rock noises can be detected and analyzed to determine their precise location relative to the mine structure. Over a period of time, plots of these data provide a pictorial representation of where stress changes are occurring, because rock noise activity tends to concentrate in those areas of the structure most actively adjusting to the changing stresses. Since these areas also represent those areas most likely to fail, potential failure areas can be pinpointed and mapped relative to the structure. Rock noise rates, or the number of rock noises occurring per unit of time, also tend to vary dramatically prior to failure. Thus, the ability to locate the source of individual rock noises provides a means of determining where failure may occur, and rate counting within each area offers a means of assessing when failure may occur. This information, properly treated, can be used as an aid to help avoid, control, or provide warning of impending failure. Successful applications of this technique have provided recent impetus to the effort of developing microseismics into a practical, reliable, and economically feasible tool.

The phenomenon of naturally occurring rock noise was discovered in 1938 by Obert (1),[2] who was measuring seismic

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[2]Underlined numbers in parentheses refer to items in the list of references at the end of this paper.

velocities in mine pillars. Seismic energy other than that which he was generating continually registered on the recording equipment. Further study by Obert and Duvall showed that the extraneous seismic energy being recorded was from rock noises that were being generated naturally within the rock in highly stressed areas. Pursuing this interesting phenomenon, both in the laboratory and in the field, they documented the dramatic change in rock noise rate prior to failure (2-3). Carrying this work further, they established the fact that in many instances, rock burst failures could be predicted by monitoring and "listening" to the rock noise activity in rock-burst-prone areas (2).

These early efforts clearly showed that microseismics had great potential as a tool for measuring or estimating the stability of mine structures. Extended testing, however, showed that sometimes rock bursts occurred with no apparent microseismic warning, and at other times, sharp increases in microseismic data were not accompanied by failure. Also, because precise location of individual rock noises was not at that time possible, one never knew where the failure was going to occur, only that failure nearby the observation point was likely. Thus, while the technique clearly offered promise, it was not at that time considered practical.

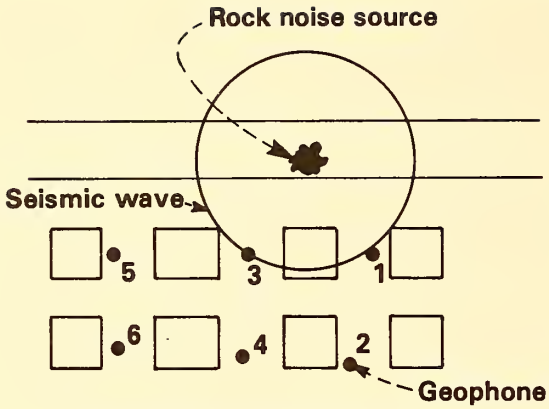
In the mid-1960's, the Bureau of Mines began a new research effort to improve the microseismic technique. Major improvements were judged possible, in great part due to the availability of new and vastly improved electronic and system components which had resulted from instrumentation developed during the space program. Thus, in 1967, development of a multichannel, broadband, microseismic system began (4). Application of this system in rock-burst-prone mines showed it to work well and provided the incentive to develop methods whereby the source location of individual rock noises could be easily and directly calculated (5-7). The improved system and the ability to locate rock noises showed through application that the microseismic technique offered new and increased potential as a useful engineering tool for the mining industry (8). The following sections briefly describe the basis for microseismics and current microseismic systems, and offer examples of how the technique has been and is being applied to mining problems and failure warning research.

HOW MICROSEISMICS WORK

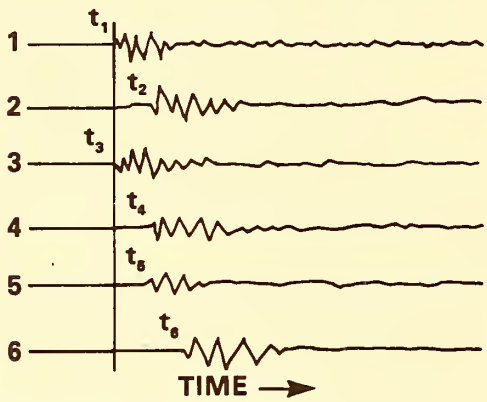
As has been stated, mining results in ever-changing load and stress conditions in the ore body and in the mine structure support system. These changes are accompanied by rock noise, or the release of low-level seismic energy, which can be detected by geophones placed throughout the mine structure. Each individual rock noise can be accurately located relative to the mine structure, thus delineating those areas most actively adjusting to mining. This process is simple and straightforward.

An example of this phenomenon is shown in figure 1, which depicts a plan view of a rock noise occurring in the barrier pillar of a room-and-pillar retreat section in a coal mine. The seismic energy released by the rock noise travels outward from the source of the rock noise in all directions at some measurable velocity (not necessarily the same in all directions), thus arriving at different geophone locations at different times, as shown in the example seismic record in the lower portion of the figure. Knowing the coordinates of each geophone, the velocity at which the seismic energy travels, and the time at which each geophone in the array sensed the arrival of the seismic energy allows one to calculate the coordinates of the rock noise source. Each source is then plotted on the mine structure map, and those areas most actively adjusting to the new stress regime are delineated by the areas in which rock noise activity is most dense. In this manner, potential failure areas are delineated and pinpointed relative to the mine structure.

Figure 2 shows such a rock noise concentration in the two orthogonal



A Example geophone array



B Example record of rock noise

FIGURE 1. - Rock noise. *A*, Plan view of a geophone array and rock noise in a coal mine room-and-pillar section. *B*, Example of arrival time information used to calculate rock noise source location.

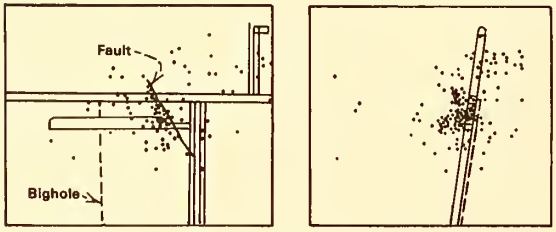


FIGURE 2. - Elevational views of rock noise concentration in a rock-burst-prone pillar.

elevation views of a rock-burst-prone pillar (a rock burst is a sudden massive and sometimes catastrophic failure). This concentration happened over a several-hour period and precisely located the area of a rock burst that occurred at the beginning of the day shift in the area. This example shows the importance of being able to locate where rock noises are being generated and demonstrates the ability of the technique to accurately delineate problem areas.

Another feature of rock noise activity is that the rate at which it occurs tends to fluctuate dramatically in the area prior to failure, which in many instances provides a warning of the failure. Figure 3 shows the rock noise rate, or number of rock noises recorded per unit of time, for the location data shown in figure 2. Note the dramatic increase in the rock noise rate prior to the rock burst.

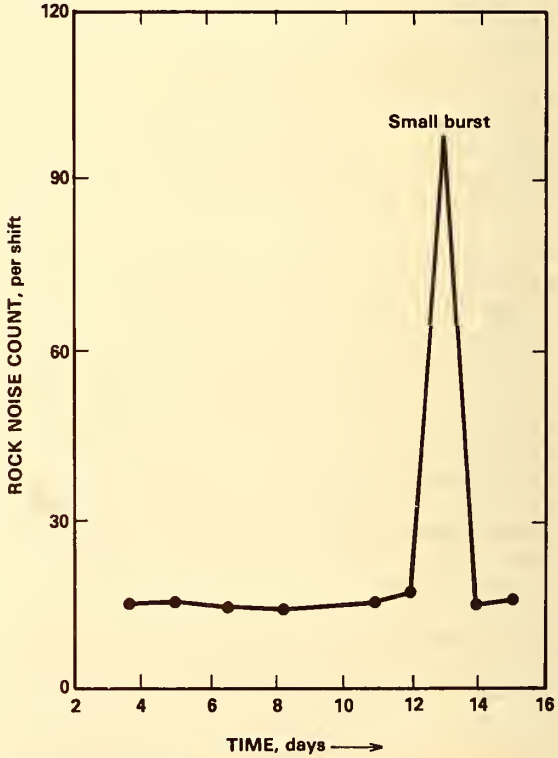


FIGURE 3. - Rock noise rate prior to a rock burst.

These two analyses procedures--i.e., event location and rock noise rate counting--in combination represent the way in which microseismic techniques are used in current practice, when using multichannel microseismic systems. As will be shown, worthwhile applications of single-channel systems to monitor localized areas of interest are also possible, using the rate counting method of analysis without regard to the location of individual events.

MICROSEISMIC SYSTEMS

Microseismic systems may vary somewhat according to the dictates of their application, but essentially all systems include the same fundamental components. A detailed examination of microseismic systems may be found elsewhere (9-11), and only an overview is presented here.

Figure 4 shows a block diagram of a basic multichannel microseismic system. Each channel consists of a sensor, a preamplifier, the data transmission cable, and sometimes a postamplifier to condition the signal for final recording. The sensor may be either an accelerometer or a velocity gauge, depending on the application. Each channel is connected to a multichannel, high-speed magnetic tape recorder and/or an automatic monitoring system that electronically measures the necessary information from the seismic signals. The data measured by this system are fed into a minicomputer or microprocessor, where the data are analyzed to compute the coordinates of the source of each rock noise, which is then plotted on a map of the mine.

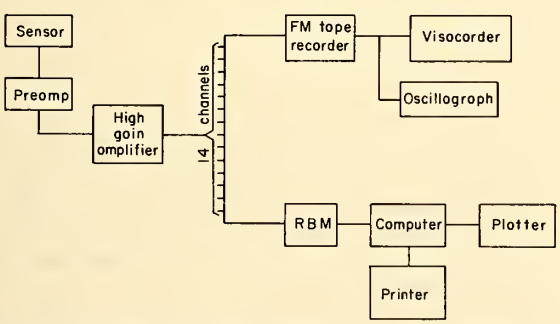


FIGURE 4. - Block diagram of a multichannel microseismic system.

Simpler versions of microseismic systems are available for application where "listening" is carried out using only one channel of equipment. Systems such as these are used in certain specific instances such as in roof fall warning monitoring, where the area of interest is well defined and limited in size. Examples of how these systems can be used are discussed in "Field Applications."

Microseismic systems are now available commercially, either as a total system or by purchasing and assembling individual components. While these systems are not difficult to use, they do require full-time attention and maintenance and are not to be considered a "turnkey" operation requiring little or no ongoing commitment of personnel and capital expenditure. Sensors are sometimes lost during mining and must be replaced, and data transmission lines are often severed or damaged, requiring attention. The data recorded require daily, preferably continuous, analysis and interpretation to be of maximum value. Thus, a microseismic system should be considered a tool to be applied to stability problems and requires the same dedication in terms of attention and maintenance as do other tools used in the mining routine. Several applications of these systems have shown this effort to be worthwhile and contributory to a safer and more productive mine.

FIELD APPLICATIONS

Microseismic systems have been installed and are in use at a number of locations throughout the world. Applications vary from those using single-channel "listening" systems and rate counting methods to those incorporating 24 or more channel systems and the combination of source location and rate counting methods. The following is therefore broken into two discussions; i.e., those applications using single channel systems, and those using multichannel systems.

SINGLE-CHANNEL APPLICATIONS

Single-channel microseismic systems are designed for monitoring well-defined problem areas of limited extent. These systems detect rock noise activity in several frequency ranges, the most common being in the 100- to 5,000-Hz range or the 40- to 100-Hz range. An advantage of single-channel units is that they are portable and may be used for periodic sampling of many areas within the mine, or for continuous monitoring at the working area. The disadvantage is that the source of the rock noise is never known, hence the precise location of a potential failure cannot be delineated. As will be shown, this disadvantage does not preclude beneficial use of single-channel technology in many instances.

Single-channel data may be recorded and used in a variety of ways, ranging from counting noises heard through a set of earphones to permanently recording and analyzing data using sophisticated electronics. In the former instance, the success of the application relies heavily on the dedication and skill of the observer. In the latter case, the system can perform its function essentially unattended. Simple systems using headphones and an observer may cost in the $1,000 range, while more sophisticated systems may cost $10,000. The choice of which system to use and consequently how much money to invest is dictated by the application.

In terms of failure warning, highly promising research has recently been done using equipment sensitive to the frequency range of from 40 to 100 Hz (12-13). A major advantage of this system over the systems sensitive to the lower frequency ranges is that manmade noise, such as that due to mining, is comprised mostly of frequencies lower than 400 kHz, hence the data recorded are mostly, if not entirely, made up of noises naturally occurring in the mine structure. The system can thus monitor right in the working area, where there is the highest risk for personnel.

The system as discussed below was constructed specifically to provide warning of impending roof falls. It was designed to provide coverage of about a 50- or 100-ft radius from the sensor, so that it would monitor only the working area minimizing the importance of individual rock noise locations. When a warning was sounded, one would simply evacuate the area immediately surrounding the sensor. In practice, this system has been found to perform well with large-scale roof falls. Figure 5 shows the commercial prototype of this device. The system is a stand-alone design, providing continuous monitoring, automatic data analysis, and warning of failure independent of human input.

This system differs from previous single-channel monitoring systems in that it measures not only the number of events that occur within its range, as do standard systems, but also estimates the total amount of energy released by those events. The event count and energy value are accumulated for 60-s intervals, and then the energy value is divided by the total event count to obtain the energy-event ratio. This ratio calculation is unique to this instrument. In application, the energy-event ratio behaves anomalously before failure, and the anomaly is sufficiently large to be easily detectable and to be used as a failure warning.

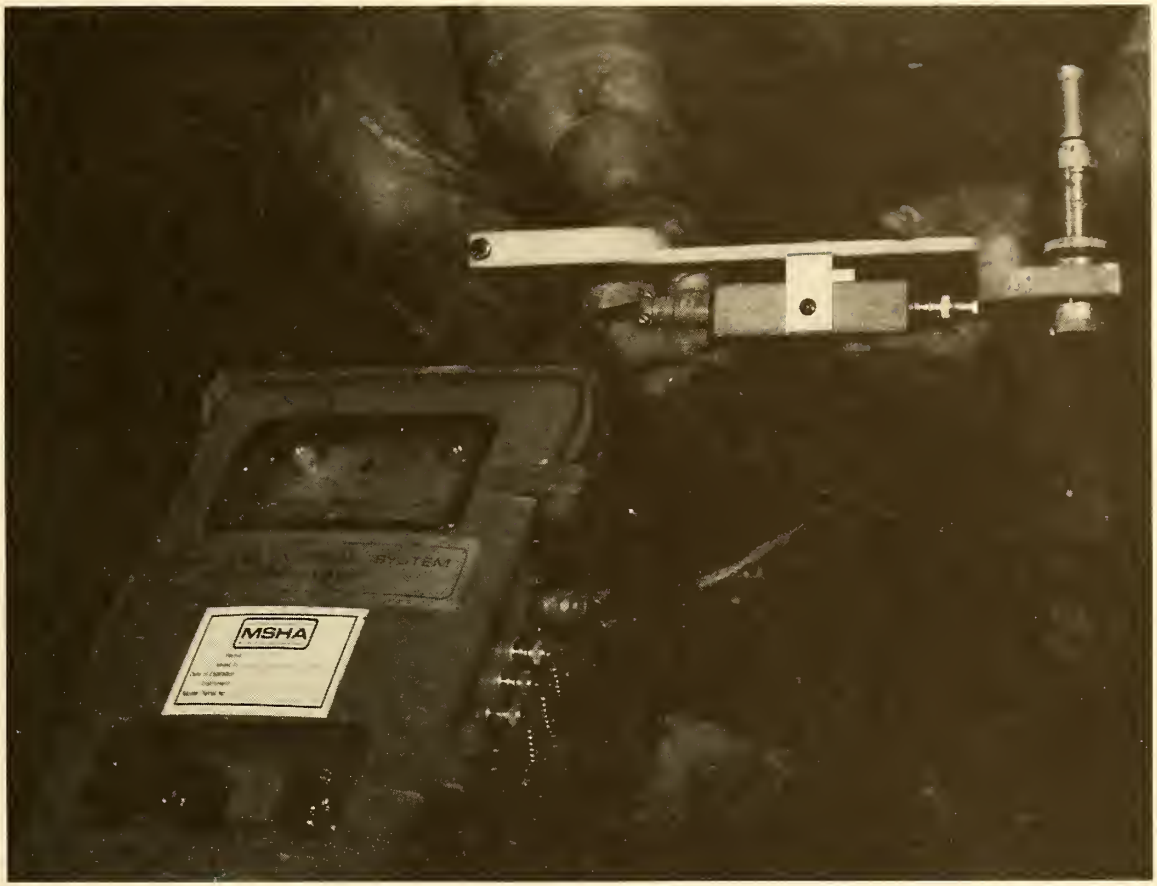


FIGURE 5. - Components of a high-frequency, single sensor microseismic system.

Figure 6 shows the behavior of the energy-event ratio before a large scale roof fall (fig. 6A) and before a coal and gas outburst, a coal mine failure similar to a rock burst (fig. 6B). In both instances, the anomaly is large and occurs sufficiently prior to the actual failure to ensure removal of personnel from the failure area.

This system has shown much success in providing warning of large-scale roof falls. Its efficacy as a warning device for small-scale roof falls, coal and gas outbursts, and other types of failures has not yet been determined, but it has been shown to work on some occasions. These areas of application are the focus for current research efforts by the Bureau of Mines with the goal of establishing the procedures to ensure proper use of the device in the field and its reliability in different applications.

Single-channel applications of this device and others have been historically proven to be of value and to be capable of providing meaningful information about the mine structure in specific areas of limited size. To carry out similar studies over a larger area, even up to the size of the entire mine, multichannel systems are necessary.

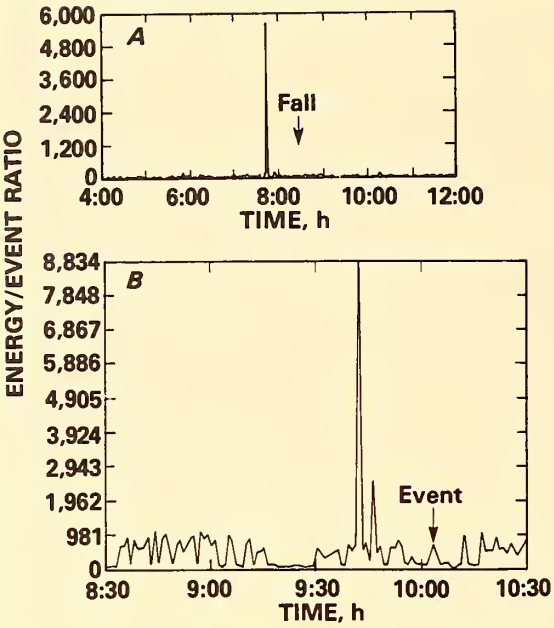


FIGURE 6. - Energy-event ratio prior to a roof fall (*A*) and an outburst (*B*).

MULTICHANNEL APPLICATIONS

Multichannel microseismic systems were developed by the Bureau in the late 1960's, and research is continuing to demonstrate their usefulness in solving specific mine failure problems and in improving total systems reliability. Meanwhile, several applications of the technology have been made at its present level of development.

Early examples of systems application can be seen in figures 7 and 8, which are Bureau of Mines research results (12, 14). The important difference between single-channel applications and multichannel monitoring is that the location of each individual rock noise can be calculated and plotted on mine maps with the latter system. Thus, a larger area of the mine, even the entire mine if necessary, can be monitored on a continuous basis, and potential failure areas can be accurately pinpointed within the mine structure. This capability sometimes allows for mine support to be modified to avoid failure, for the application of destressing techniques to control failure, or for the removal of personnel from the area prior to failure.

Figure 7 shows the rock noise activity plots for a 5-day period before a rock burst in a stope pillar in a metal mine. As can be seen, this procedure precisely defined the location of the eventual failure. Figure 7F shows the rock noise rate which describes how rock noises in the potential failure area occurred as a function of time. As in single-channel applications, note the dramatic increase in rock noise rate prior to the rock burst.

A similar example, this time comprised of data from a Bureau study in a coal mine, shows the rock noise data recorded in conjunction with a coal mine bounce (a failure very similar to a rock burst) (fig. 8). The rock noise activity has been contoured in terms of density, so that the inner contours represent the area in which the most rock noise is occurring. Again, over a period of days, notice the dense pattern of rock noises that occurred in the eventual failure area, precisely defining its location. Also, note the similar reaction of the rock noise rate from the failure area before its failure, as shown in figure 8F.

Another recent example of results, from the current Bureau study in a longwall coal mining system, is shown in figure 9. The cumulative rock noise location data shown in figure 9A, were recorded during the period April 7-14, 1983. A major bounce occurred in this section on April 20, 1983, and caused damage in the tailgate and along the face exactly in the area delineated by the rock noise data. The rock noise rate also changed during this time period, indicating unusual behavior of that face area.

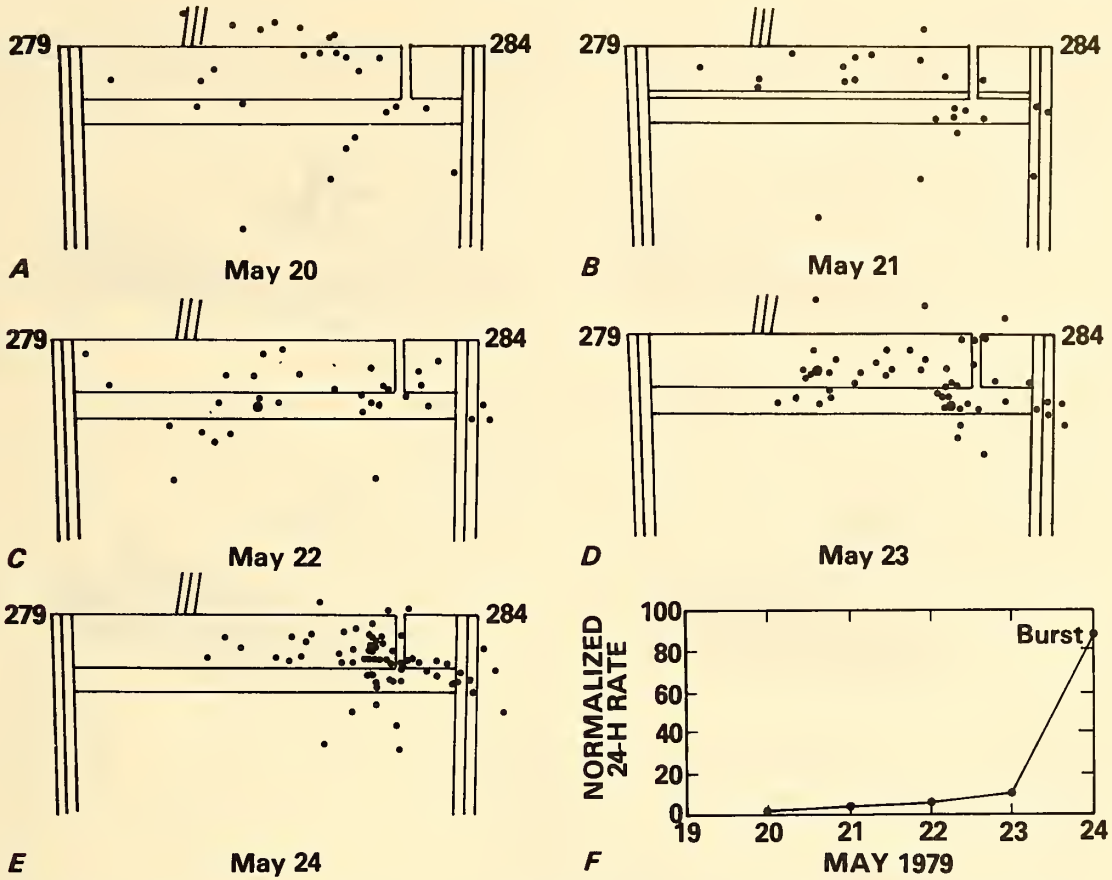


FIGURE 7. - Elevational views of rock noise concentrations and rate change prior to a rock burst.

This data set is incomplete, owing to irregularity in the monitoring procedure during this time period, and the prehistory and posthistory of this isolated data set are unknown. The data cannot therefore be said to have offered conclusive evidence regarding failure warning. The data are important however, in that they (1) provide additional support to the hypothesis that bounce areas can be delineated well in advance of their failure, and (2) indicate the viability of these techniques in the mechanized longwall mining environment, an important contribution in light of the growing number of longwalls in the coal mining industry.

The above examples were obtained using small geophone arrays comprised of many sensors to obtain precise locations of rock noises and to provide precise failure data of specific areas of interest. Similar applications are made in industry, but the geophones are more widely spaced to obtain widespread coverage, and

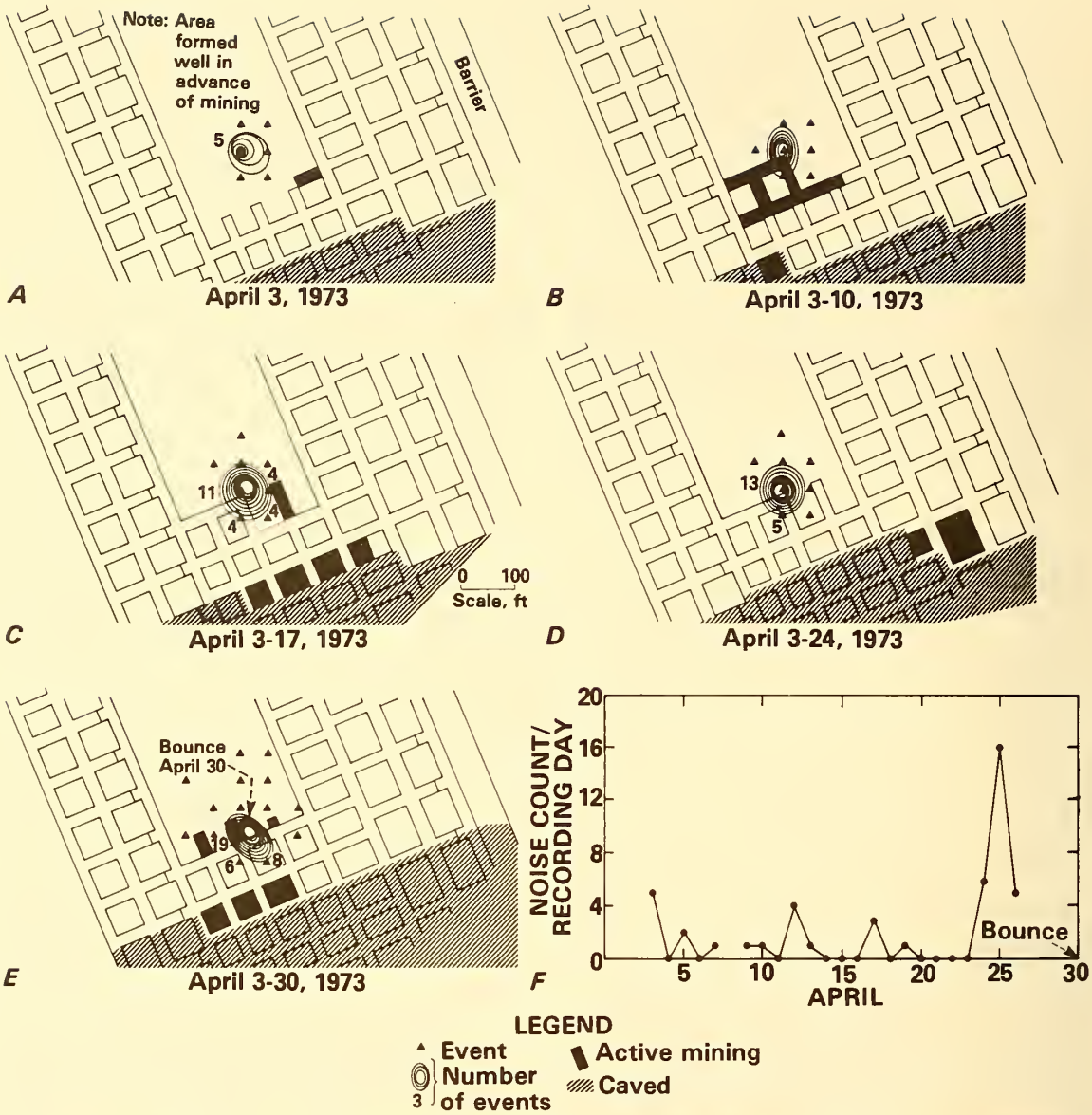


FIGURE 8. - Plan views of rock noise concentrations and rate change prior to a coal mine bounce.

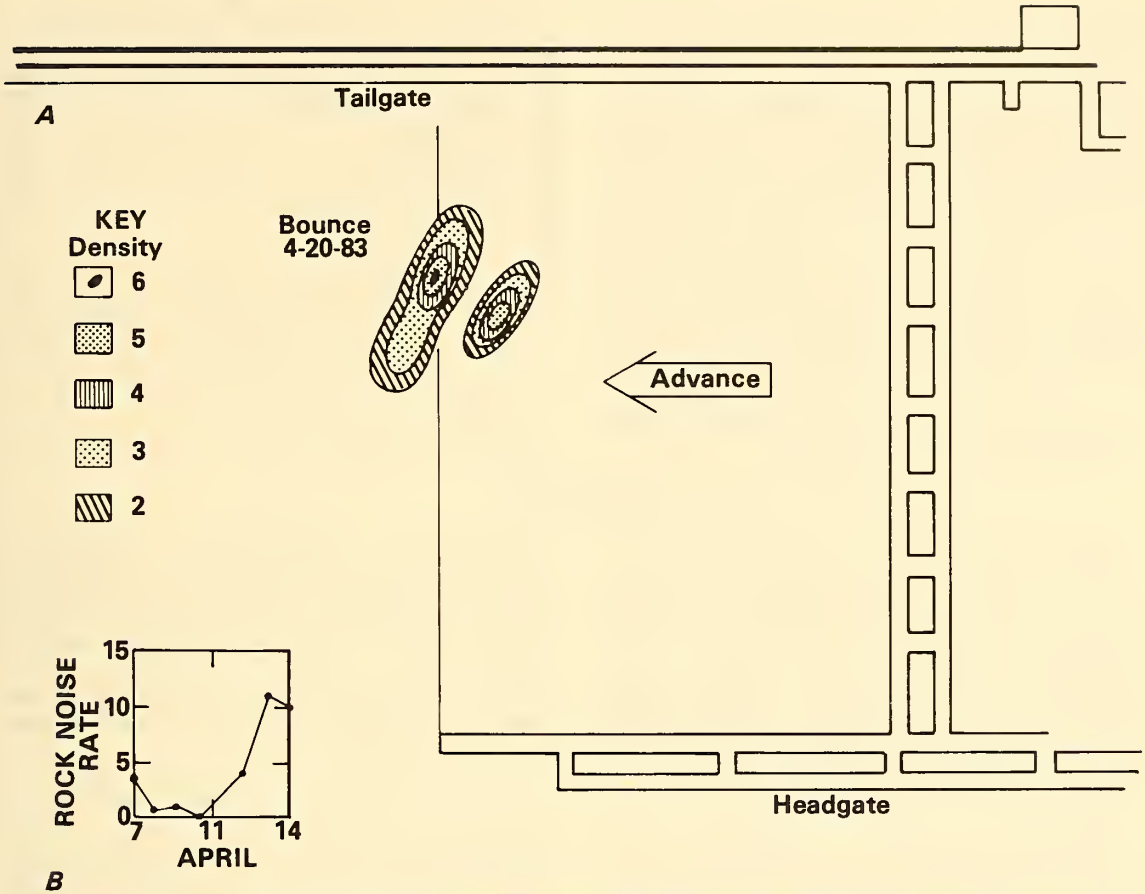


FIGURE 9. - Plan view of rock noise concentration and rate change before a coal mine bounce in a longwall section.

accuracy of rock noise locations is only valid to about a 40-ft limit. This proves sufficient for monitoring several stoping areas and provides information relavent to the whole of the stope pillar. This practice, called minewide monitoring, has been and is presently being used with moderate success around the world. This type of monitoring provides the potential to evacuate a specific working area before a burst, as can be seen from data such as shown in figure 10, a case history from the Star Mine, Burke, ID (15). It also provides an insight into general mine stability and an opportunity to watch the development of problem areas and to take remedial action to control or avoid failure. In the Couer d'Alene mining district of northern Idaho this type of analysis has successfully been used to determine when destressing techniques should

be initiated to control rock-burst-prone pillars (16).

Similar applications have been made in other parts of the world for both research and production. Research efforts have increased dramatically around the world within the last 5 yr, in both hard rock and coal mining. Mining companies have installed available systems to monitor and study specific problems, and in some instances are carrying out their own research efforts to study new problems. These new efforts, on many fronts, will have a major impact on the future of microseismic techniques in the mining industry.

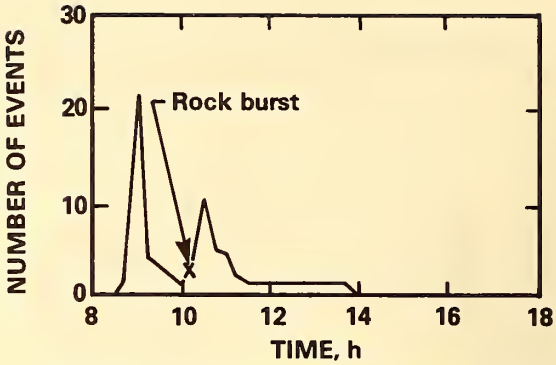


FIGURE 10. - Rock noise rate change prior to a rock burst.

SOME PROBLEMS

The microseismic technique has developed into a potential tool that can provide for vastly increased safety to mine personnel and can be used as an aid in production planning and problems. To demonstrate this, clear-cut and highly successful applications have been shown; however, problems and deficiencies of the present technique do exist.

Reliability is the foremost of the problems with microseismic techniques at their present level of development. Reliability does not mean equipment reliability, although microseismic systems do require constant maintenance and attention, but rather the fact than not all failures are predicted as easily or clearly as those presented, and sometimes indicated failures may not occur. This presents the user with the problem of determining how many times he is willing to be wrong. Experience has shown that the technique has undeniable potential and is beneficial in its present form when properly used. That same experience, however, shows that at times, false alarms will be sounded, and even worse, sometimes no alarm will be sounded when one was necessary.

The reliability problem cannot be attributed solely to improper use of microseismic systems or inadequate analysis of their data. The solution to the problem lies in further research to develop better and more accurate methods of data analysis, and possibly the incorporation of supplemental methods to be used in conjunction with the application of microseismic techniques. While not perfect, the positive aspects of the method generally outweigh the potential problems to the user.

CONCLUSIONS

The microseismic technique has been developed a great deal in recent years and has been used with moderate success to provide warning of failure in several different applications in the mining industry. Its use is growing and more potential users of the technique are becoming aware of its capabilities and are making efforts to use the technology. As mining continues to greater depths and stability problems become more intense, this use can only be expected to expand, and with expanded use, we can expect expanded technology and improved methods of utilization.

Current Bureau research, in both hard rock and coal mine environments, is aimed at establishing better utilization of the microseismic technique. Results continue

to indicate the viability of the technique and that its use can contribute substantially to both safety and mine design. The ability to delineate problem areas before failure is becoming better established, but the overall reliability of current techniques remains undefined, particularly with regard to providing warning of failure. The continued efforts of the Bureau are aimed at establishing the data base and experience necessary to evaluate present reliability problems and to improve the overall effectiveness of microseismic techniques applied to the problems of the mining industry.

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MECHANICAL AND ULTRASONIC CLOSURE RATE MEASUREMENTS

By Roger McVey[1]

ABSTRACT

The Bureau of Mines has constructed two intrinsically safe closure rate instruments that provide the mine operator a means for predicting an imminent roof fall during pillar robbing. This improves operator and machine safety and prevents delays in digging out equipment. One instrument system consists of two rugged retrievable extensometers connected by long electrical cables to a digital readout unit for reading closure and closure rate. Once a predesignated closure rate is reached, the extensometer is retrieved by pulling it from the imminent roof fall area by its electrical cable. The equipment and mine personnel are also pulled back to await the fall, which usually occurs within minutes after the designated rate is reached. Although the unit is primarily designed for retreat mining operations, it can be used for any activity requiring measurement of displacement or rate of displacement. Measurement range is 0 to 6 in with 0.1 pct accuracy for openings of 4-1/2 to 12 ft.

The Bureau is also evaluating a small ultrasonic unit to make these measurements. The new instrument provides unobstructing measurements up to 35 ft. The ultrasonic transducer can be attached to a roof bolt, tossed into an unsupported area, or handheld. Total distance and rate of change are displayed digitally to 0.001 ft.

INTRODUCTION

Roof control is a major problem in all aspects of underground mining, especially in room-and-pillar retreat operations. Room-and-pillar retreat mining is begun by developing a state of multiple entries. The coal pillars between the entries and crosscuts are then extracted in a retreat sequence and the roof allowed to cave in. The key is to mine as much of the pillar as possible, then remove both personnel and equipment before the final portion of the roof collapses.

An extensive study had been made earlier at the Southern Utah Fuel Company (SUFCO) No. 1 Mine by Hamid Maleki, Colorado School of Mines, and Doug Johnson, of SUFCO, in determining the critical roof-to-floor closure rate for predicting a roof-caving during retreat mining. They timed the roof-to-floor closure change to determine the critical rate of closure. The critical rate was determined to be 0.2 in/min. Roof-caving prediction based on this value was so successful that the mine suggested the Bureau develop an automatic closure-rate instrument.

Subsequently, the Bureau designed and built two types, mechanical and ultrasonic, of automatic closure-rate instruments that would digitally record both the rate of closure and the accumulative closure, and also provide audiovisual warnings when the closure rate reached a preset critical value.

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ACKNOWLEDGMENTS

The author wishes to thank Bob Ochsner and Tom Heaps of SUFCO for their help in testing the closure-rate instruments.

MECHANICAL INSTRUMENT

DESCRIPTION

A mechanical closure-rate instrument was built first. It consists of two rugged telescoping potentiometric extensometers and a digital readout-control box (fig. 1). The extensometer (fig. 2) is designed to accommodate a height of from 4-1/2 to 12 ft with a measurement range of 6 in. It is spring-loaded over this 6-in range. Long (100- to 125-ft) cables connect the extensometers to the readout box, permitting the operator to remain in a safe, supported area. A breakaway feature on each extensometer allows it to be pulled from the fall area by its electrical cable.

The operator watches the digital readout on the control box (fig. 3) during mining of the pillar. When a predetermined critical rate of closure is indicated, he retrieves the extensometer (fig. 4) and signals the miner operator to pull back. The control box (fig. 5) provides two digital visual readouts. One display shows the closure rate, the other total accumulative closure from time zero. The operator can preset any closure rate from 0 to 1 in/min. When the preset rate is reached, an alarm light illuminates and an audible alarm is sounded. The control box can monitor two extensometers individually or alternately. The system is battery-operated,

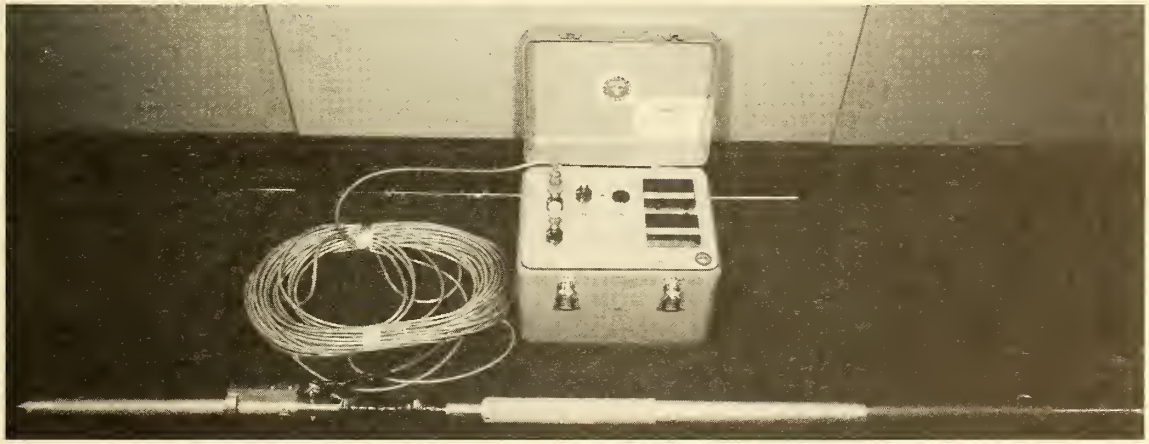


FIGURE 1. - Mechanical closure-rate system.

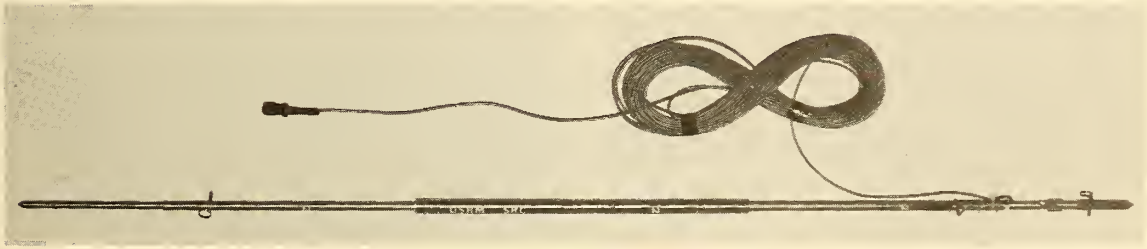


FIGURE 2. - Extensometer.

FIGURE 3. - Control box, on-site.

completely portable, and can be quickly moved from one place to another.

The closure-rate instrument, though primarily designed for retreat mining, can be used for any activity where knowledge of roof-to-floor closure rate or total displacement is required. The system provides a 0- to 6-in measurement range with 0.1 pct accuracy for both rate and total closure. Any extensometer installation displacement can be zeroed out with zeroing potentiometers. This zero value can be recorded and reset if multiple extensometers (greater than two) are used. The auto alarm can be set to any closure-rate value from 0 to 1 in/min, with a 0.01-in resolution and 0.1 pct accuracy.

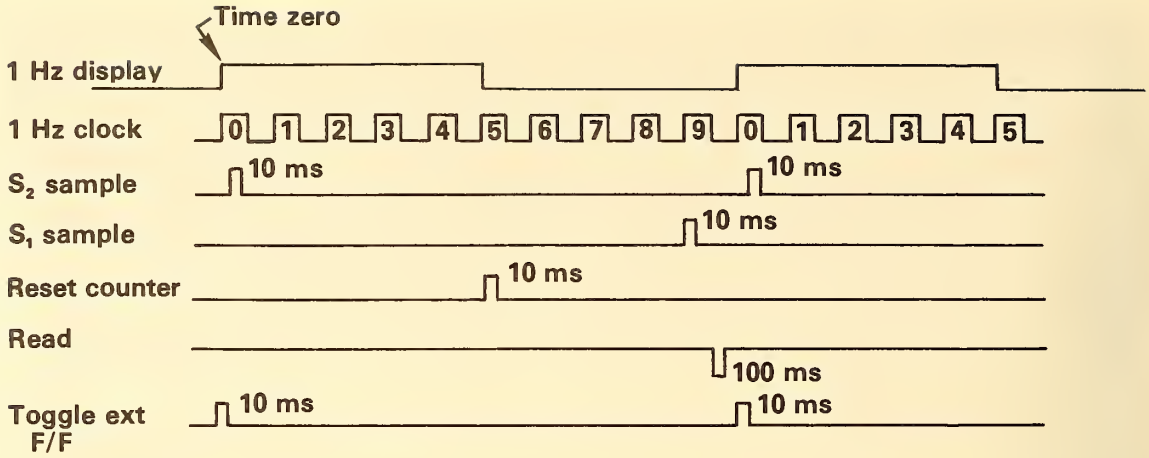
FIGURE 4. - Installed extensometer.

FIGURE 5. - Control box, closeup.

CIRCUIT DESCRIPTION

Figure 6 is a timing diagram for the operation of the electronic measurement circuits. Because of the small changes in actual rate, normally up to about 0.2 in/min, it was decided to use a 10-s sample period for greater accuracy. The extensometer is sampled at the beginning (S_2) and at the end (S_1) of a 9-s period by sample and hold circuits. The two 10-ms samples are compared, and any difference is converted to a directly proportional frequency, counted for 100 ms, and displayed as rate in inches per minute. A total read cycle consists of taking the two samples, resetting the counter, reading the value, and displaying it digitally. The extensometers are automatically measured alternately or can be continuously monitored on a singular basis. The extensometer circuit (if in auto) will automatically cycle to extensometer No. 2 with the extensometer toggle switch in auto position. When No. 2 has been sampled and displayed, No. 1 extensometer is automatically toggled back into the circuit.

Figure 7 is an electrical diagram of the readout-control box. Each



Note: Timing as follows

1. Extensometer 1 is selected by "Toggle ext pulse."
2. Extensometer voltage is sampled "S_2" and is held to be compared with S_1.
3. S_1 is sampled and compared with S_2.
4. Counter is reset ready to count.
5. Counter reads V to F count.
6. Count displayed for 5 s.
7. Extensometer 2 is selected by "Toggle ext pulse."
8. Same sequence of reading for Extensometer 2.
9. Extensometer 1 is selected by "Toggle ext pulse."

FIGURE 6. - Timing diagram.

extensometer electrical output is fed directly to a high-input impedance amplifier. The outputs from these amplifiers are routed to a field-effect transistor switch, which selects the extensometer to be measured. One route converts the extensometer voltage to frequency and is displayed as inches of total accumulative displacement. The same voltage is routed to the sample-and-hold circuitry, where it is sampled, summed, and converted to rate in inches per minute. This difference, or summed voltage, is also sent to the comparator for the alarm function.

INSTRUMENT SPECIFICATIONS

Measured range:	0 to 6 in displacement
Closure rate:	0 to 6 in/min
Accuracy:	+ 0.1 pct
Resolution:	0.001 in
Alarm:	0 to 1 in/min with 0.01-in resolution
Readout:	Digital LED display: two; one for total displacement; one for closure rate.
Power:	Battery power 12-h capacity. Intrinsically safe (MSHA approved).

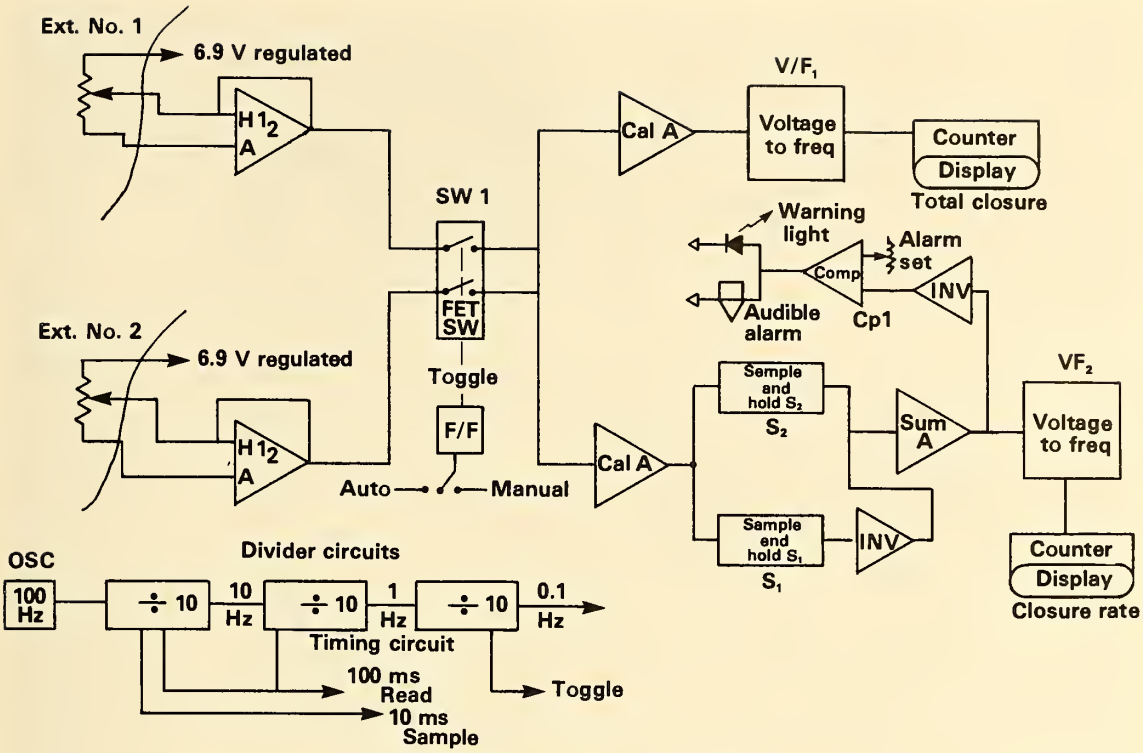


FIGURE 7. - Block diagram of closure-rate instrument.

ULTRASONIC INSTRUMENT

A nonobstructing method of measuring roof-to-floor closure has always been desirable in underground ground-control measurements. The mechanical extensometer has several drawbacks, such as high cost and being hard to use in high-traffic areas. Thus the Bureau undertook a project to determine the feasibility of using ultrasonics for convergence measurement underground. The new Polaroid sonar camera transducer and ranging system became a prime candidate. A small ultrasonic transducer of this type could provide an inexpensive, nonobstructing means of measurements.

The Bureau has thus far developed a small, yet inexpensive, handheld ultrasonic unit, shown in figure 8, that can measure distances from 1 to 35 ft ±0.02 ft. The unit is excellent for general survey work, measuring high roofs, etc.

A small, portable closure-rate measuring device was also built with an ultrasonic remote transducer that can be placed up to 100 ft or more from the readout instrument. This unit is shown in figure 9. This instrument reads both distance as well as rate of closure. It has a measurement range of 1 to 35 ft ±0.01 ft with 0.001-ft resolution. The first reading displayed is the distance. The rate is displayed 6 s later. A thumbwheel switch allows for setting an alarm limit for rate of closure. A visual and audible alarm is provided. The alarm limit can be set from 0 to 9.9 ft/min.

Because the ultrasonic instrument is still in the design and testing phase, and since permissibility approval has not been received from MSHA, its electrical schematics and operational information will be made available at a later time.

FIGURE 8. - Portable handheld ultrasonic unit.

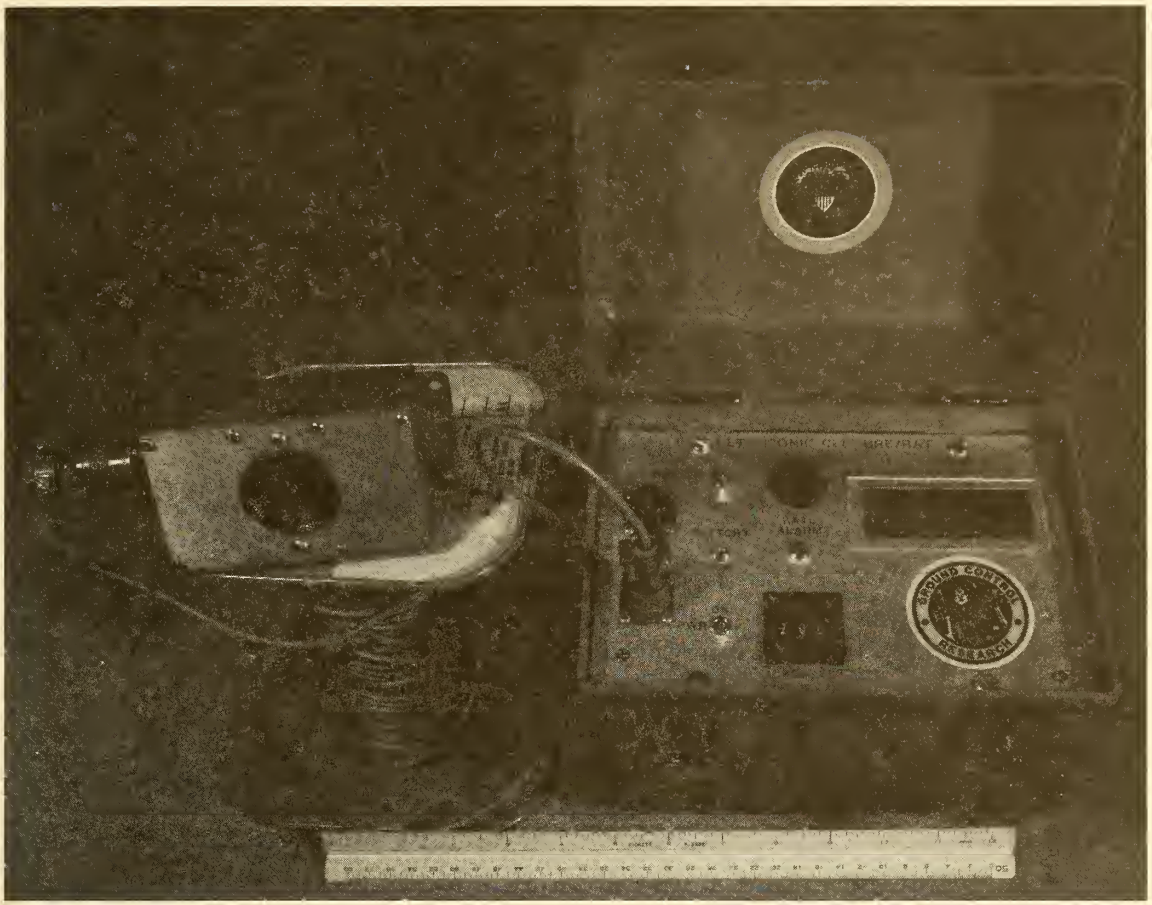


FIGURE 9. - Portable ultrasonic closure-rate instrument.

FIELD TESTS

In June 1981, the mechanical closure-rate instrument was placed for field testing in SUFCO's No. 1 Mine (figs. 3-4) near Salina, UT. The test results have exceeded expectations, and roof-caving predictions have proven very accurate. In most cases, the instrument has warned of an impending roof-caving within minutes of the event. It was reported that use of the closure-rate instrument has, in general, led to increased coal recovery and productivity.

The readout instrument, except for a fuse blown during battery replacement, has been trouble-free. A design change in the breakaway mechanism and substitution of heavier gauge extensometer rods are the only modifications made to the system thus far. It is noteworthy that, to date, no extensometers have been lost during caving.

CONCLUSIONS

With 22 months of field tests we conclude that the mechanical closure-rate instrument appears to be a viable tool for roof hazard prediction in retreat mining operations once the critical closure rate has been determined for a particular mining location. Most retreat cycles, in general, have increased tonnage per production shift.

Preliminary test results of the ultrasonic closure-rate unit appear excellent. It is hoped that this unit will also improve safety underground.

GROUND INSTALLATION EQUIPMENT

REMOTE MANUAL ROOF BOLTERS

By John E. Bevan[1]

ABSTRACT

Coal mine accident statistics show that 18 pct of all roof-fall fatalities involve roof bolting. This is 30 pct higher than for any other occupation category. Industry analyses show that roof bolters were involved in 15 pct of lost-time accidents. Rapid placement of permanent roof support appears essential to safety as well as long-term roof stability. This paper investigates one method for placement of permanent roof support.

BACKGROUND

The MESA report, "Analysis of Fatal Roof-Fall Accidents in Coal Mines, 1972-1975," indicates that 18 pct of all roof-fall fatalities involved roof-bolter operators and helpers. This group of workers experienced 30 pct more fatalities than any other occupation category.

The report "Injuries Associated With Roof or Rib Bolting and Bolting Machines in Underground Coal Mines, 1978-1982" analyzed 5,777 underground coalmine bolting or related bolting machine injuries reported to HSAC from 1979 through 1982. The results were--

1. Drilling of the roof or rib accounted for 2,457 injuries (45 pct).

2. Installation of bolts in roof or rib--1,634 (28.3 pct).

3. Tramming the machine--727 (12.5 pct).

4. Unknown (due to insufficient data to classify)--959 (16.6 pct).

Industry analyses of lost-time accidents are not complete, but the following figures are typical. One company's Safety Department reports that roof-bolt operators were involved in 15 pct of their lost-time accidents. Of the roof-bolting accidents, 19 pct were caused by roof falls, and 48 pct were caused by being struck or caught by rotating tools.

The above information indicates the danger involved with roof bolting and roof control. Many other non-roof-bolter injuries and fatalities are also a result of insufficient roof support. Rapid placement of permanent roof support also appears to be beneficial to mid- and long-term roof stability as well as immediate safety.

RATIONALE

The Bureau of Mines has addressed these problems through two areas of research: (1) by removing the bolter operator from the immediate dangers of bolt installation; and (2) by providing means for a safer and more timely bolting system.

A roof-bolt inserter (RBI) developed allowed a longer-than-seam-height bolt to be installed by bending the horizontally carried bolt into the vertical orientation of the mine roof-bolt hole. A longer-than-seam height (LTSH) drill or flexible roof drill developed by contract allowed long holes to be drilled in low coal. The RBI and LTSH drill add flexibility of package design, which

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allows the operator to be removed from the immediate bolting area, under supported roof and away from the dangers of rotating and moving equipment. The operator can be placed outby the last permanent row of roof support and/or under canopy protection. Here the operator can be protected while still using his or her abilities.

An automated miner-bolter to use the concept of mounting the two systems on a continuous miner would allow one-pass or truly continuous mining.

However, many problems of an automated system may be caused by the substitution of the operator's function with mechanical-electrical-hydraulic systems and artificial intelligence. Manual dexterity, memory, logic, audio, visual, and sensory capabilities of the operator have been replaced in an attempt to remove him or her from the dangers of the immediate bolting station. It appears that any viable system must be greatly simplified; hence, replacing some of the capabilities of the operator is unjustified and unwarranted if the operator can supply these functions while being protected.

Six concepts featuring hands-off drilling, remote control and/or automated sequencing, and improved productivity features were developed. From these six concepts and initial designs, the articulated remote manual roof-bolter (ARM bolter) and the remote-manual bolter (REM bolter) were chosen for further development.

The ARM bolter (fig. 1) is a roof bolter for low seams, which enables an operator to perform bolting functions while under a canopy protection and permanent roof support. The operator sits in a reclined position in a cab with his head approximately 8 ft from the bolthole location. From this location, the drilling, bolting, and torquing of bolts are controlled. Bolts ranging in length from 4 to 8 ft are fed into the bolter component assembly. The machine's overall tram height is approximately 33 in and it is capable of installing bolts in seams ranging from 37 to 60 in. The limitation in seam height is due to bolter design and not necessarily limited by conceptual considerations.

The heart of the ARM bolter is the bolter component assembly. This assembly houses a flexible roof drill, roof-bolt inserter (RBI), torque thrust assembly, plate magazine, feed and receive mechanisms, and a component carriage to house

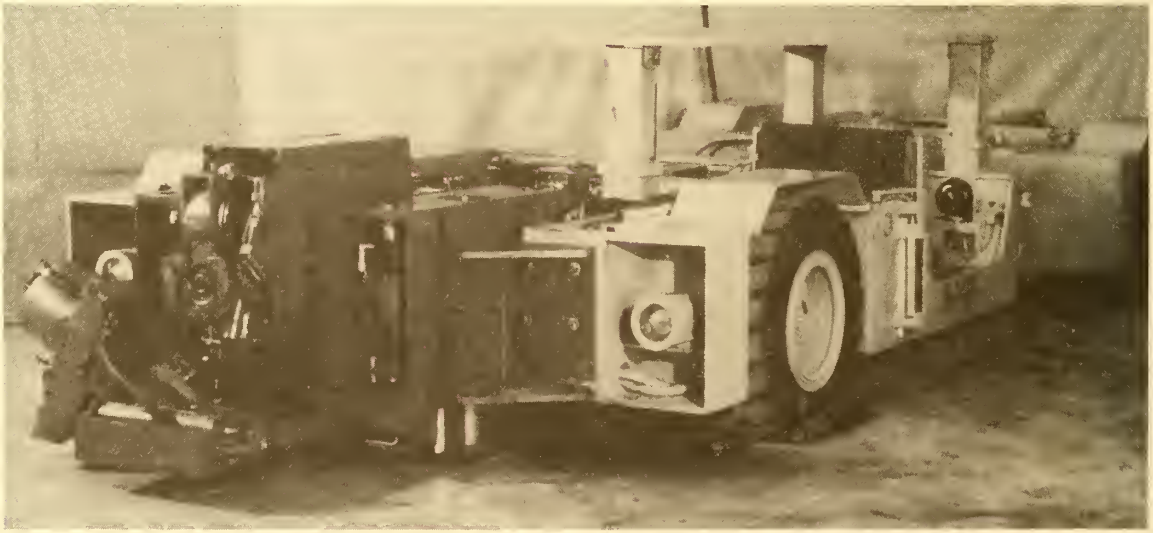

FIGURE 1. - Articulated remote manual (ARM) roof bolter.

the above elements. The bolter component assembly interfaces with the front end of the articulated vehicle and is raised and lowered by means of two elevation assemblies. The operator manually assembles the mechanical bolt and anchor, then loads the assembled bolt (less anchor plate) into the feed and receive mechanism prior to drilling the hole. The anchor plates are loaded into the plate magazine at the same time. The operator enters the operator station (under canopy support) and trams the bolter to the proper bolting position as dictated by the mine plan. The bolter component assembly is raised to the roof, and the bolt hole is drilled by the longer-than-seam-height drill. After the hole is drilled to the desired length, the drill string is retracted and the drill is indexed away allowing the RBI-torquer assembly to be indexed to align with the drilled hole. The plate feed assembly installs the plate on the roof bolt, then the RBI installs the bolt into the hole. The RBI expands allowing the torquer to engage, insert, and torque the bolt assembly. The RBI and drill are indexed to their stow position, the bolter component assembly and roof jacks are lowered, and the ARM bolter is ready to tram to the next bolt installation. The control is a combination of air logic control and remote operator control. Remote operator control is available in the event of an air logic system failure. These operations, done during bolt installation, are achieved while the operator is under canopy and permanently supported roof.

The second concept being developed (fig. 2) by the Bureau is the remote manual bolter (REM bolter). This concept uses items from the longer-than-seam-height drill program, but previous problems with automated modules directed attention toward enchancing, rather than replacing the operator. Many of the same design criteria used for the ARM bolter were used in the development of the REM

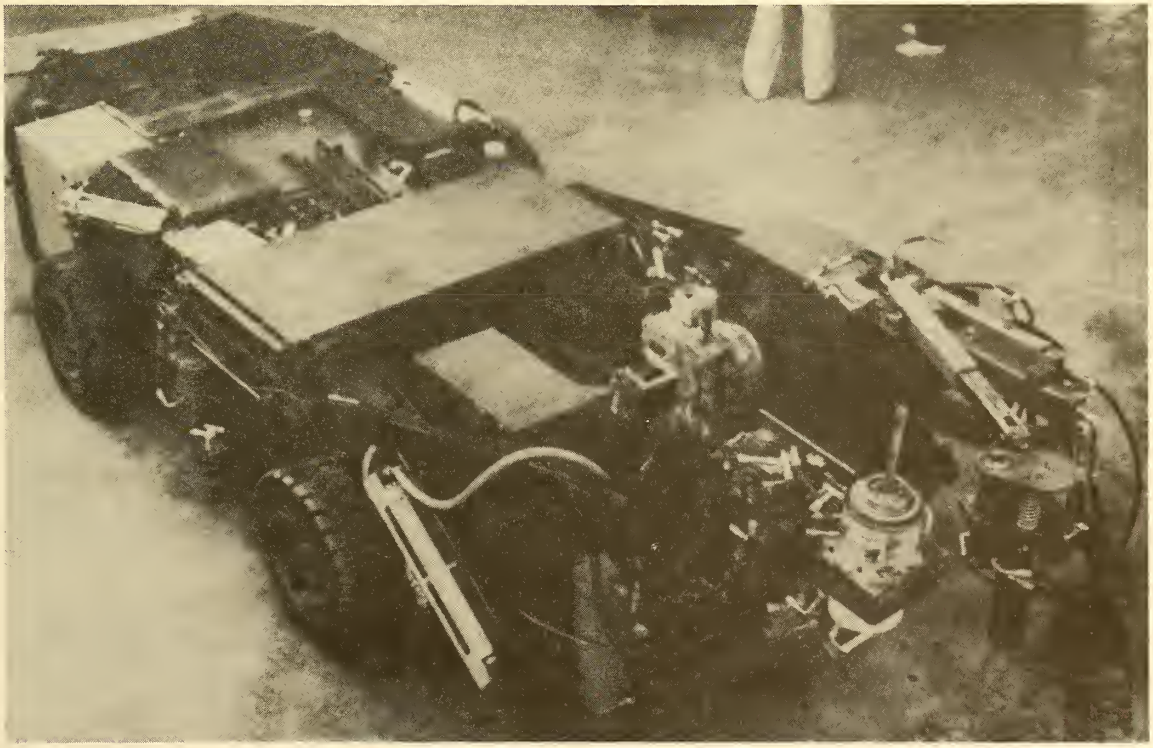

FIGURE 2. - Remote manual (REM) roof bolter.

bolter. The operator is used more in the REM bolter concept than the ARM bolter (attempting to make a simpler system). The same steps are required to install the bolt, but here the operator is responsible for assembling the complete bolt. Any anomaly must be compensated for by the operator.

The REM bolter is trammed into position, and the drilling is initiated. The RBI is mounted on a track which allows it to slide back near the operator where the assembled bolt is manually placed into the RBI. When the drilling cycle is completed, the drill is indexed to its stowed position, and the RBI is run forward on the track and indexed under the drilled hole. Final positioning of the RBI, if required, can be done by moving the bolter or moving the arm that carries the RBI. The RBI then inserts the bolt into the hole and is indexed to its stow position. The torquer is indexed into position and coupled with the bolt. The bolt is torqued, the torquer is stowed, the floor jack is released, and the bolter is ready to be trammed to the next bolting position. The operator has not moved from his protected position.

All control functions of the REM bolter are operator-controlled. No sensory feedback or logic circuit, lockout, etc., is employed. This is done for simplicity and its associated reduction in cost, weight, and size.

The future for remote bolters should hold great promise. The remote-manual concept can easily be adapted to resin bolts or a combination of mechanical and resin. Water-jet-assisted flexible drills could increase the number of roof conditions where the bolter can be used. (Water-jet-assisted flexible drills may replace rotary-impact drilling now used on severe roof conditions). New concepts like the inorganic grout injection devices could be easily adapted.

RESULTS

A 4-month underground test of the REM bolter began February 25, 1983, at a mine near Daisytown, PA. A soft band of shale in the mine roof resulted in flex drill problems. The dust system appeared to be plugging, which resulted in impacting the drill string in the hole. Another problem area was the operator's difficulty in inserting the bolt and coupling the torquer to the bolt head after it was inserted.

Another working section was made available by mine personnel and initial drill problems did not reoccur. Bolt installation of 50 holes per shift has been achieved.

The ARM bolter has been tested for a total of 5 weeks in a West Virginia mine. A total of 163 bolts have been installed. No major problems were encountered; however, some correctable problems have been encountered with the drill, cycle time, and dust collection system.

CONCLUSIONS

Both the ARM bolter and the REM bolter address the same problems, but the path taken by each is different. Some of the questions to be answered are:

1. What degree of automation or semi-automation is required?

2. Can the operator develop the needed skills to couple and uncouple the torquer to the bolt or can a control system do it better?

3. Can the operator find the hole to insert the bolt, and can it be done in varying seam heights with undulating top and/or bottom?

4. Will the operator be fatigued or will active participation result in a safer, more alert operator?

The program goal for the REM and ARM bolters was to develop a concept that

will protect the operator, be economically feasible, and be implemented by industry into a production machine. The program development and subsequent testing has shown the concept as viable. Automated bolters were costly and complex. The remote bolters are less expensive and much simpler while maintaining operator safety and high production levels.

Future generation machines will be more efficient and probably less costly. The degree of automation that can be effectively used in the commercial underground mining environment will increase as the state-of-the-art in robotics, controls, software, etc., increases, but any successful machine must be designed around the human operator, still the most important element of any system.

FIELD TEST OF AN AUTOMATED TEMPORARY ROOF SUPPORT (ATRS) USED ON A LOW-COAL, SINGLE, FIXED-HEAD ROOF BOLTING MACHINE (SQUIRMER)

By Charles T. Chislaghi[1] and Thomas E. Marshall[2]

ABSTRACT

An economical, remotely operated (automated), temporary roof support (ATRS) has been developed by the Bureau of Mines for use on a single, fixed-head roof bolting machine (squirmer) that operates in low-coal seams (<42 in thick). This ATRS eliminates the need for workers to go under unsupported roof to set or remove temporary support prior to or during the roof bolting cycle--a task that annually accounts for approximately 20 pct of all roof fall fatalities. It can be adapted on any squirmer used in the U.S. low-coal fields. A prototype ATRS, designed and built at the Bureau's Pittsburgh Research Center, was field-tested at Imperial Colliery Co.'s Mine No. 20 in Eskdale, WV. The Mine No. 20 amended roof control plan, which requires the use of the Bureau's ATRS as temporary support during face bolting, has been approved by the Mine Safety and Health Administration (MSHA).

INTRODUCTION

A statutory provision of the Federal Coal Mine Health and Safety Act of 1969 states that "No person shall proceed beyond the last permanent support unless adequate temporary support is provided."[3] However, since the time the law was written, there have been no means available for squirmer operators and helpers to set temporary supports from under permanently supported roof. This provision was interpreted to mean "In areas where permanent artifical support is required, temporary support should be used until such permanent support is installed,"[4] and "Only those persons engaged in installing temporary support should be allowed to proceed beyond the last permanent support until such temporary supports are installed."[5] Annually, approximately 20 pct of all roof-fall fatalities involve miners who have gone beyond the last permanent support to set or remove temporary roof support prior to or during the roof bolting cycle.

Because of space limitations in low coal, not many ATRS's have been commercially developed for squirmers, although many different ATRS systems have been commercially developed for roof bolting machines used in high coal. Most ATRS's designed for squirmers create a situation that reduces or compromises the existing safety level, with a greater safety hazard to squirmer operators and helpers working inby the last row of permanent support.

Over 3,500 squirmers are in use today in southern West Virginia, eastern Kentucky, and southwestern Virginia, and approximately 60 pct of these have no ATRS, cab, or canopy. Moreover, low-coal mine operators and owners in West Virginia have a need for ATRS because West Virginia mine law requires that roof bolting machines used in working places of West Virginia coal mines be equipped with ATRS, regardless of coal seam height.

All design work and prototype fabrication was done by the Roof Support Group at the Pittsburgh Research Center. All fieldwork was done in the No. 2 Gas seam (36 to 42 in thick) at Mine No. 20 of the Imperial Colliery Co. in Eskdale, WV.

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[2]Engineering technician.

Pittsburgh Research Center, Bureau of Mines, Pittsburgh, PA.

[3]30 CFR 75.200.

[4]30 CFR 75.200-13(a)(1).

[5]30 CFR 75.200-13(a)(2)

ACKNOWLEDGMENTS

The Bureau acknowledges the cooperation it received from personnel of the Imperial Colliery Co., MSHA Mt. Hope Subdistrict, and MSHA Bruceton Safety Technology Center. Without their technical suggestions and assistance, this project could not have been completed.

DESCRIPTION OF ATRS

The Bureau of Mines ATRS is based on a modified and improved Lee Engineering design for a squirmer ATRS. It consists of a 10-ft-long, steel, wide-flange beam supported by two double-acting, telescoping hydraulic cylinders (fig. 1). A steel sleeve, mounted on the bottom center of the beam, is designed to fit over the top of the squirmer drill head. The ATRS is carried from place to place and row to row on the squirmer drill head, but is not an integral part of the squirmer during bolting. During bolting it is connected to the squirmer only by two hydraulic lines. Because the ATRS only weighs about 400 lb, the squirmer drill head and boom do not have to be rebuilt to carry it. Total cost of the beam and cylinders is approximately $1,800. In-house fabrication of the ATRS took 8 worker-hours.

The Bureau's ATRS design meets MSHA's general design requirements and West Virginia's design and operating requirements for such support. Both hydraulic cylinders supporting the ATRS have check valves to prevent sudden collapse of the ATRS in the event of a ruptured hydraulic line or broken hydraulic fitting. In addition, the ATRS hydraulic circuit contains an accumulator, charged by squirmer

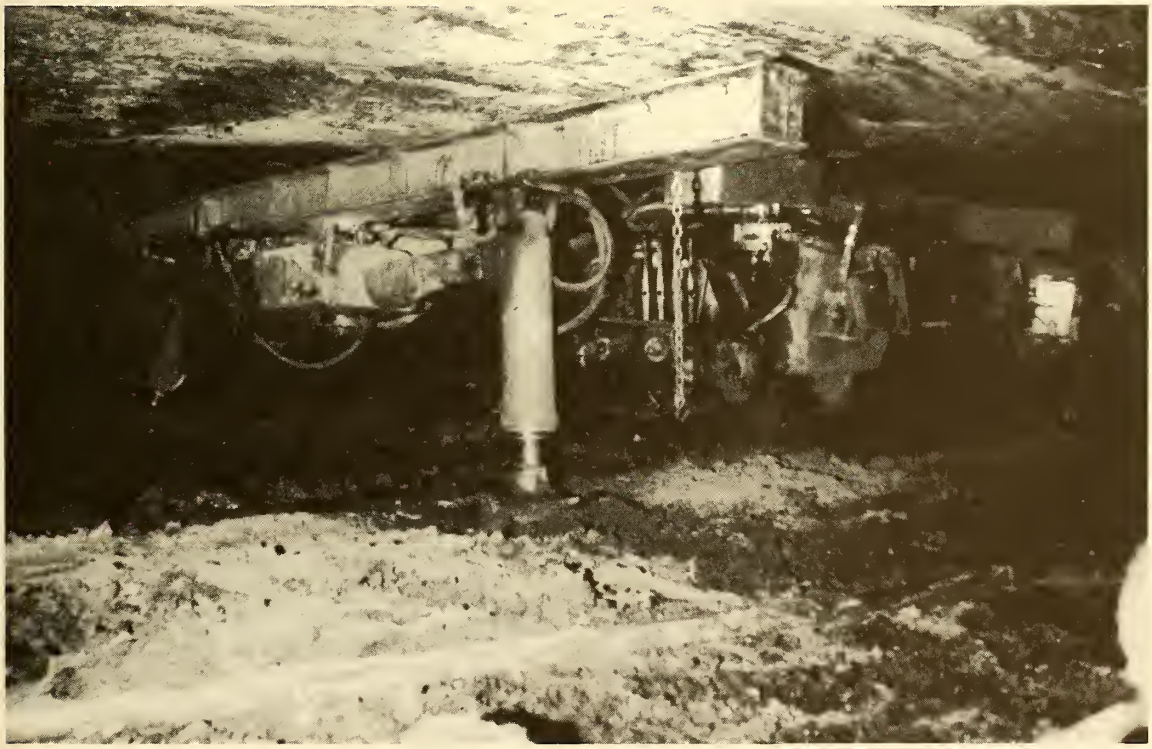

FIGURE 1. - Bureau of Mines ATRS.

line hydraulic pressure, which keeps the ATRS firmly set against the mine roof even if the roof rock is pulled up during the bolting cycle. The ATRS can elastically support the minimum required deadweight load of 33,750 lb; this capacity is certified by a professional engineer.

SQUIRMER STREAMLINING

Imperial Colliery personnel streamlined a 15-yr-old FMC model 300 squirmer for the field test. West Virginia State Mine Law requires the streamlining of any roof bolting machine before it can be retrofitted with ATRS. The ATRS controls were located 5 ft back from the drill head so that they can only be operated from beneath permanently supported roof. Inch tram controls were located at the drill station, and inch tram speed was reduced to 65 ft/min. Full tram controls were located with the ATRS controls, and the full tram speed was left at 150 ft/min. No ATRS controls were located at the drill station. Other streamlining work included removal of the bolt tray and tram deck, installation of low-value, high-torque tram motors, and moving the squirmer front wheels 8 in forward to provide space for the ATRS and full tram controls. Total cost of this work, which required 96 worker-hours, was $5,500. The Bureau piped the drill circuit to the ATRS and piped the ATRS circuit on beam at a cost of $150 and 32 worker-hours.

FIELD TEST AND RESULTS

With MSHA and West Virginia approval, Imperial Colliery placed the ATRS in the production cycle at Mine No. 20 for 5 months. Bolting was on 4-ft center, with 20-ft-wide entries and crosscuts. The cycle was the following:

Step	Description
1	The squirmer operator, at the full tram controls, trams into the center of a place and stops when the ATRS is under the last row of permanent support.
2	The operator lowers the drill head (and ATRS) to the mine floor using a boom control located beside the full tram and ATRS controls; moves from the full tram position to the beam (ATRS); and unhooks the hydraulic cylinder leg on the operator side that is chained to the beam, while the helper does the same to the leg on the right side.
3	The operator raises the drill head (and ATRS), using a boom control at the drill station, just high enough to let the legs hang down without scraping the mine floor; locks the legs perpendicular to the mine floor; moves back to the full tram and ATRS controls; and trams the squirmer inby without the legs scraping mine floor or the beam scraping mine roof.
4	The operator stops when under the last row of permanent support. The ATRS is now 5 ft inby the last row of permanent support and 5 ft from each rib.
5	The operator places the ATRS against the roof using the boom control located beside the full tram and ATRS controls, and then lowers the legs to the mine floor using the ATRS control until the beam is firmly set against the roof and the legs are firmly set against the mine floor.
6	The operator lowers the drill head away from the beam using the boom control located beside the full tram and ATRS controls.
7	At this point, the operator moves to the drill station, pushes in the diversion valve which diverts all hydraulic fluid from the full tram circuit to the inch tram circuit, and "inches" the squirmer to the left rib to begin bolting. During bolting the squirmer is connected to the ATRS by only two hydraulic lines.

8 After a row of permanent support is installed, the operator raises the drill head into the beam using the boom control at the drill station; pulls out the diversion valve which diverts all the hydraulic fluid back to the full tram circuit from the inch tram circuit; moves to the full tram and ATRS controls; and raises the legs using the ATRS control.

9 The operator lowers the drill head (and ATRS), using the boom control located beside the full tram and ATRS controls, just enough to tram the squirmer inby without the legs scraping mine floor or the beam scraping mine roof.

10 When under the row of permanent roof support that has just been installed, the operator stops and repeats steps 5 through 9. This cycle is repeated until the last bolt is in place.

11 Then the operator raises the drill head into the beam using the boom control at the drill station; unlocks the legs; pulls out the diversion valve; moves to the full tram and ATRS controls; raises the legs using the ATRS control; moves back to the drill station; lowers the drill head (and ATRS) to the mine floor using the boom control at the drill station; chains the leg, on the operator side, to the beam while the helper does the same to the leg on the right side; moves back to the full tram and ATRS controls; turns the squirmer 180°; and trams to the next place where steps 1 to 11 are repeated.

No operating or maintenance problems were encountered during the 5 months of testing. With the addition of the ATRS, the squirmer could still turn 180° within the 20-ft-wide entries and crosscuts and could tram through check curtains and line brattice without pulling them down. Comparative time studies of the same bolting crews showed that it took an average of 5 min less to bolt a place with the ATRS than with mechanical jacks. The bolting crews preferred the ATRS. After testing, an amended roof control plan requiring the use of the Bureau's ATRS during face bolting at Mine No. 20 was submitted by the Imperial Colliery Co. and approved by MSHA--District 4, Mount Hope Subdistrict, Montgomery Field Office.

CONCLUSIONS

The Bureau's ATRS eliminates the need for squirmer operators and helpers to go under unsupported roof to set or remove temporary support prior to or during the roof bolting cycle. The squirmer operator will always be under permanently supported roof while setting or removing the ATRS and will not be able to bolt inby the ATRS because of its control location. The inexpensive and lightweight ATRS does not reduce the squirmer operator's work space. It has the potential to be immediately used in some 70 pct of U.S. low-coal mines. Although the Bureau's ATRS was field tested only with the FMC model 300 squirmer, it can be adapted to the drill head of any squirmer operating in low coal and it can be fabricated in any mine shop. If a streamlined squirmer is available, the ATRS can be retrofitted to the squirmer during maintenance shifts, and if problems occur with the ATRS during the bolting cycle, it can be disconnected from the squirmer to allow bolting to continue with mechanical jacks. It has the potential to eliminate roof fall fatalities and injuries and may lead to increased productivity.

ROOF SUPPORT SYSTEMS

DEVELOPMENT OF EPOXY GROUTS AND PUMPABLE BOLTS

By Robert R. Thompson[1]

ABSTRACT

Good roof control is critical to the coal mining industry. The Bureau of Mines recognized the need for a remotely placed roof bolt of noncorrosive materials, which could be remotely installed in longer-than-seam-height lengths. The system developed has a fiberglass core. The core is made up of four 1/4-round sections, which can be coiled on the machine for storage and then formed into a hollow core during insertion into the drilled hole. The adhesive used is a fast-setting epoxy resin. The resin was tested underground in cartridge form. The test results showed bonding strengths greater than those available with the polyester resin now being used. Core handling and pumping equipment was designed, built, and placed on an existing roof bolter. The equipment is now being laboratory-tested and will be tested underground in a coal mine using a standard MSHA-defined, two-intersection test.

INTRODUCTION

Personnel safety is the first benefit of good roof control, but productivity can also be affected significantly by improved roof control procedures. Most of the easily mined coal has been extracted, and future mining will take place in areas with difficult roof-control problems. The Bureau of Mines recognized the need for a remotely placed roof bolt of noncorrosive materials that could be remotely installed in longer-than-seam-height lengths.

BACKGROUND

The system envisioned would consist of a hollow fiberglass-reinforced bolt and an adhesive that would be pumped into the annulus between the bolt and rock. The Bureau initiated work to develop such a system in 1978.

EARLY DEVELOPMENT

During the early part of the work, polyesters, urethanes, acrylics, epoxies, and inorganics (cement and gypsum) were examined for use as an adhesive. Polyester resins widely used today in roof bolting are supplied in cartridges that are used with steel bolts. The polyesters did not lend themselves to pumping because of resin instability and low flashpoints. Urethanes and acrylics were easily pumped and had fast set times, but were either too expensive or could not meet adhesive strength requirements. A coal-tar-based epoxy, one part resin to one part hardener, was developed which could be mixed in a static mixer and pumped by conventional liquid-handling equipment. Gel time requirements of the epoxy were met by preheating of the components. Cements had fast set times but unacceptable mixing and pumping characteristics. It was determined that the epoxy and cements were the most promising and would require additional laboratory testing during the second phase of the program.

[1]Research structural engineer, Spokane Research Center, Bureau of Mines, Spokane, WA.

Also, during the initial phase of the program, several shapes of fiberglass bolt cores were examined. The core shape requirements were such that the core could be coiled on reels for storage on the machine, then formed into a hollow core bolt during installation. Surface modifications on the smooth fiberglass-reinforced polyester (FRP) core to increase bonding strengths of the adhesives appeared necessary. A variety of special wraps and cloth meshes, fabricated on the outer surface of the smooth cores, delaminated under the severe pull strengths used to evaluate the system. The best method found was to cut a diagonal groove into the outer surface. This provided the mechanical interlock necessary to achieve the required strengths.

Initial designs led to two FRP bolt core shapes for laboratory testing. An axially pleated, flat-sheet material was tried. It could be stored in a flat coil and pulled as a "tape" to the placement head, which would fold it to form a hollow, cylindrical core. The second core configuration was based on discreet lengths of FRP, each a 1/4-round section which could easily be bent at the head, and the four pieces formed into a hollow core prior to insertion into the drilled hole. Again, it was determined that both bolt core concepts required additional laboratory testing during the second phase of the program.

PHASE II DEVELOPMENTS

The second phase of the program permitted more extensive laboratory testing of the core and grout candidates. Concrete blocks and test equipment were constructed to allow grouting and pull testing of both FRP bolt cores with epoxy and cement adhesives.

With the aid of several chemical companies, providing their own funding, inexpensive hardeners were developed which produced fast gel times without need for preheating. Additional laboratory work with the cements still produced unacceptable mixing and pumping. It was determined that the epoxy showed the most promise and that work with the cements would be terminated.

A series of four epoxy formulations was developed that met the 1-min gel times and early strengths needed over the required temperature range of 40° to 100° F. They had a 1:1 ratio of resin to hardener, were highly filled, and easily mixed within a static mixer. A twin cylinder, positive-displacement piston pump was used to meter, mix, and dispense the epoxy resin.

Both the flat-sheet and the segment core designs were adaptable for continuous insertion of longer-than-seam-height lengths. The folded sheet core tended to collapse on itself under cross-shear, while the segmented core design remained stable and was stronger in cross-shear. It was therefore decided to use the segmented core design.

Laboratory tests are continuing to verify that the segmented core design and the new epoxy systems meet or exceed all the strength requirements recommended by the Bureau.

EPOXY CARTRIDGE DEVELOPMENT AND FIELD TEST

Evaluation of epoxy grouts in mine conditions bypassed the pumping system development by packaging in cartridge form. Steel bolts with epoxy cartridges were installed in several mines under conditions similar to those used for polyester cartridges. In this way, fully grouted roof bolts using the epoxy formulation could be easily compared to existing support systems.

Pull tests on roof bolts with 1-ft point anchors were conducted off-section in the mine. Underground results

verified laboratory testing, in that the epoxy bolts could be installed as easily as the polyester and with greater bonding strengths.

A double intersection roof-bolt test was carried out in a freshly mined area (pretimbered) of a working Eastern coal mine. Over 200 epoxy-resin-grouted steel bolts were installed in the test without any installation problems. After a suitable return passage had been mined, the timbers were removed and the roof was evaluated for stability. The results indicated that the bolted strata were successfully supported and that the epoxy cartridges could be used as a viable alternative to the polyester system. Several private companies are pursuing the possible marketing of epoxy cartridges.

NEW PROGRAM

As a result of the laboratory and mine evaluations of the epoxy bolting system, the Bureau entered into a cost-sharing research program to develop the necessary mineworthy installation equipment to evaluate the concept of a remotely placed, pumpable, longer-than-seam-height, epoxy FRP, noncorrosive, roof-bolt system.

The contractor is furnishing a bolter chassis for the test period. Four major subsystems must be added to the chassis for placement of the pumpable roof bolts: the pumping equipment, purge system, fiberglass core-forming equipment, and a placement head for interfacing each of the subsystems with the drilled roof-bolt holes.

The contractor will incorporate, on the bolter, all the new design features shown to be needed. The compact pumping system design enables easy material loading and repair from the rear of the bolter. The resin will be pumped by two piston pumps which are driven by a cylinder mounted between the pumps. The system also includes a static mixer, valves, and hoses.

Laboratory testing indicates that a high-pressure inexpensive water purge completely cleans the static mixers.

The bolter system design includes a three-segment 3/4-in-diam core, three-reel storage, and handling system. The three segments of core will be pulled from the storage reels and driven through rollers designed to form a 3/4-in hollow core bolt when the three segments arrive at the drilled hole.

The placement device is designed to bring together the epoxy dispenser, purging system, and core former into a common head where the components can be prepared for roof bolting. This head will interface directly with the mine roof to provide multiple composite bolt placement.

FUTURE PLANS

During the next 4 months, the equipment will be laboratory tested. The results will be reviewed and systems redesigned as needed to ensure a meaningful field evaluation.

Field evaluation will be conducted off-section in a coal mine. When all involved are satisfied with the system, a standard MSHA-defined, two-intersection test will be conducted in a cooperating coal mine.

DEVELOPMENT OF LIGHTWEIGHT HYDRAULIC SUPPORTS

By John P. Dunford[1]

ABSTRACT

The installing of temporary roof support is an integral part of underground coal mining. The most common forms of temporary supports are wooden posts and metal jacks. Both wooden posts and nonyielding steel jacks are heavy and cumbersome to install. Yielding hydraulic jacks, while easier to install and more functional, are extremely heavy. With this in mind, the Bureau of Mines studied ways to reduce the weight of contemporary hydraulic supports without sacrificing performance.

In 1979 a contract was awarded to design and test a lightweight hydraulic mine support. Laboratory testing indicated some changes were needed in the basic design selected. After these modifications were made, 33 units were sent to the field at three different locations. These units were designed for use in a 6- to 8-ft seam, have a 22-ton capacity, are fully self-contained, provide a 5- to 7-ton roof preload, and yield to overload. The total weight of each unit is 55 lb, compared with 110 lb for a commercial steel unit. All three of the Western coal mines were extremely pleased with the units and requested to keep using them for as long as possible. During the field testing some problems did occur, such as corrosion on the piston surfaces, weak pump handles, and short life span of some internal seals. All of these problems were corrected.

During the field tests, the need became apparent for units that would function in seam heights other than 6 to 8 ft. Units of the same configuration and capacity were built and tested for seam heights in the 4.5- to 6-ft, 8- to 10-ft, and 10- to 12-ft ranges. After the testing proved successful, arrangements were made to install 10 of the 10- to 12-ft units in a mine for field testing. The results have not been completed at this time.

INTRODUCTION

The installing of temporary roof support is an integral part of underground coal mining. Historically, wooden posts, metal screw jacks, and, more recently, telescoping hydraulic supports are used for this purpose. Wooden posts are time-consuming and cumbersome to install, not consistent in size and strength, and also represent a sizable constant cost. Screw jacks, while easier to install, have no yield capability and are prohibitively heavy where high strength supports are required. The all-hydraulic, telescoping cylinder concept remains the best choice for reusable temporary support.

There are now on the market various hydraulic supports made of steel tubing and steel components that perform very well as a temporary support. Unfortunately, due to increasing support-load requirements and the thicker coal seams found in the West, these steel supports are becoming extremely heavy and cumbersome to use. A support rated at 22 tons and used in a 6- to 8-ft coal seam weighs in excess of 110 lb. In addition, the same type support designed for use in a 12-ft seam weighs approximately 300 lb.

With this in mind, the Bureau of Mines funded a contract to look at state-of-the-art technology in lightweight hydraulic cylinder design in an effort to construct a high strength-to-weight ratio yielding temporary roof support.

This work was started in 1978 and completed in February 1982. This paper deals with the evolution of the project through the contract and in-house phases.

[1]Mining engineer, Spokane Research Center, Bureau of Mines, Spokane, WA.

CONCEPT EVALUATION AND DETAILED DESIGN

The contract initially called for examination of previous work in the area of lightweight supports, as well as existing commercially available materials and products. The review identified the following major desirable designs features:

1. A 22-ton capacity.

2. A goal weight of 50 lb.

3. A prop with controlled yielding at overload.

4. A fully self-contained unit (i.e., integral reservoir and pressurization unit).

5. A remotely recoverable prop.

6. A prop easily and quickly installed.

Two concepts selected for detailed design incorporated hydraulic mechanisms that would yield at overload and be remotely recoverable. Of the two designs (totally hydraulic and hydromechanical), the all-hydraulic version was selected for prototype production and testing (fig. 1). Although the hydromechanical support was lighter in weight, it was rejected because of its mechanical complexity, cost, and complexity of installation in a mining situation.

FABRICATION AND TESTING OF LIGHTWEIGHT SUPPORTS

FABRICATION

Four prototype models of the all-hydraulic lightweight support were fabricated to verify the design. Based on value and manufacturing engineering considerations, some design changes were made prior to release for fabrication. Additional changes were found to be necessary during assembly and functional testing. Chief among these were modifying the pump block assembly and redesigning the main cylinder.

FIGURE 1. - Six- to eight-hydraulic support.

For structural testing, all four prototype units were modified. The four were laboratory tested to ensure functional stability and correctness of design. Two props were tested to ultimate load failure to verify column ultimate strength calculations.

Following the completion of successful functional and structural testing of the models, a production lot of 40 additional units was constructed (fig. 2). It was intended that 10 of these supports would be provided to each of four different mines for a 6-month demonstration and evaluation period.

Following the final assembly, each of the production units was subjected to operational testing. This consisted of pressure testing the internal preload limit valve, the 22-ton yield bypass valve, and preloading the unit in a test frame overnight to detect valve or seal leaks.

An independent testing program was performed by the Bureau to corroborate the

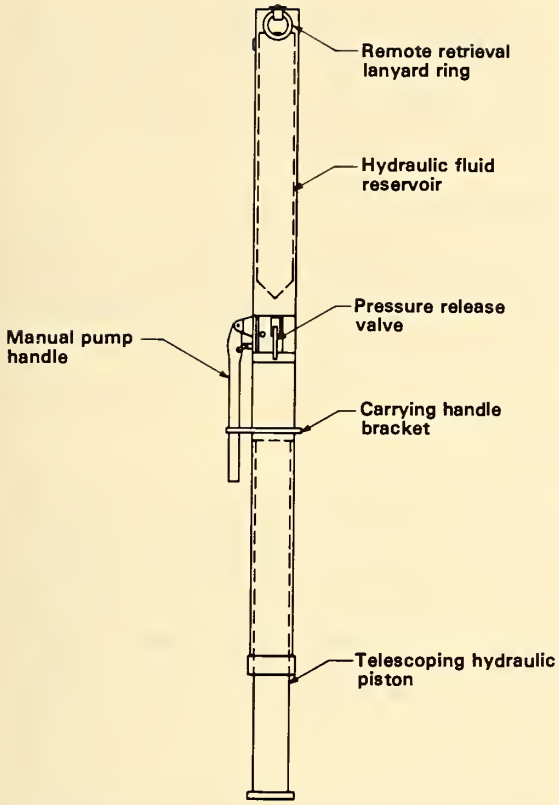

FIGURE 2. - Components of the hydraulic support.

contractors' findings. During these tests, one of the reservoir tube welds cracked and failed. Subsequent analysis by both the Bureau and FMC Corp.'s Materials Laboratory revealed that the weld was faulty. Other faulty welds were detected through radiographic analysis, so it was decided to reweld and reheat-treat all of the reservoir tube assemblies.

UNDERGROUND TESTING

During the latter part of the fabrication phase, underground coal mines were contacted to solicit interest in participation in the demonstration of the lightweight hydraulic supports. Three mines, two in Colorado and one in Utah, were selected to receive the props for underground testing. Each mine was given 10 supports to use under actual mining conditions.

One of the test sites in eastern Utah incorporated the supports into a continuous mining development section. The miners used the supports continuously, except for the 3-month-long United Mine Workers strike (six of the supports were left, fully loaded, across an entry for 3 months with no failures), and felt the lightweight supports were superior to what they had been using as far as handling and setting were concerned.

The supports were removed from the mine in November 1981 for repair. All of the handles were replaced with steel ones. Eight of the ten supports were rebuilt with cannibalized parts from two supports that had been hit by mining equipment and were beyond repair. All of the work was done locally and sent back to the mine for further use. The units were rebuilt again in June 1982. The six remaining units were then used in a continuously advancing development section as part of a prop and beam temporary support system. These were used until August 1982 when they were pulled from the mine for overhaul. Due to lack of production, it was decided to send the units back to the Bureau for rebuilding and inspection.

Another site, located in western Colorado, did not install the supports until May 1981. Eight units were being used as part of their longwall tailgate support plan. As of April 1982, the only problems encountered were one broken pump handle and one sticking release valve. After that date the longwall was shut down and the supports were used in various applications such as setting brattice curtain lines, setting chain-link fence at pillar ribs, and in longwall panel development work.

The supports were removed from the mine in November 1982 for overhaul. In March 1983, the units were again sent underground for use in a development section and also to be used as additional support around a drilling operation for a methane drainage project.

At the third test site, located outside Grand Junction, CO, the supports were

used in development work. In order to work in the 8-1/2-ft coal seam, a 2-ft steel extension was added to the support.

The supports were used in slow development work from April until November 1981, at which time most of the units needed some repair. Problem areas were failure of the main seal, sticking pressure release valve, and pressure pump piston. These supports were sent back to the Bureau for inspection and rebuilding.

OVERALL TEST CONCLUSIONS

In the three test mines, all comments about the test were positive. In all cases, both miners and management liked the supports and wanted to continue to use them. Although several areas of weakness became evident, they were not major problems. It should be noted that although the duty-cycle was only about 6 months, the problems exhibited were consistent with those props currently on the market.

CURRENT STATUS

All three test mines have expressed a desire to continue using the supports in their mining sequence. These mines and various other mines have requested information about commercial availability of the props.

A project to test the various lengths of lightweight supports is being performed by the Bureau, since the various lengths require slightly different designs. The first step in this project was to have prototype supports built using the updated shop drawing package. A contract was let to fabricate lightweight supports in the 4.5- to 6-ft, 8- to 10-ft, and 10- to 12-ft range. These supports were received in February 1983. A structural testing program to confirm design calculations was performed in late June 1983. At the same time, modifications are being made to the support, based on field test results, that will increase the duty-cycle and operating ability. Some of these changes included: redesign of the top end of the cap for more bearing surface and to facilitate nailing a cap piece on to the longer supports prior to setting the unit. Also, a redesign of the lanyard ring assembly will be performed along with plating the pump pressure piston and pressure release piston. Adding 3 in to the pump handle will help in applying the preload pressure. The use of various seals in the longer units will be investigated. An aluminum extension has been fabricated and is due for field testing in June 1983. Three units were fitted with sight pressure gauges and sent to Alaska to be used during a rock-mechanics project in a permafrost gold mine. Six other 6- to 8-ft units were fitted with pressure transducers and will be used in a retreat coal mining sequence to monitor loading in a breaker prop row.

Several of the longer supports will be fabricated and tested. Evaluations will be completed and results published in the near future.

MOBILE ROOF SUPPORT AND APPLICATIONS IN RETREAT MINING

By Robert R. Thompson[1]

ABSTRACT

Retreat pillar mining is highly productive, but dangerous. The primary danger during pillar removal is premature caving of the roof. The Bureau of Mines has developed a remotely operated machine that will place and retrieve temporary roof support. The prototype machine worked well but had several problems, the primary one being tramming. Two second-generation machines were built under a cost-sharing program with a Utah coal mine and a mining equipment company. The machine carries four 50-ton jacks and is remotely controlled by radio. The machines are presently being tested underground in a Utah coal mine.

INTRODUCTION

Retreat pillar mining is highly productive because supply, haulage, ventilation, and power systems are established, and there is also the advantage of the knowledge gained during development such as roof and ground behavior and hydrologic factors. The primary danger during pillar removal is premature caving of the roof.

The roof must cave in a predictable and dependable manner to prevent inducing excessive abutment loads in adjacent pillars, which can result in rib bursts, floor heave, or crushed pillars. The safety of the miners is dependent on successfully controlling the roof. Roof support during retreat is usually obtained by setting posts, cribs, hydraulic props, roof bolts, or a combination of these devices. They are set manually by a miner working in a hazardous area. Many of these devices can never be recovered and thus become part of the cost of extracting coal. The problem is how to set and retrieve these roof supports safely.

MOBILE ROOF SUPPORT SYSTEM

The Bureau embarked on a project to develop a system that would place and retrieve temporary roof supports without danger to the operator. In conjunction with a contractor, a mobile roof support (MRS) machine (fig. 1) was designed, built, and field-tested. THE MRS was remotely operated, battery-powered, and rubber-tired. It carried four jacks, two on the body of the machine, and two at the end of hinged arms. The jacks extend to form columns between the floor and roof, each with 30 tons of potential support. The jacks were hydraulically locked, and the load distributed to three points on each jack, without loading the machine chassis.

The MRS was tested underground in an Illinois coal mine. The prototype machine worked well but had several problems, the primary one being tramming over the soft floor. Tramming was slow and, at times, the tires became buried and machine had to be pulled out.

[1]Research structural engineer, Spokane Research Center, Bureau of Mines, Spokane, WA.

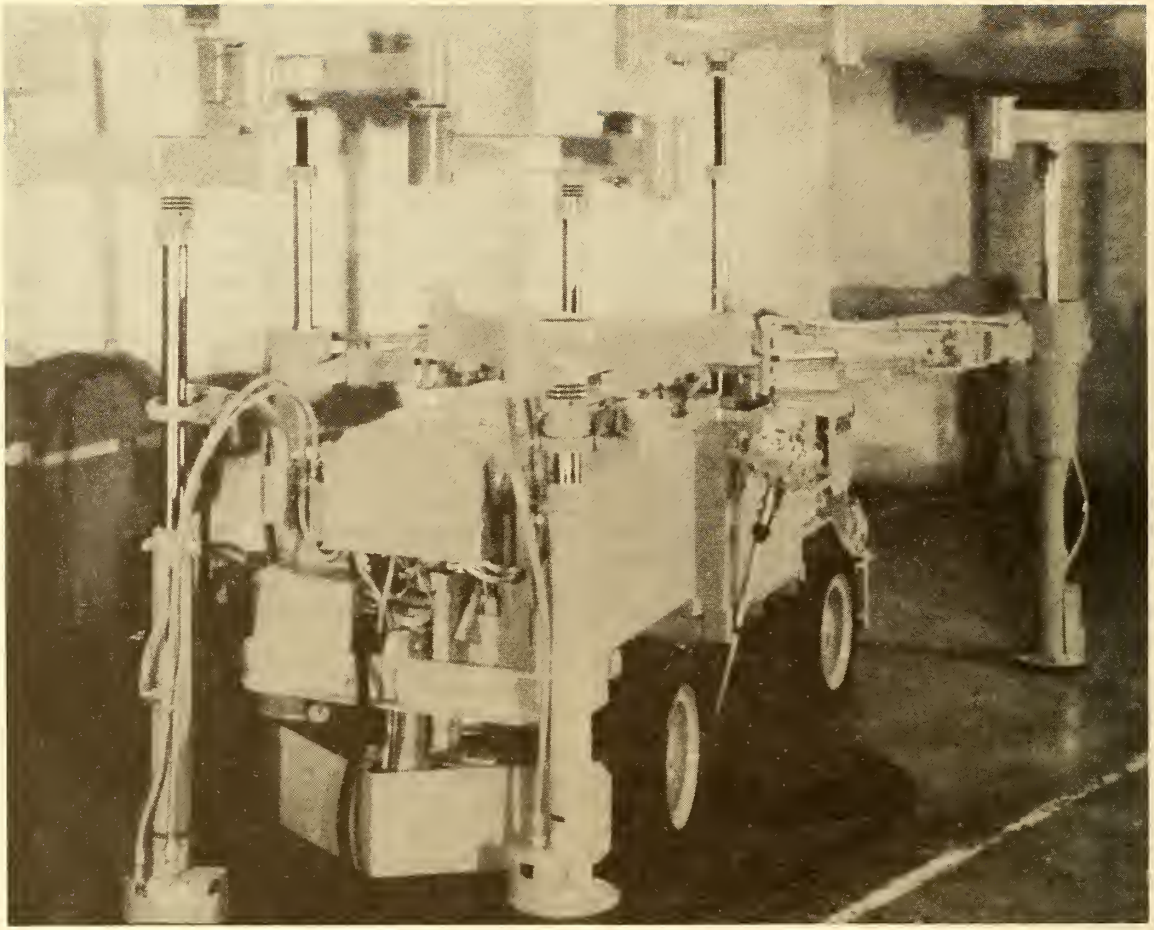

FIGURE 1. - First-generation mobile roof support.

SECOND-GENERATION MOBILE ROOF SUPPORT MACHINE

Results of the field test were encouraging. The concept had acceptance by both mine management and the miners. Several industry personnel witnessed the field trials and felt the MRS would improve the safety and productivity of their retreat mining sections. With this encouragement, the Bureau decided that a second-generation machine should be built, correcting the problems of the prototype.

A Utah coal mining company offered its mine as a test site. It also offered to work with the Bureau during the design and fabrication of the second-generation machine and to share some of its costs. This operation removed 12 ft of a 15-ft coal seam and had a four-member support crew setting posts. This time-consuming and hazardous task, at times, occurred under unsupported roof.

A mining equipment company became interested in producing the MRS. It offered to help cost-share by supplying some of the parts for the new machines. It also offered to assist during the design and fabrication.

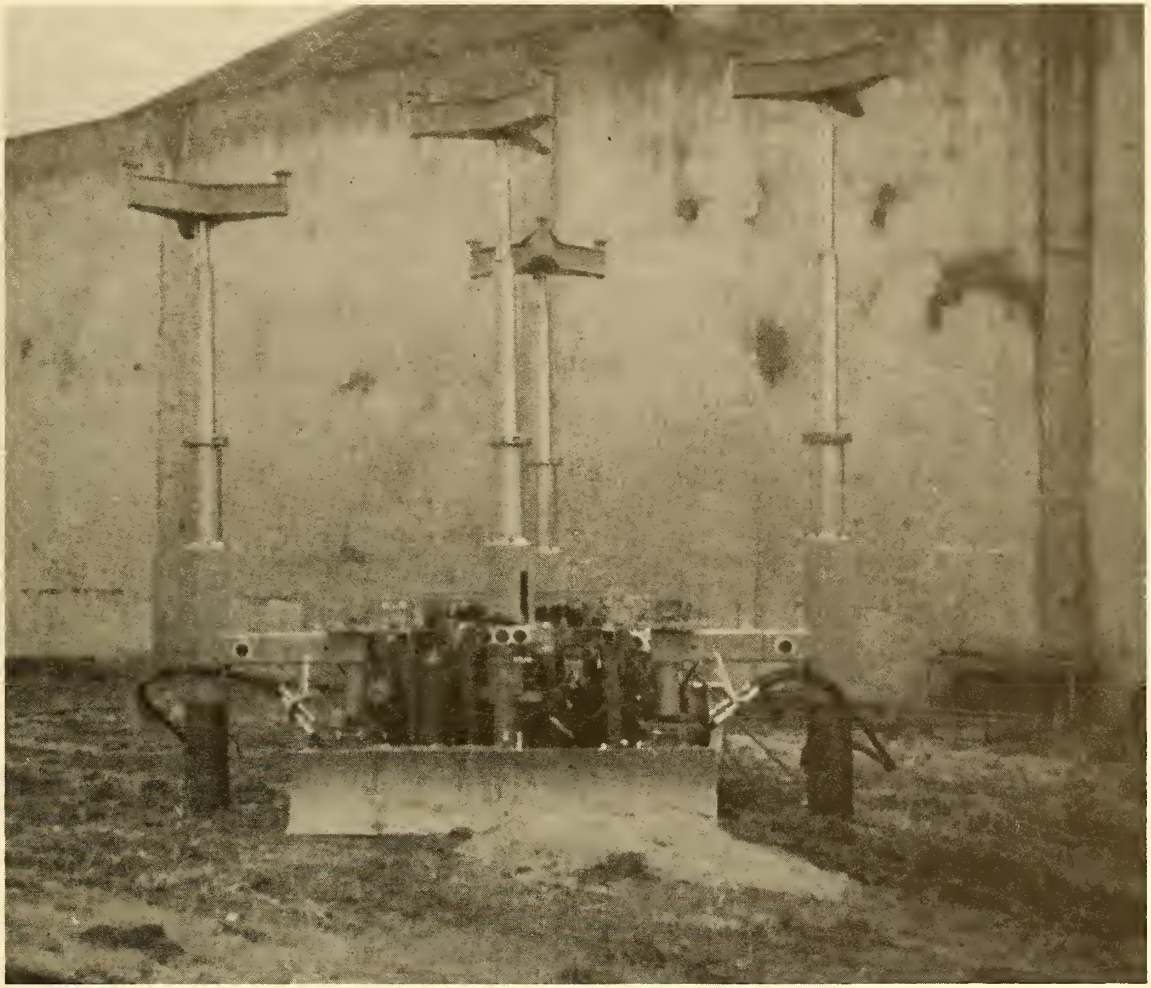

FIGURE 2. - Second-generation mobile roof support.

BASIC MACHINE REQUIREMENTS

The Bureau embarked on a research program to design, fabricate, and field-test two second-generation MRS's (fig. 2). During the first 3 months, participants met monthly. During this preliminary design phase, all participants agreed that the machine in the tram mode should be 8 ft wide and 10 ft long, or less, with at least a 12-in ground clearance. Also, it should be of rugged construction with towing hooks on both ends and mounted on independently controlled crawlers that exerted 20 psi, or less, ground pressure. The machine should be powered by a 40-hp, 460-V ac permissible motor from a 260-ft reeled trailing cable and transmitter remote controls. Reversible variable tram speed of at least 80 ft/min on 20 pct grade, and freewheeling for emergency towing, were to be included. Front and back dozer blades of 12-in range are required.

The machine was to carry two chassis mounted and two swing-arm jacks of 7- to 15-ft working height. The jacks were to have a 3- to 8-ton installation and 50-ton maximum loading capabilities. The

swing jacks are to form a breaker row with 6-ft separation between chassis jacks and have a visual load indicator. In case of heavy ground, the ability to remote-jettison either or both swing jacks is incorporated. In order to attain rapid egress under bad roof conditions, the jacks will be capable of retraction, so as to provide 1 ft of ground clearance and 1 ft of roof clearance in 30 s. The mine requested that the machines be remotely controlled by radio.

UNDERGROUND TESTING

After the machines are fabricated, they are being shipped to the mine to be tested for a period of 6 months. Figure 3 shows the test mine's ground control plan during full-pillar extraction. It required setting 24 posts for pulling each fender. The use of the two machines will eliminate the requirement for setting these posts. Figure 4 shows the planned sequential operation with the MRS. Again, the machine will be moved and set remotely, thus eliminating exposure of the miners setting the posts by hand. Yet to be determined is the possible use of some posts to act as "squealers" or warning devices. These posts, if required, would be set after the MRS is in place and supporting the roof.

The mine is expected to use the machines for a period of up to 10 yr, thus providing long-term testing. The projected mass-produced costs of the machines, with tethered remote-control rather than radio, are estimated at $125,000. The radio remote-control is expensive and is considered as an extra.

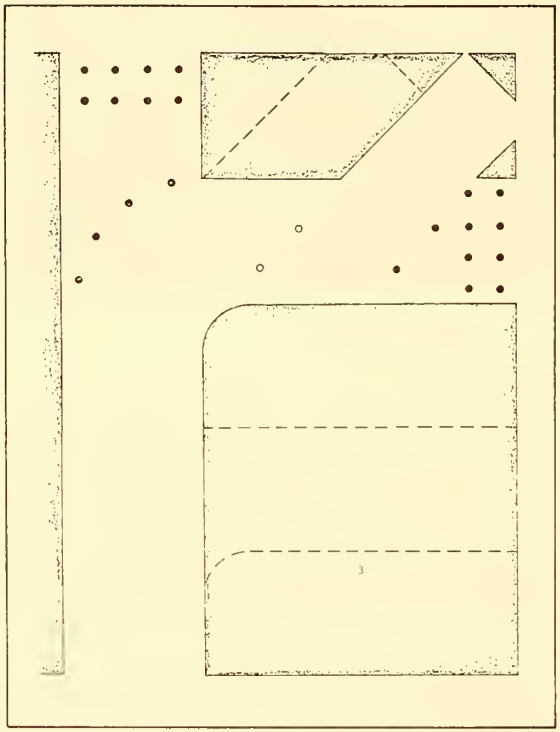

FIGURE 3. - Ground control plan during full pillar extraction.

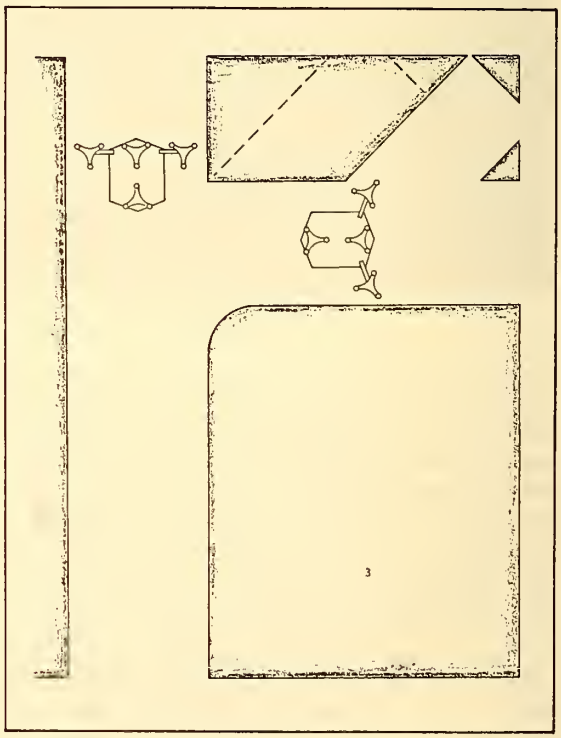

FIGURE 4. - Sequential operation with machines.

ROOM-AND-PILLAR RETREAT MINING

The Bureau published a manual for the coal industry,[2] which is to provide mine managers and engineers with:

1. Assistance in making decisions to retreat mine and in selecting the best mining technique for their specific condition.

2. Information on efficient retreat mining design.

3. Information to develop a section foreman's handbook on retreat mining safety and operation.

Copies may be obtained from the Superintendent of Documents, U.S. Government Printing Office, Washington, DC 20402.

SUMMARY

The Bureau has designed and built a support system for retreat mining that can be set and retrieved remotely. The system is now being tested in a Utah coal mine. This system provides added safety for the miner, by eliminating the need to work in a hazardous area setting posts, cribs, or hydraulic props. The MRS will also increase productivity, since the number of manual support setting operations has been decreased.

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[2]Kauffman, P. W., S. A. Hawkins, and R. R. Thompson. Room and Pillar Retreat Mining. A Manual for the Coal Industry. BuMines IC 8849, 1981, 228 pp.

INORGANIC GROUTS FOR ROOF BOLTING

By Jack E. Fraley[1]

ABSTRACT

The Bureau of Mines investigated rapid-hardening material substitutes for the resin used in mine roof bolts. Gypsum plasters ($CaSO_4 \cdot 1/2H_2O$) were selected because they have high early strength while being readily available and inexpensive. Gypsum plaster-water capsule cartridges provide a substitute for resin cartridges. Gypsum plaster holds promise for injection in roof bolt holes as a premixed slurry because of improved operator safety and greater economy.

INTRODUCTION

Fully grouted resin bolts are a relatively new phenomenon to the mining industry. In 1972, the advantages of resin bolts became apparent, and by 1980, an estimated 20 million of these bolts were installed. Since resin bolts are more costly than mechanical bolts, their superior performance is illustrated by the large increase in their usage.

The price of resin doubled between 1973 and 1975. Since resin is petroleum-derived, its cost and future supply are uncertain. Because resin cartridges are flammable, they are a potential underground fire hazard.

To overcome these resin disadvantages, the Bureau of Mines started to investigate rapid-hardening material substitutes. Gypsum plasters ($CaSO_4 \cdot 1/2H_2O$) were selected because they have high early strength while being readily available and inexpensive. To achieve desired rapid hardening, the plaster is accelerated by adding 1 pct K_2SO_4 (by weight of dry plaster).

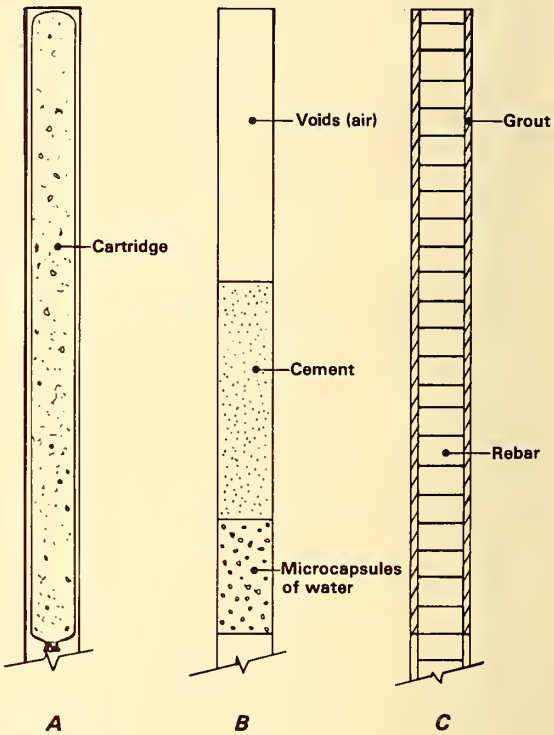

FIGURE 1. - Gypsum-plaster, water-capsule bolt.

[1]Chemical engineer, Spokane Research Center, Bureau of Mines, Spokane, WA.

WATER-CAPSULE CARTRIDGES

A gypsum-plaster, water-capsule cartridge was made, as shown in figure 1. A cartridge (packaged similarly to resin cartridges) (fig. 1A) is inserted into a drilled hole. The cartridge wrapper is filled with accelerated gypsum plaster and water capsules, but also contains air as void spaces between the fine gypsum particles, as shown in figure 1B. During rebar insertion (fig. 1C), the water capsules rupture, releasing the water, which mixes with the plaster to form hardened gypsum.

Figure 2 shows each component in the system. The plaster is on the upper left, and the water capsules on the upper right. A cartridge is in the center, while a short length of rebar is at the bottom.

The water capsules appear in figure 3 alongside a penny, so their size can be noted. Typically, the capsule diameters are 1,800 μm (0.071 in). The water capsules are a modified wax shell surrounding water (encapsulated water). They contain over 60 wt pct water; and to be of adequate quality, they must retain the water and be durable enough to withstand normal handling during cartridge production and installation.

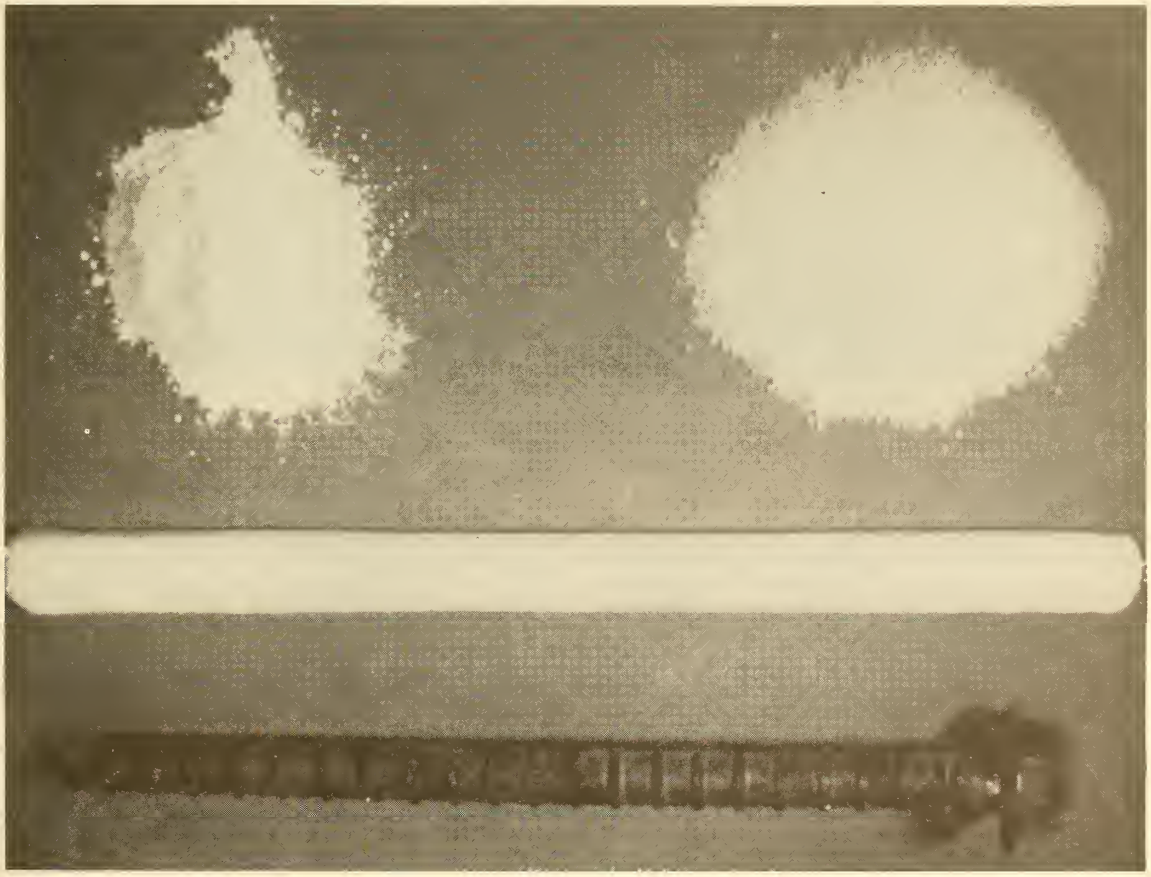

FIGURE 2. - Components of the gypsum-plaster, water-capsule bolt.

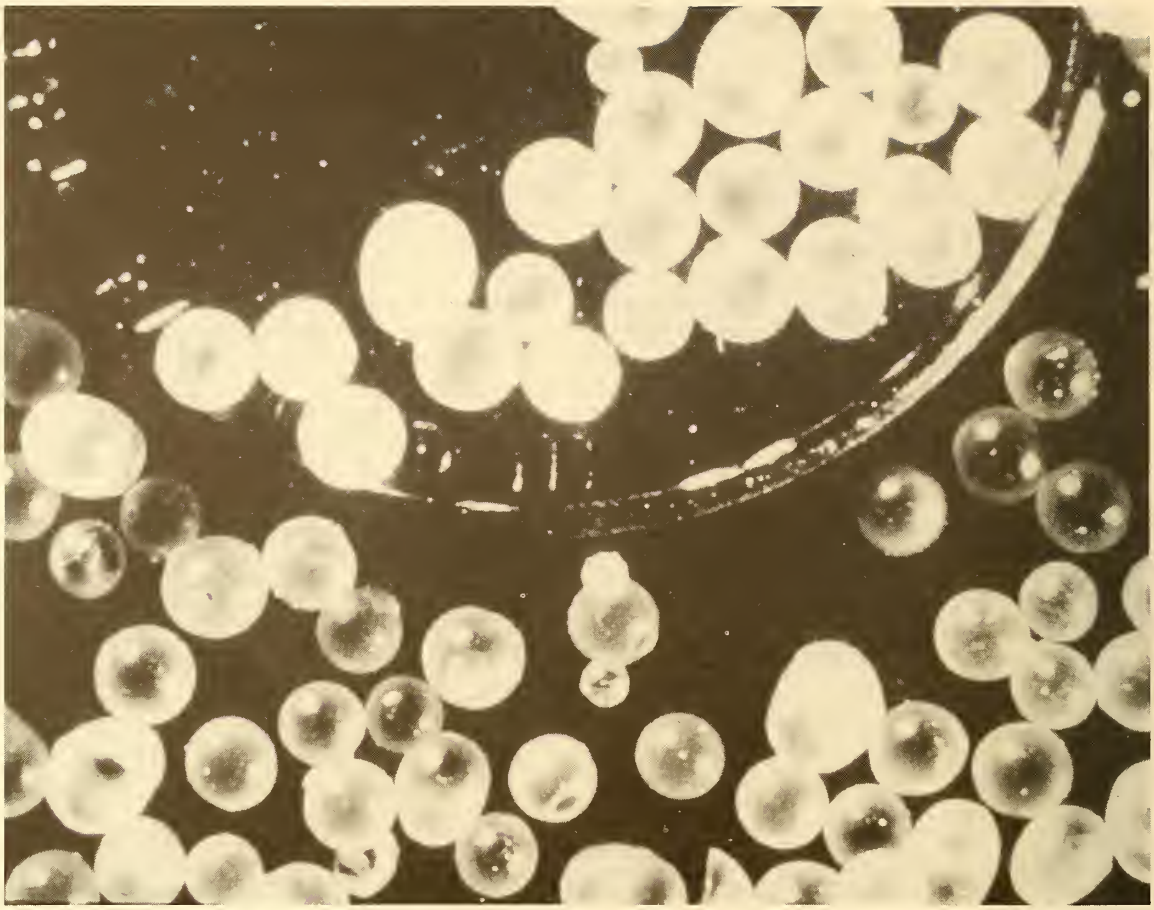

FIGURE 3. - Water capsules.

During installation, the cartridges are manually inserted, as shown in figure 4. The plaster-water mixing is shown in figure 5. The water capsules contain blue dye, which is visible as the rebar is inserted through a cartridge inside a clear plastic tube.

The installation procedure for the inorganic cartridges is similar to that for resin cartridges, except that less rotation for mixing is required. After the cartridges are inserted in a drilled hole, the bolt is inserted part way while being rotated. A wrench is placed on the bolting machine rotation chuck, which allows complete insertion with the available vertical movement. After the wrench is in place, the bolt is rotated during the remainder of the insertion. Extended spinning, with the bolt fully inserted, is not required. As the bolt is inserted, pressure builds within the hole and ruptures the water capsules. The wet mix that is formed is stiff enough so the bolt remains in place, allowing immediate lowering of the bolter head.

Ninety percent of the gypsum strength is developed in less than 10 min. Pull strengths of the bolts (4-ft, Grade 50, No. 6 rebar) done in concrete blocks exceeded the bolt yield point which is over 22,000 lb. Table 1 shows the pull strengths 10 to 13 min after installation.

FIGURE 4. - Manual insertion of cartridges.

TABLE 1. - Pull strength of gypsum-water capsule bolts

Bolt	Pull, lb	Time, min
2.....	26,150	10
3.....	22,400	13
4.....	21,400	10
5.....	25,200	10
6.....	22,400	10
7.....	21,400	10
9.....	21,400	11
11.....	24,300	12
17.....	22,400	10

When the bolt is thrust into a confined cartridge, the pressure has an effect on subsequent pull strength. Pressure as high as 5,500 psi has been measured during bolt insertion in 4-ft holes. Four pull test samples were made by pouring concrete in 4-in pipes that were 4 ft long. After installation of 4-ft bolts in the samples, the samples were cut into eight 6-in sections. Each section had an extension rod attached to the bolt for pull testing. The average pull strength for the four 6-in sections closest to the

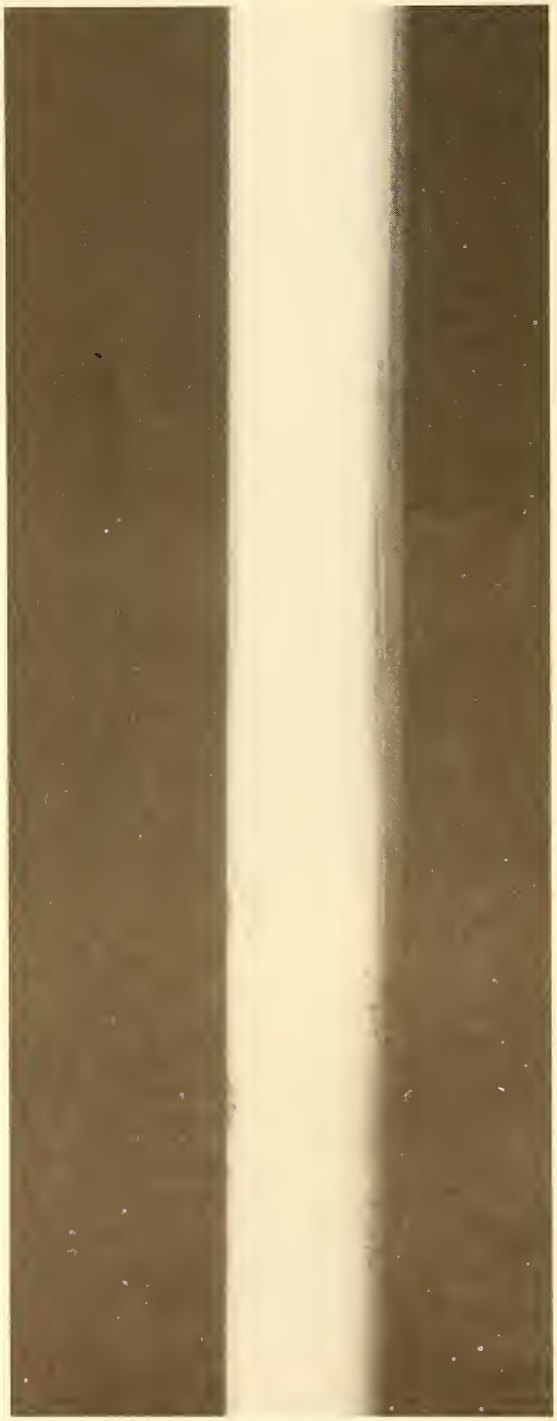

FIGURE 5. - Mixing of gypsum plaster and water capsules.

bolt head (sections A-D), as well as that for the four 6-in sections farthest from the bolt head (sections E-H), is given in table 2. The pull strengths were higher in the half of the bolt furthest up the hole, where pressures were higher, owing to increased cartridge confinement.

TABLE 2. - Average pull strength of 6-in bolt section, pounds

Bolt	Section A-D average	Section E-H average
1.....	8,119	8,756
8.....	6,450	6,963
12.....	11,156	17,538
16.....	14,725	14,794

Bolts were installed in foamed concrete to measure the effect of the weaker rock on pull strength. One bolt gave 9,000 lb and another 15,700 lb pull. More spinning during bolt insertion appears to enlarge the hole in weaker materials like foamed concrete. The smaller amount of required spinning in installing the gypsum-water capsule bolt may become an advantage in weaker rock.

The cartridges are made to have a water-to-plaster weight ratio of 0.30 to 0.35. Less water gives increased strength; however, rebar insertion is more difficult, so the 0.30 weight ratio is the approximate lower limit for repeatable rebar installation.

An additive that increases the fluidity of freshly mixed grout at any water-to-plaster ratio makes the mixing and rebar insertion easier and reduces bolt insertion time. The additive allows rebar insertion at water-to-plaster ratios as low as 0.21. This provides a greater margin for successful underground installation with variations in the cartridges and operator-equipment techniques. By lowering the quantity of water as water capsules, cost savings of a couple cents per cartridge are possible.

A research contract was issued to develop the packaging technology for commercial production of the cartridges. An encapsulation system to provide the

water capsules was developed along with the equipment to produce 20 cartridges per minute. The contractor needs private venture capital to develop a complete production plant and marketing system for the cartridges.

WATER TUBE CARTRIDGES

Cartridges that replace the water capsules with a tube of water are being studied. Figure 6 shows two methods of storing the water within the cartridge. Panel A shows a continuous-length water tube inside the cartridge. As the rebar begins to push on the cartridge, the compression shortens the cartridge length. Flexible water tubes like lay-flat tubing are thought to bend as the cartridge compresses, rather than rupture and release the water. The result is hard rebar insertion due to poorly mixed plaster.

A semirigid tube that retains its shape releases its water sooner to enhance mixing and ease rebar insertion. Several tube materials have been investigated to find one capable of retaining water, withstanding normal handling during cartridge preparation and installation, and breaking as soon as the cartridge is compressed.

Figure 6B illustrates alternate packages of water and plaster within the cartridge wrapper. As with the continuous-length water tube, best rebar insertion is obtained if the water is released as the cartridge is first compressed.

To reduce the loss of fluid material from the hole during rebar insertion, a plastic cap can be inserted over the rebar, which plugs the hole as the rebar installation progresses (fig. 7).

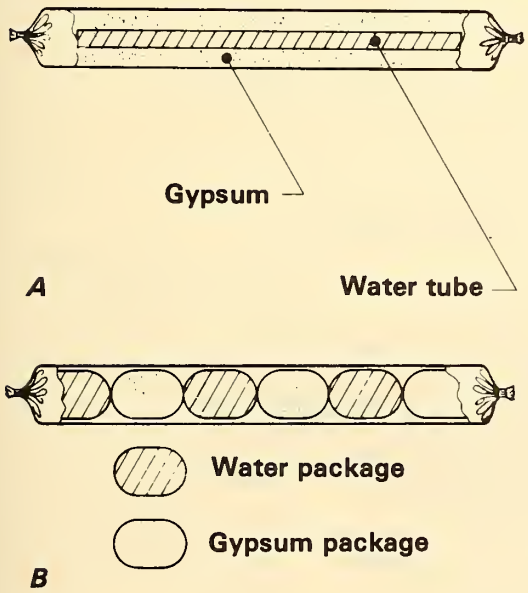

FIGURE 6. - Water tube cartridges.

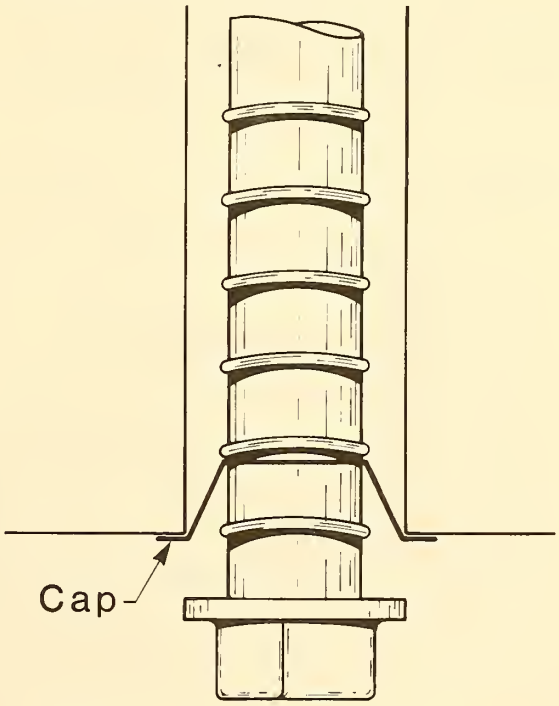

FIGURE 7. - Cap to reduce loss of fluid from hole during rebar insertion.

Water tube cartridges require more mixing than water capsule cartridges because the water is not dispersed throughout the cartridge. To ease water-plaster mixing, an additive can be mixed with the plaster to increase its fluidity at a given water content. Also, the surface tension of the water can be reduced to increase its ability to wet the plaster.

SLURRY INJECTOR

The slurry injector is a bulk injection method that lends itself well to remote-control operations. The operator can work under supported roof during full-column bolting. Dry gypsum plaster is automatically mixed with water to form a slurry, pumped into a delivery hose, and injected up the roof-bolt hole, without placing any mechanical device in the hole. The system uses a twin-screw extruder (fig. 8) normally used for processing plastics. The geometry of the screws makes them self-cleaning. The extruder mixes and pumps the grout into a 20-ft delivery hose attached to a transfer device. After the grout is in the hose, the transfer device inserts a plastic "rabbit" or plug behind the grout. High-pressure air then drives the rabbit and grout through the hose and nozzle into the roof-bolt hole. The delivery hose is cleaned by the rabbit. The nozzle is positioned beneath the hole by a linkage mounted on the bolter drill head.

To operate the system, the hopper is filled with gypsum plaster and a tank is filled with water. A control knob selects the proper grout volume and water-to-plaster weight ratio for the size hole being drilled. After the hole is drilled, the nozzle is positioned under the hole and the rabbit inserted into the transfer device. The machine is switched on and all operations through complete bolt insertion are then automatic.

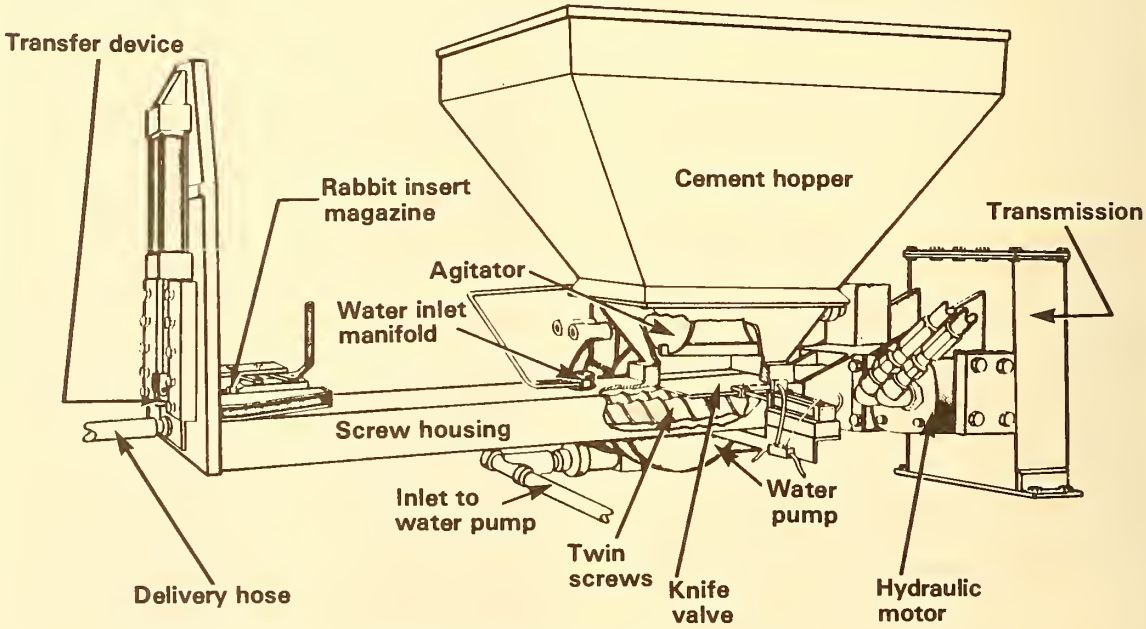

FIGURE 8. - Slurry injector components.

CONCLUSIONS

1. Gypsum plaster-water capsule cartridges provide adequate bonding of No. 6 rebar in 4-ft holes.

2. The gypsum plaster-water capsule cartridges provide a substitute for resin cartridges.

3. The gypsum plaster-water capsule cartridges may be an advantage in softer rock where more spinning disturbs the integrity of the hole.

4. Capital is necessary for commercial production of the gypsum plaster-water capsule cartridges.

5. Water tube cartridges should be further investigated, as they may be easier to manufacture and more economical.

6. The slurry injector is a promising concept because of possible improvements in operator safety and economy of bolt installations.

RESEARCH, DEVELOPMENT, AND USE OF STEEL-FIBER-REINFORCED CONCRETE CRIBBING FOR MINE ROOF SUPPORT

By Thomas W. Smelser[1] and Dale A. Didcoct[2]

ABSTRACT

Through the combined efforts of the Bureau of Mines and private industry, steel-fiber-reinforced concrete crib block to improve coal mine safety in the area of roof control has been developed and is currently in use commercially. The development objective was to improve health and safety conditions with a stiffer, stronger, nonflammable roof support at a competitive cost. During the time this method of roof control has been in use in the mining industry, it has proven to have definite functional and economic advantages.

INTRODUCTION

In 1975, the Bureau initiated developmental research in the area of a steel-fiber-reinforced concrete crib block as an alternative to the use of wood cribs. Aside from the obvious disadvantages of low stiffness and strength, deterioration from chemical and bacteriological attack, methane liberation, and flammability, wood cribs are subject to variables in cost and availability of suitable varieties in many areas of the country. In the Eastern States, a reliable source of pressure-treated wood is often difficult to find, and the scarcity of any type of wood in some Western States makes it extremely expensive. Concrete offered the lowest cost support and appeared to be the best choice for replacing wood supports if a solution could be found for its poor failure characteristics. The addition of steel fibers to the concrete would greatly improve the safety factor by eliminating the possibility of sudden brittle failure.

Starting in 1976, a project was conducted at the Bureau's Spokane (WA) Research Center, under the direction of G. L. Anderson, Research Structural Engineer, and Thomas W. Smelser, Supervisory Mechanical Engineer. The many types of fibers that were considered in the initial testing included glass, asbestos, aramid, nylon, carbon, polypropolene, and steel. Except for steel, the aforementioned fibers were each eliminated owing to various problems with mixing characteristics, economy, health hazards, strength, etc. Steel fibers also appeared to be the lowest cost solution, as compared with the use of steel-reinforcing bars or steel wire mesh. The steel fiber types considered included straight-round, straight-flat, crimped-full-length, melt-extracted, deformed-full-length, and bent-end. The bent-end type fiber was selected, based on performance and cost considerations.

Upon completion of the selection of the fibers to be used, the next step was to select a concrete mix design and determine the quantity of fibers for the mix. The steel-fiber-reinforced concrete (SFC) support members that were developed offered a significant improvement in stiffness and strength in compression compared with wooden cribs, yet they avoided the brittle or catastrophic compressive failure mode of plain concrete. Full-scale compression testing showed the ultimate compressive strength of SFC blocks to be 4,000 psi, compared with 500 psi for wood. Because eight times more wood is necessary to equal the strength of the steel-fiber-reinforced concrete crib block, the use of SFC greatly increases the area for movement of

[1]Supervisory mechanical engineer, Spokane Research Center, Bureau of Mines, Spokane, WA.

[2]Vice president, Underground Technology Div., Burrell Group, Morristown, TN.

personnel and equipment and for ventilation airflow. The 4,000-psi concrete was selected as optimum strength by most closely approximating the strength of a typical roof and floor structure of a coal mine.

Bureau Report of Investigations 8412, published in 1980, contains support system design and results of laboratory investigation and full scale testing.[3]

A successful installation of steel-fiber-reinforced concrete cribs at Kaiser's Sunnyside Mine in Utah was made in 1976 as part of the single-entry longwall demonstration at that mine. The wood cribs being used to support the single entry were not stiff enough to hold the tailgate section of the entry open after passage of the first longwall face. Although limited, the demonstration showed the promise of concrete mine support systems and resulted in the project, covered by this report, to more thoroughly characterize and evaluate materials in the laboratory for improved support characteristics, safety, and economy. A follow-on program field-tested the support systems to verify installed cost, structural behavior, and industry acceptance.

DEVELOPMENT OF MANUFACTURING PROCESS

The prototype blocks made by the Bureau in the research and testing phase were manufactured on a small production basis and cost approximately $7 per block. This cost was prohibitive, and a method of mass-producing the steel-fiber-reinforced blocks had yet to be developed. A major manufacturer of concrete block and steel fiber gunite mixes, Burrell Construction and Supply Co., New Kensington, PA, was contacted by the research team.

With the Bureau contributing their technical specifications, and Burrell Construction and Supply Co. providing the formulation, production technique, and mix methods, an economical SFC block was produced in the summer of 1980.

By summer 1981, Burrell had developed a process and method (patent applied for) for mixing the concrete and steel fibers to produce crib blocks that fulfilled the standards set by the Bureau. Many full-scale crib tests were performed at Lehigh University, Pittsburgh Testing Lab, and on the Mine Roof Simulator at the U.S. Department of Energy (DOE) in Bruceton, PA (figs. 1-4). The results of the DOE testing are contained in the DOE Report MRS-DR-81-05.[4]

[3]Anderson, G. L., and T. W. Smelser. Development Testing and Analysis of Steel-Fiber-Reinforced Concrete Mine Support Members. BuMines RI 8412, 1980, 38 pp.

[4]Byrd, R. J., and J. L. Thompson. Mine Roof Simulator Data Report Steel-Fiber-Reinforced Concrete Roof Cribs (Three Configurations). U.S. Dep. Energy, Min. Equip. Test Facility, MRS-DR-81-05, Aug. 1981, 40 pp.

FIGURE 1. - Solid crib failure mode (DOE tests).

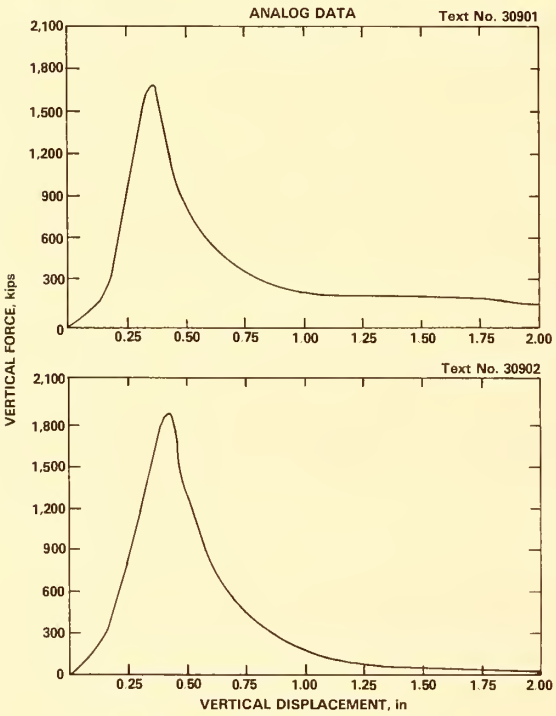

FIGURE 2. - Solid crib-vertical load versus displacement (DOE tests).

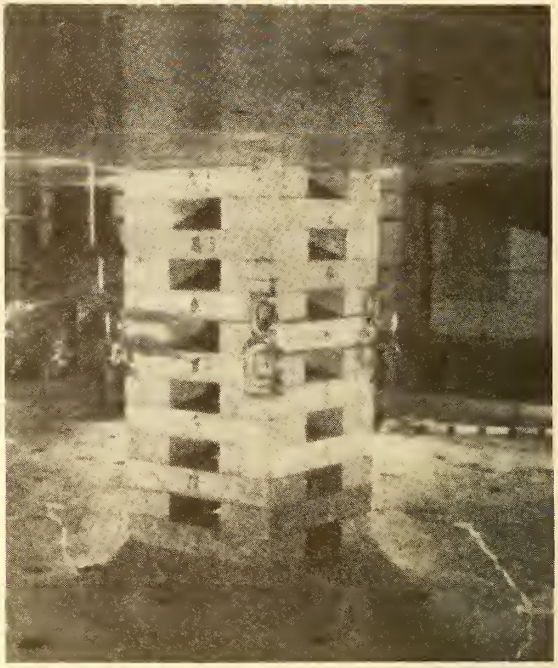

FIGURE 3. - Open crib failure mode (DOE tests).

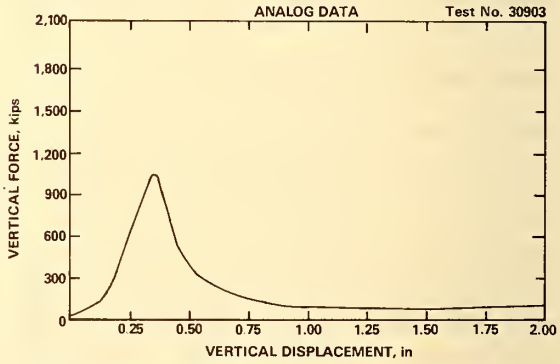

FIGURE 4. - Open crib-vertical load versus displacement (DOE tests).

The resulting mass production technique reduced the cost per unit to $2.29 FOB plant. The block is presently being manufactured under licensed agreement with Burrell at locations in Pennsylvania, Ohio, Virginia, Utah, and Alabama. Other block producers are beginning to enter the market with similar products.

UNDERGROUND EXPERIENCE

Since the original installations in 1976, at Kaiser's Sunnyside Mine, the Bureau has been involved in cooperative underground evaluations at several other locations:

Kaiser Steel, York Canyon, NM

Price River Coal, Helper, UT

Snowmass Coal, Carbondale, CO

Bethlehem #131, Van, WV

U.S. Steel #9, Gary WV

In all these cases, the cribs were used in the tailgate section of the entries on longwall panels (fig. 5). Because of the successes of the initial installations, some of the above are expanding the use of SFC to areas where permanent support systems are required, such as ventilation entries, and to support the base of ventilation shafts.

Several other coal companies have elected to institute the use of this product in longwall entries, long-life main entries, bleeder entries, and stoppings:

Eastern Associated Coal Company--WV

Emery Mining Company--UT

Trail Mountain Mining Company--UT

Barnes & Tucker #25--PA

Penn Allegheny Coal--PA

Canterbury Coal--PA

FIGURE 5. - Concrete cribs supporting tailgate after passage of longwall face. Kaiser's York Canyon, NM, mine.

Carpentertown Coal & Coke--PA

Scott's Branch Mine--KY

Martin County Coal--KY

Texas Gulf, Inc.--WY

Westmoreland Coal Company--VA

Helvetia Coal Company--PA

ARMCO Inc.--WV

Consolidation Coal Co.--WV & PA

Island Creek Coal Co.--VA

Jim Walters Resources Inc.--AL

Kit Energy--WV

In almost all applications of the tailgate entries for longwall, the SFC has not only provided added safety, better airflow, and more area for movement, but has also proven cost-effective. For example, in a mine using a double row of wood cribs set on 5-ft centers, they are now installing SFC cribs in a single row on 7-ft centers (figs. 6-7).

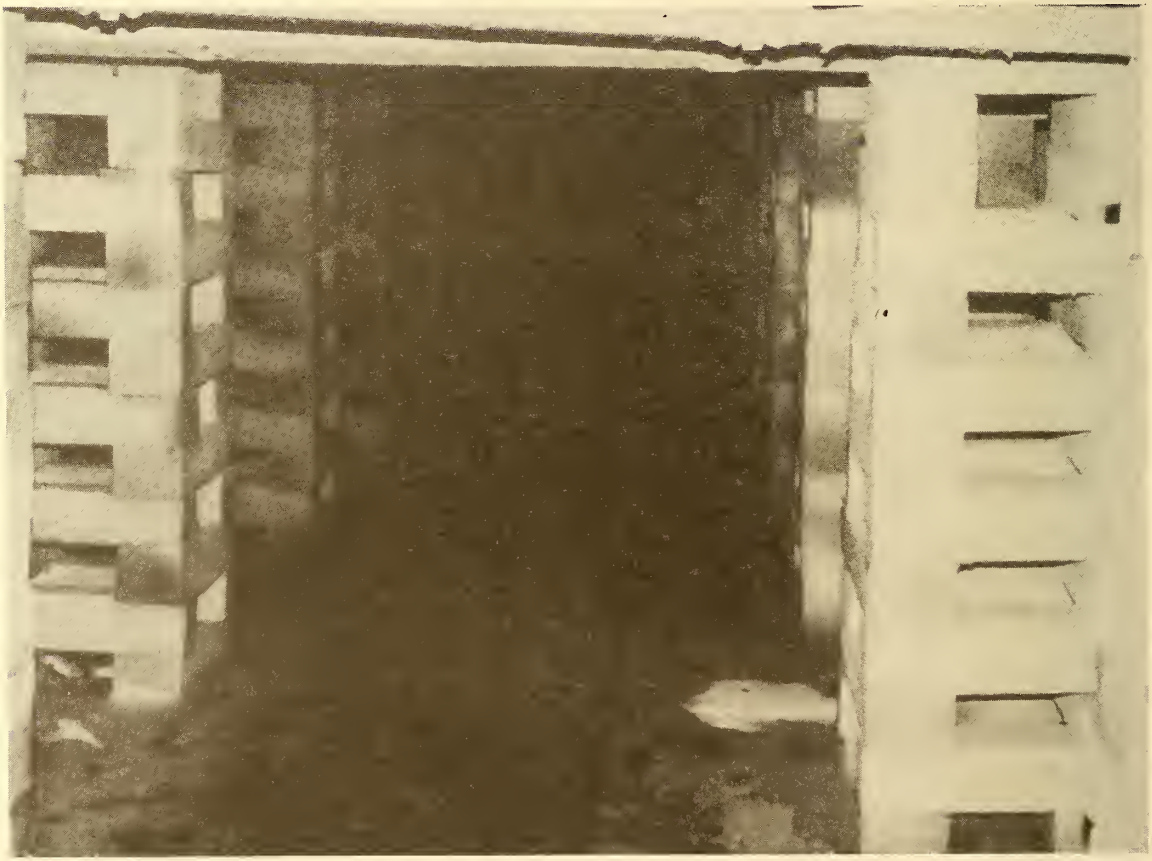

FIGURE 6. - Longwall tailgate entry with double row of wood cribbing. West Virginia mine.

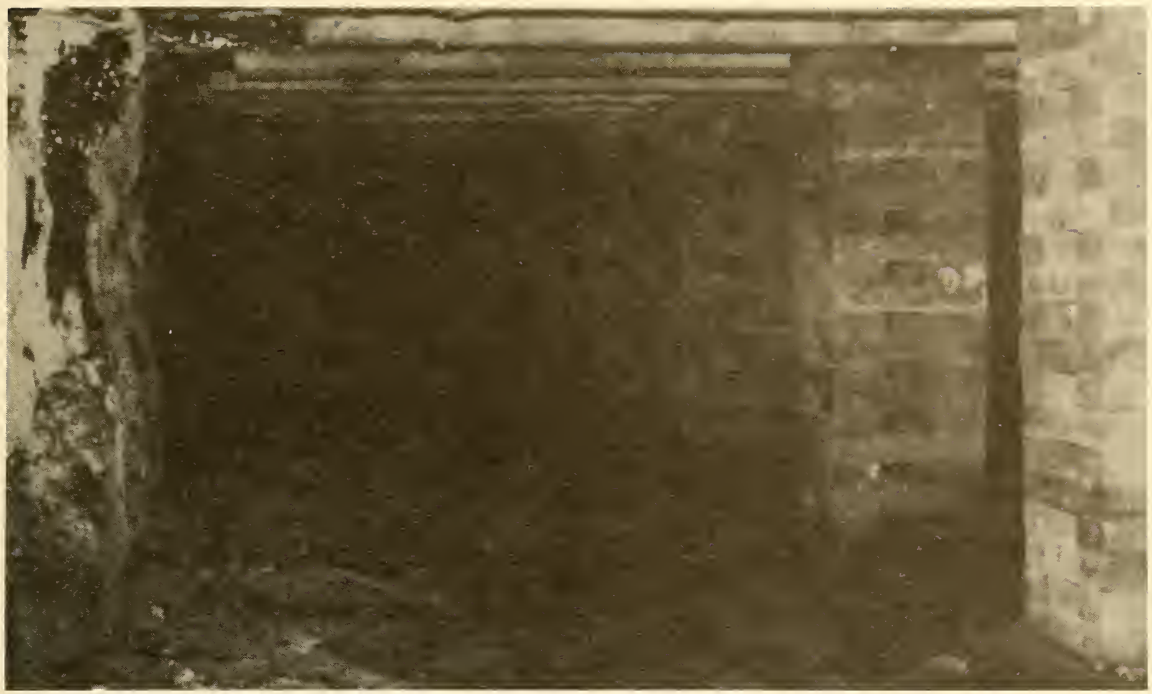

FIGURE 7. - Longwall tailgate entry with single row of concrete cribbing. West Virginia mine.

COST ANALYSIS

Following is an example of a cost comparison for this West Virginia mine:

FIBERCRIB versus Wooden Cribs

(Based on 84-in height--1,000-ft advancement

Wooden Cribs - (6 by 8 by 30 in)--Double Row on 5-ft centers--28 blocks per crib @ $2.39 FOB Mine = $66.92 per crib

400 Cribs @ $66.92 = $26,768.00

FIBERCRIB Cribs - (3-5/8 by 7-5/8 by 23 in)--Single Row on 7-ft centers--46 blocks per crib @ $2.88 FOB Mine = $132.48 per crib

142 Cribs @ $132.48 = $18,812.00

Labor Costs - The labor cost to build each crib is approximately equal. Note that 400 wood cribs were required in this example, as opposed to 142 FIBERCRIB cribs, resulting in a labor cost saving of over 60 pct. The total resulting cost saving in this example is over 40 pct for the FIBERCRIBS.

In another instance, where the mine was setting a single row of wood cribs skin-to-skin, they have found the use of SFC cribs on 6-ft centers to be cost-effective.

In an Alabama mine, a longwall tailgate, originally supported with a single row of wooden cribbing 9 ft on center, is being successfully supported with a single row of SFC cribs 15 ft on center. This will result in a total savings of over 55 pct for labor and materials expended for support of this tailgate (figs. 8-9).

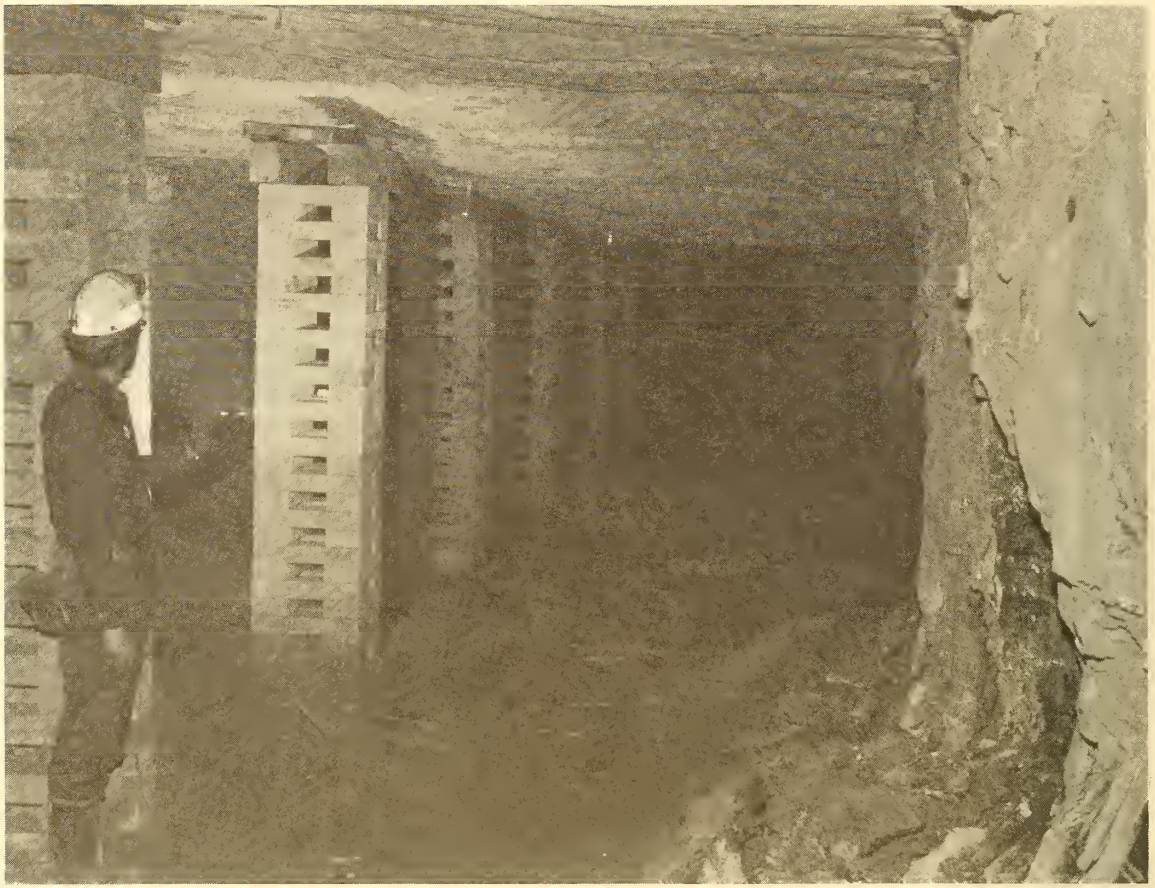

FIGURE 8. - Tailgate supported with concrete cribbing ahead of longwall face. Alabama mine.

Many of these mines have also found the SFC cribs to be efficient in areas where longevity is a factor and, although the cost may be initially higher in some cases, the longer life of the SFC cribbing will make them more economical in the long run. The industry has found that in the case of long-life entries, the SFC cribbing is an advantage because of the inherent qualities of stiffness, strength, nonshrinking, nonrotting, and nonflammability. Replacement costs on wood cribbing in these applications have proven to be very high. In any area where a wood crib would need replacing because of rot and/or failure, the SFC block would eventually prove to be more economical.

In most applications it is possible to use fewer SFC cribs than wood cribs in order to achieve the same support. A Pennsylvania mine, with their belt and track lines running parallel, was using wood posts skin-to-skin and, in some cases, two to three deep to support the roof. They are now building solid SFC cribs on 6-ft centers with an "I" beam from crib to crib. This has given them much greater support and they now have access between beltline and track. This particular crib installation was inspected by representatives of the Office of Deep Mine Safety, Pennsylvania Department of Environmental Resources. The two inspectors recommended, "An approval be granted to Burrell Construction and

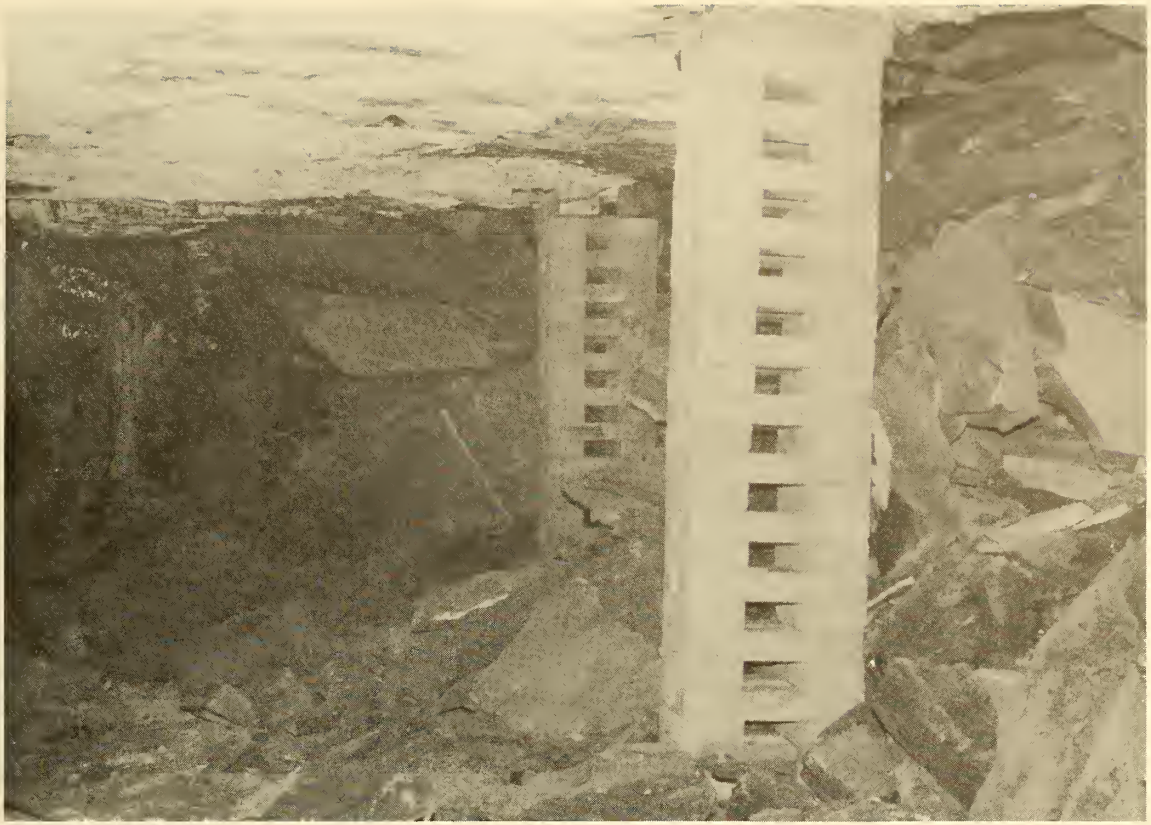

FIGURE 9. - Tailgate supported with concrete cribbing after passage of longwall face. Alabama mine.

Supply Company for use of FIBERCRIB in bituminous underground coal mines in Pennsylvania, providing FIBERCRIB blocks strictly comply with the manufacturer's specifications" (fig. 10).

To date, a combined total of about 20 miles of longwall tailgate entry, main entry, and bleeder entry is being supported with SFC cribbing in the major U.S. coal mining regions. SFC cribbing has also been used to build stoppings and overcasts where normal cinder block had proved ineffective due to crushing. The SFC block is approximately 2-1/2 times stronger than a regular cinder block and is not subject to brittle failure.

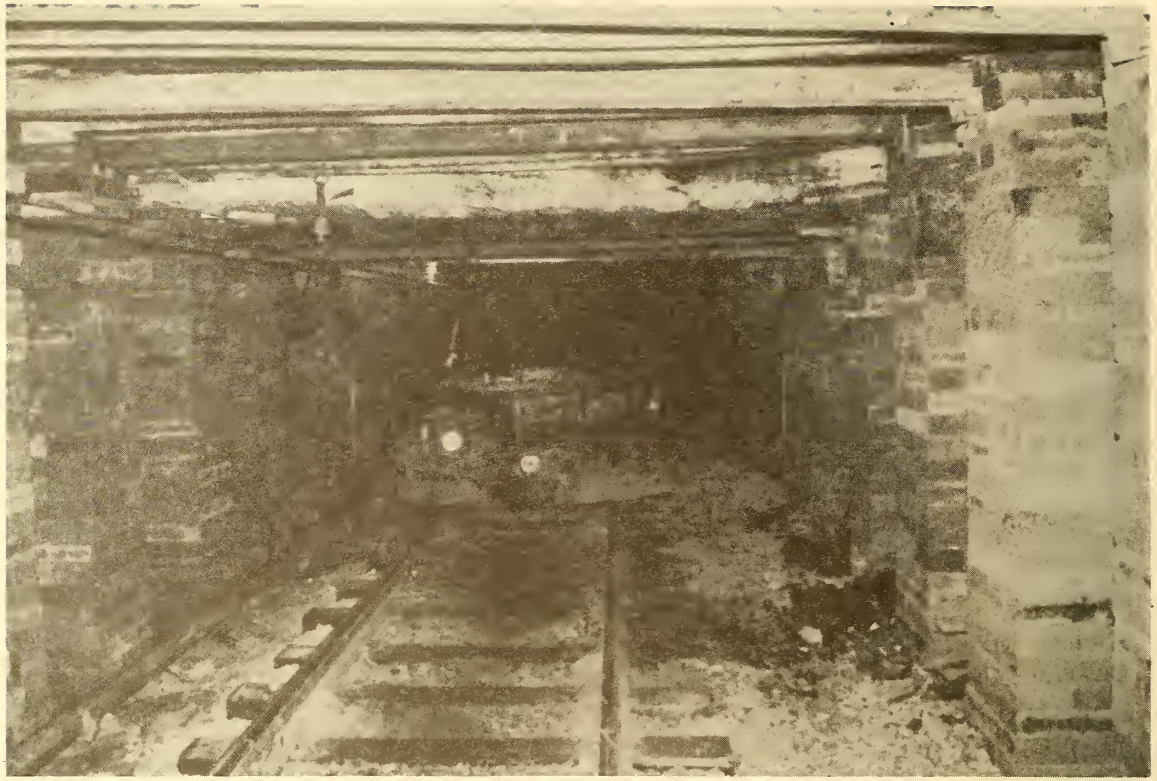

FIGURE 10. - Solid concrete cribs placed 6 ft on center supporting a main haulage entry. Pennsylvania mine.

INSTALLATION PROCEDURES

The procedure for installation of SFC crib blocks, suggested by the Bureau, is as follows:

CONCRETE CRIBS (FIBERCRIB)

1. Prepare floor area level and flat, large enough for 23- by 23-in, 15- by 23-in, or 15- by 15-in crib, as required.

2. Place first layer of crib blocks and check for level with hand level.

3. Stack block in open or solid configuration according to plan.

4. While stacking, keep blocks straight, square, and plumb and remove dirt, etc., from each layer before placing next layer.

5. Top of crib must be finished with wood plank or beam and wedged tightly with wood wedges. The total thickness of wood including wedges must be a minimum of 1 in for each foot of crib height (4 in for 4-ft crib, 6 in for 6-ft crib, etc.) (fig. 11).

CONCRETE STOPPINGS USING FIBERCRIB BLOCKS

1. Follow above procedure except excavate floor for stopping dimensions.

2. Stack block flat and in overlapping fashion as brick is typically laid.

3. Finish top of stopping with the least amount of wood wedging possible or fill with mortar mix.

4. Seal one side of stopping with suitable mortar or sealant.

FIGURE 11. - Example of insufficient wood ot top of crib. Point looding.

SUMMARY AND CONCLUSIONS

There is no question of the strength and longevity of the steel-fiber-reinforced concrete cribbing and its success in present installations. Although wood has been the traditional material in use for roof support in the mining industry, the problems of supply and cost are increasing rapidly to the point where a viable alternative must be found. Concrete products are readily available in all parts of the country, and, indeed, the world. The steel-fiber-reinforced concrete cribs are proving to be the answer to many roof control problems experienced in the past and are the most economical alternative to wood.

One industry source in the field of mine safety asserted: "The new cribs are much safer due to the inherent characteristics of steel reinforced concrete. We have had no failures, and we have long-term installations in which we have confidence that these cribs are a reliable source of roof support." Some mines have already decided that, in their operations, wood cribbing is a thing of the past.

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