MASTER

# Proceedings First Conference on Ground Control Problems in the Illinois Coal Basin

Editors
Yoginder P. Chugh and A. Van Besien

Southern Illinois University
at Carbondale

June, 1980

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# Proceedings
# First Conference on Ground Control Problems in the Illinois Coal Basin

*held at*

Southern Illinois University at Carbondale
Carbondale, Illinois

**August 22-24, 1979**

*Edited by*

Yoginder P. Chugh* and A. Van Besien**

*Department of Mining Engineering
Southern Illinois University at Carbondale

**Carbondale Mining Technology Center
U.S. Department of Energy

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Southern Illinois University at Carbondale

June, 1980

# FOREWORD

The First Conference on Ground Control Problems in the Illinois Coal Basin was held at Southern Illinois University at Carbondale August 22-24, 1979. The Conference was sponsored by Carbondale Mining Technology Center, U.S. Department of Energy, Department of Mining Engineering, SIUC and Coal Extraction and Utilization Research Center at SIUC in cooperation with the Division of Continuing Education at SIUC. Approximately 130 persons from industry, academic and research institutions, federal and state agencies, and consulting companies throughout the U.S. participated in this two and one-half day Conference.

The Conference consisted of five sessions: 1) Geology of the Basin and Geologic Factors Affecting Mining, 2) Ground Control in the Room-and-Pillar Mining System, 3) Artificial Supports, 4) Longwall Mining, and 5) Subsidence. The primary objectives of the Conference were to 1) compile available information and experience related to ground control problems in the basin and disseminate it by publishing it as conference proceedings and 2) present current operating problems and ongoing ground control research pertinent to basin mines. The longwall and subsidence sessions were considered very pertinent because several mining companies are actively considering longwall mining systems in the basin coal mines. A total of twenty-six (26) papers were presented. The technical papers for this Conference were sought by invitation as well as by mailing requests for call for papers to appropriate individuals in universities, federal and state agencies, and consulting companies.

I would like to take this opportunity to extend sincere thanks to all those who contributed to the success of the Conference. Special thanks are due to authors, session co-chairpersons and the Conference organizing committee. I particularly wish to acknowledge the assistance and cooperation of Mr. A. Van Besien, Director, Carbondale Mining Technology Center (Conference co-chairperson), Dr. Kenneth E. Tempelmeyer, Dean, College of Engineering and Technology, Dr. Philip K. Davis, Chairperson, Department of Engineering Mechanics and Materials, Dr. Lyle V. A. Sendlein, Director, Coal Extraction and Utilization Research Center, and Professor Harold Engelking of the Division of Continuing Education.

June 1980

Yoginder P. Chugh
Conference Co-Chairperson
Department of Mining Engineering
Southern Illinois University at Carbondale

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# SESSION I

## GEOLOGY OF THE BASIN AND GEOLOGIC FACTORS AFFECTING MINING

Session Co-Chairpersons:

Mr. A. Van Besien, Carbondale Mining Technology Center
U.S. Department of Energy

Dr. R. R. Dutcher, Department of Geology
Southern Illinois University at Carbondale

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# COAL GEOLOGY AND UNDERGROUND MINING, ILLINOIS COAL BASIN

M. E. Hopkins

Consulting Geologist
Harry Williamson, Inc.
Benton, Illinois

## Introduction

Because of the relatively thick and persistent coal seams, which resulted from uniform depositional environments prevailing over wide areas, underground mining conditions in the Illinois Coal Basin are generally favorable for large, high-productivity mines. A wide variety of problems, however, result from recognizable geologic conditions. Undoubtedly, the most important group of geologic factors affecting minability relate to the nature of the stratigraphic section over the coal and its amenability to roof control. Significant depositional variations involving competent and non-competent roof strata are present on both regional and local scales. Regional changes have to some extent been mapped; local variations normally cannot be mapped from subsurface data but their presence can usually be recognized by careful observations of cores and logs.

Other geologically-related factors affecting the minability are: coal depth, tectonic and non-tectonic faults, igneous intrusions, fracture systems, water invasion, local high grades, methane, soft floor conditions, and diagenetic mineralizations (e.g., coal balls and roof concretions). Careful evaluation of these factors during mine planning can often minimize future problems.

## Underground Mining

Essentially all the extensive underground mining in the Illinois Coal Basin has been confined to a few coals occurring in the Carbondale Formation which is found near the middle of the Pennsylvanian System. The most important coals are the Herrin (No. 6) Coal (No. 11 in western Kentucky and the Herrin Coal in Indiana) and the Springfield-Harrisburg (No. 5) Coal (No. 9 in western Kentucky and Springfield Coal V in Indiana). A few other coals have been extensively mined in certain areas of the three states, but on a relatively small scale considering total production.

Underground mining accounted for 40 percent of the total Illinois Coal Basin production during 1977. For each state the underground production totalled the following amounts (the percentage is that portion of the total production):

Illinois-29.4 million tons (55%)
Western Kentucky-23.4 million tons (45%)
Indiana-.5 million tons (3%)

Because the largest proportion of the total reserves available are beyond the depth limit of strip mining, it is expected in the future that relative quantity of underground-mined coal will increase. The total demonstrated reserves and the percentage of which is deep are as follows:

Illinois-65.7 billion tons (81% deep)
Western Kentucky-12.6 billion tons (69% deep)
Indiana-10.6 billion tons (84% deep)

Room-and-pillar mining has been the method employed in most of the mines, and recovery seldom exceeds 50 percent over the area of the mine. Where surface subsidence rights are controlled, this percentage can be increased. Longwall mining, after several unsuccessful attempts, is increasing, but its application will generally be confined to those areas where surface subsidence rights are, or can be, controlled by the operating company.

## Mining Conditions

The most important characteristics of the Illinois Basin coals which make for the well-known high mine productivity are: persistence of thick, relatively parting-free coals; gentle dips; relatively dry conditions; low methane liberation; and generally good roof conditions. The only widespread undesirable characteristic, as compared to other mining areas, is the relatively soft floor, a condition resulting from the almost universal presence of poorly consolidated underclay (or "fireclay") immediately beneath the coal. Most of the mining problems in areas characterized by coal of adequate thickness results from variations in the quality of roof strata.

### Thickness

Coals presently being mined by underground methods in the Illinois Basin vary from about 4 feet up to about 8 feet, with the average being at least 6 feet. In order for the 4- to 5-foot coals to be competitive, market or transportation factors must be particularly favorable, such as shipping by river barge or being a part of a large complex which includes strip mines (such as a mine with portals driven off of a strip highwall). It is expected in the future that the average thickness will gradually decline, as the thicker coals are exploited at a faster rate.

For the No. 6 and No. 5 coals, and probably for others as well, the thickest deposits are found in belts several miles wide on either side of prominent drainage courses, now mappable as extensive coal cutouts as shown for Illinois (Figs. 1 and 2). The No. 6 Coal (Fig. 1) has wider development of thicker coal (7 feet or more) than the No. 5 (Fig. 2). The channel system with the No. 6 Coal is also wider, indicating a larger, or more long lasting, stream system, than is the case for No. 5 Coal. (The major east-west sandstone cutout in southern Illinois, portrayed on Fig. 1, which shows no relation to coal thickness, is filled with Anvil Rock Sandstone, a unit stratigraphically much younger than the prominent north-south Walshville Channel, the channel system associated with thicker coal, gray shale roof and low-sulfur coal.)

### Depth of Mining

The deepest large mine ever to have operated in the basin is currently active; the shaft is about 950 feet deep. The deepest known was a small mine operating many years ago at a depth of more than 1,000 feet. The greatest depth that the No. 5 Coal attains over a substantial area is slightly less than 1,300 feet (about 1,250 feet for the No. 6). This is in the large relatively flat-bottom central part of the basin in Jasper County, Illinois. (Locally, greater depths, up to at least 1,800 feet, are found in down-dropped fault blocks in western Kentucky.) Most of the present underground mining is in areas where the No. 6 Coal is 300 to 500 feet deep but several large mines are in the 600- to 800-foot range. Serious mining problems related to depth are not to be expected in the major coals, which do not exceed the 1,300 foot depth as mentioned above for the No. 5 Coal.

Mining problems do result from shallow depths. Where shallow mining occurs under valleys, or under glacial drift-filled bedrock valleys, poor roof and/or wet conditions may cause ground control

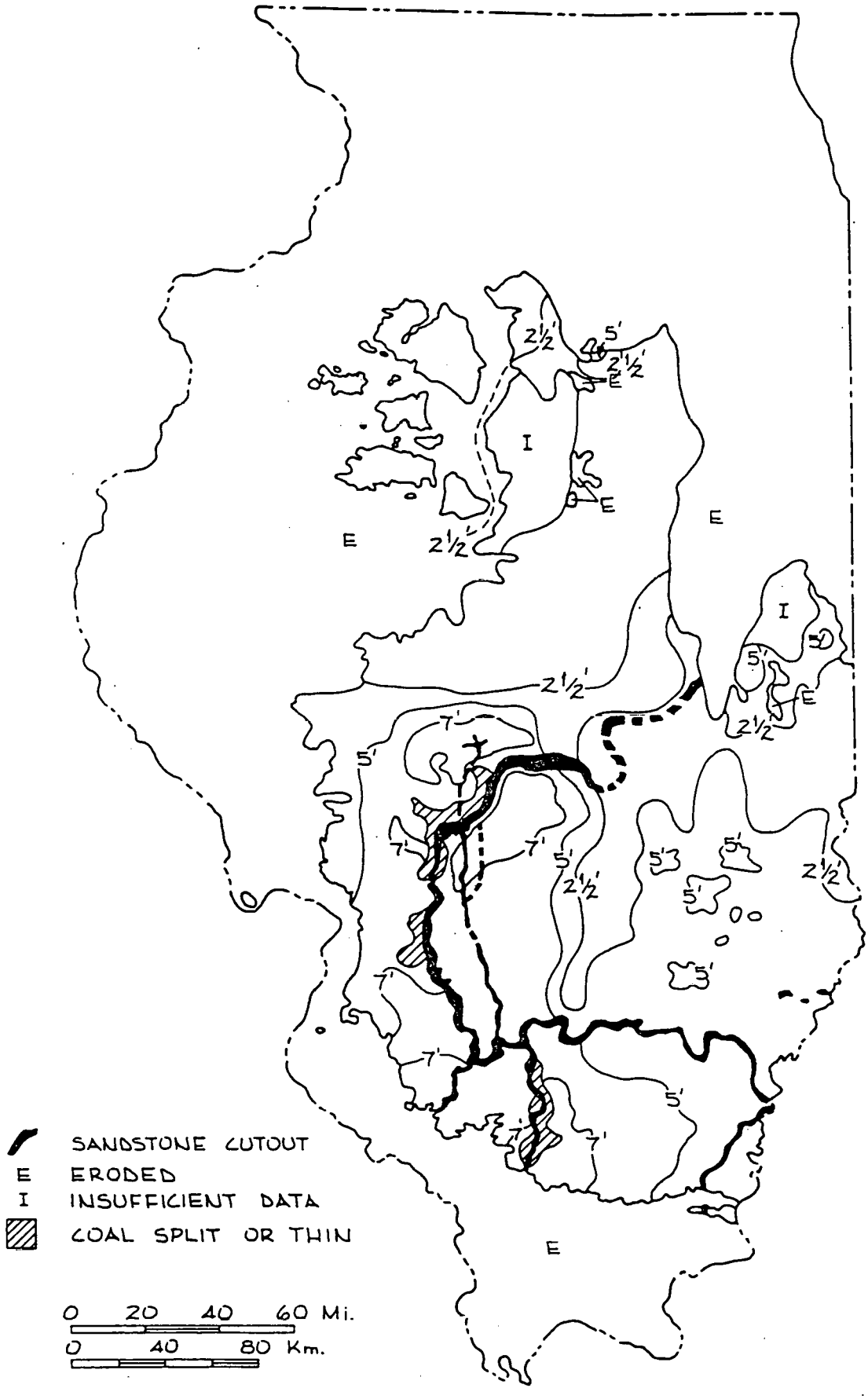


Fig. 1. Generalized thickness, Herrin (No. 6) Coal.

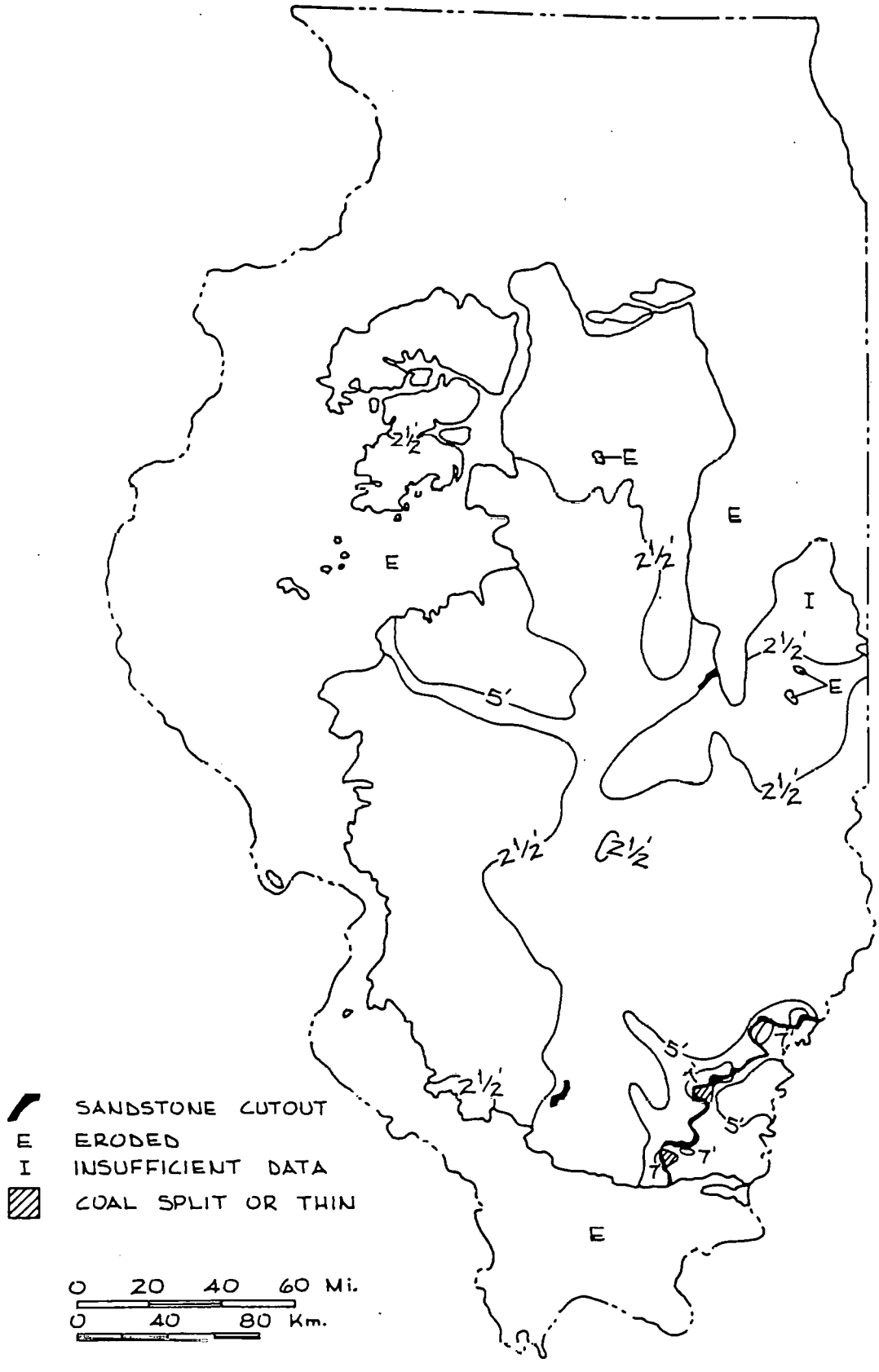


Fig. 2. Generalized thickness, Springfield-Harrisburg (No. 5) Coal.

and/or water problems. The glacial drift, composed of unconsolidated clays, silts or sands, has very little strength and acts primarily as a "dead load." Also, the silts and sands (and occasional gravels) are commonly good aquifers. Any breaks in the roof may become avenues for water movement into the mine. Generally speaking, 100(±) feet of bedrock is desirable, but good conditions are known to occur with less overburden if competent rocks such as limestone are present in the roof sequence.

## Roof Conditions

Relative to the general nature of the rocks overlying the Herrin (No. 6) and Springfield-Harrisburg (No. 5) coals, there are two general types (Figs. 3 and 4). Over most of the basin, the coals are overlain immediately and abruptly by up to 5 feet of hard, black, "slaty" shale which is, in turn, overlain by a limestone unit of similar thickness in the case of No. 6 Coal and generally less than 2 feet in case of the No. 5. Both of these units contain marine fossils. In places, especially with the No. 6 Coal, the black shale pinches out and the limestone constitutes the immediate roof. This black shale/limestone roof condition is characteristic of the typical "high-sulfur" Illinois Basin coals.

The second general roof type is silty gray shale which may be as much as 60 feet (occasionally 100 feet) thick. This shale, more localized than the black shale/limestone roof type, occurs stratigraphically between the coal and the black shale/limestone. It is associated with a large fluvial/deltaic complex which is characterized by extensive channel sandstone deposits, split coal, and "wash-outs." Marine fossils are generally rare with land plants being the dominant fossil form. The stream complex existed during and after coal accumulation.

Normally, the coals under these gray shales are much lower in sulfur than the same coal in adjacent black shale/limestone roof areas. The increase in sulfur, for the most part, is considered as a product of marine conditions occurring immediately after coal accumulation.

## Black Shale/Limestone Areas

Stratigraphic variation in the immediate roof of the coal is the most important factor in affecting the roof quality. This has been most often documented in the case of the No. 6 Coal in Illinois. Local depositional conditions resulted in lenticular occurrences of the Brereton Limestone as illustrated in Fig. 5. Here the Brereton Limestone has pinched out and the Anna (black) Shale has thinned.

The much weaker Lawson Shale, characterized by poor bedding and abundant slickensides, has much more influence on the roof quality over the central mine entry. A roof fall might occur as shown by the arched configuration of the dashed line. In this case roofbolt holes have been drilled into a lense of Anvil Rock Sandstone, the localization of which may have been determined by the thinner interval between it and the top of the coal. Although not a good aquifer, this sandstone has been observed to be a source of water leaking into the mine. A roof fall would also expose more sandstone, thus increasing its water yield.

Appropriate control might prevent a roof fall in the case of Figure 5. Extra long bolts (extending into the competent Bankston Fork Limestone), epoxy-grouted bolts, rail bars, or other techniques, might provide successful control. Normal conditions, where the limestone and black shale are both present are found in the two outside entries and roof quality should be good. There are also extensive areas where the Brereton Limestone is the immediate roof and here, roof conditions are usually excellent.

These variations discussed above have been observed in many mines throughout Illinois and have also been identified in drilling for potential new mining.

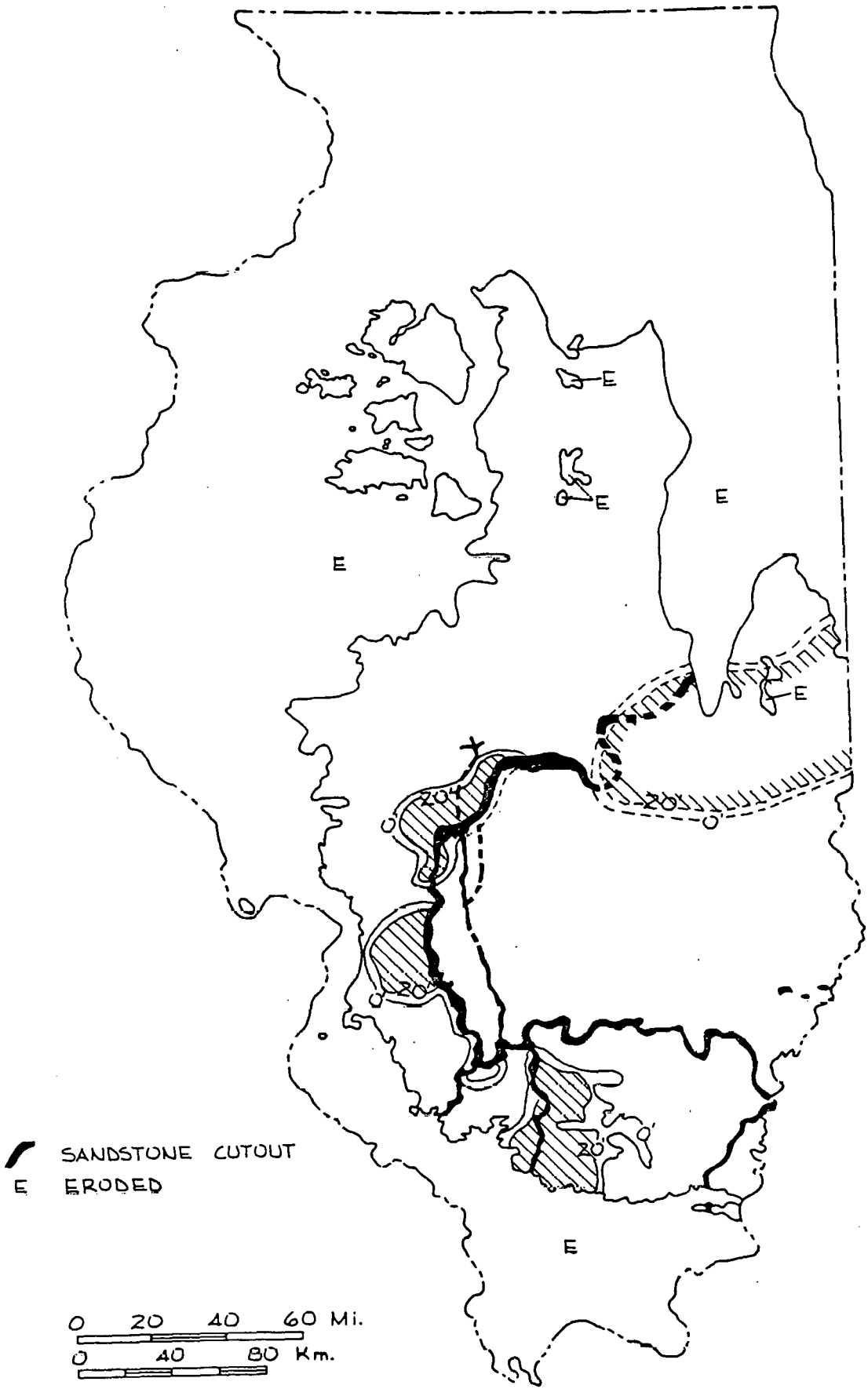


Fig. 3. Generalized roof-character of the Herrin (No. 6) Coal. Isopachs show thickness of Energy (gray) Shale. Note its relation to the prominent north-south cutout. Areas with more than 20 feet of Energy are hachured.

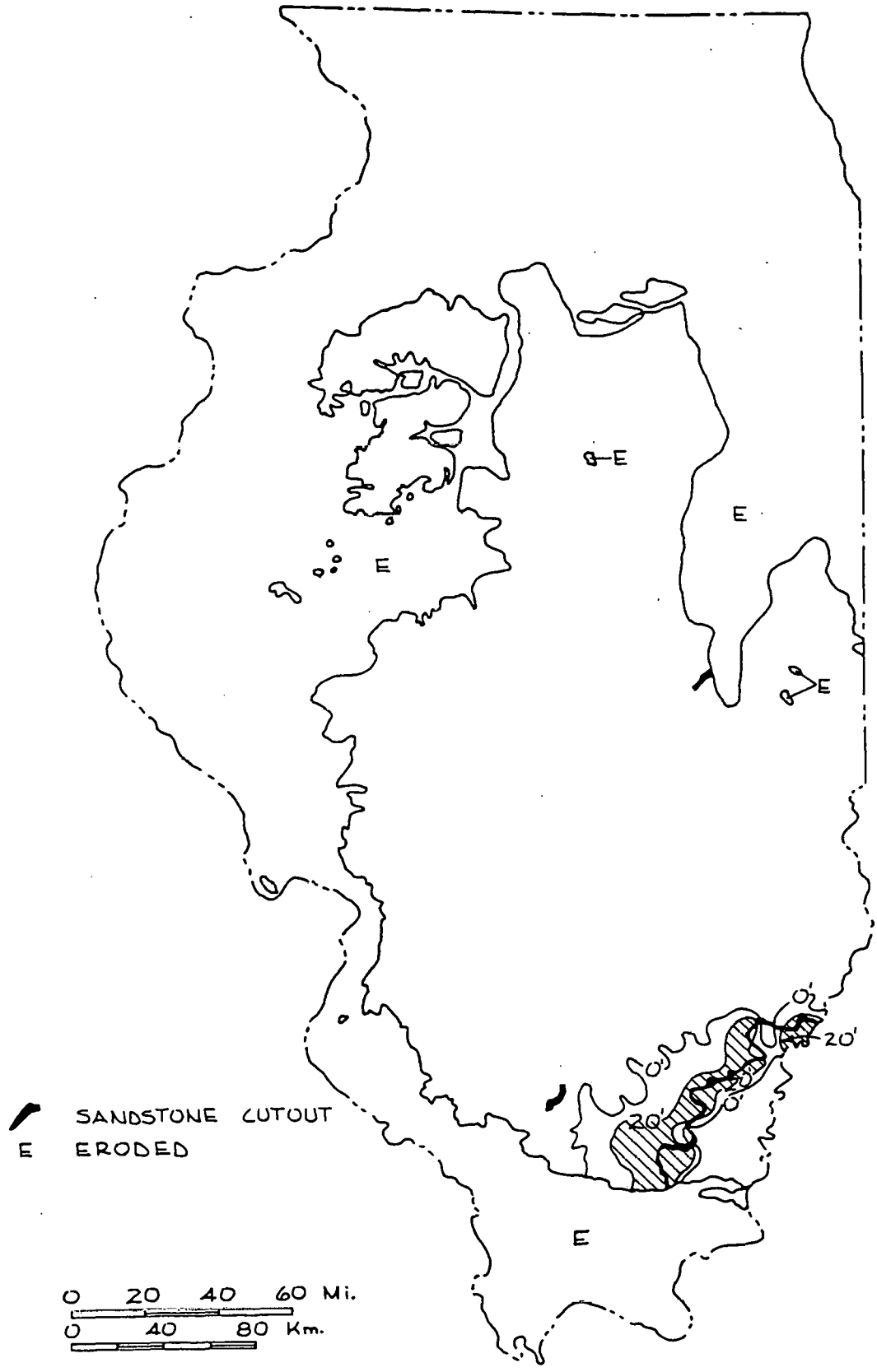


Fig. 4. Generalized roof-character of the Springfield-Harrisburg (No. 5) Coal. Isopachs show thickness of Dykersburg (gray) Shale. Areas with more than 20 feet of Dykersburg are hachured.

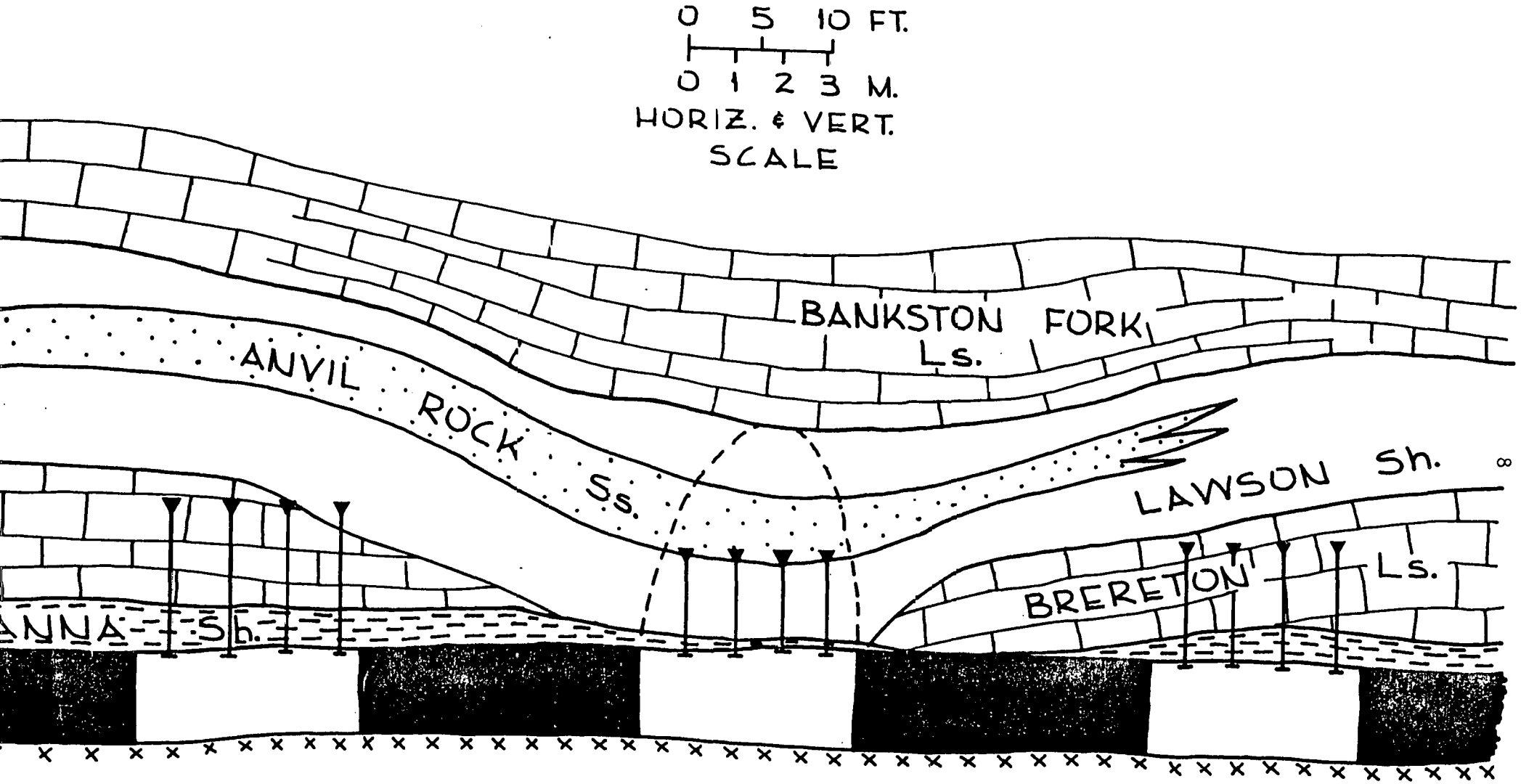


Fig. 5. Generalized section of strata over No. 6 Coal in area of black shale/limestone roof type. Potential roof fall in middle entry where the Brereton Limestone has pinched out.

The lateral rapidity of the stratigraphic changes shown in Figure 5 preclude their being mappable from the normal number of drill holes employed in mine exploration. However, a few holes encountering some of the variation would serve as a warning of what might be expected. Conditions in nearby mining areas should be investigated. The variations appear to result from depositional variation, probably the result of paleotopographic control of the black shale, limestone, and associated strata, and not from erosion of previously deposited uniform layers.

Another example of stratigraphic variation in the roof sequence of the No. 6 Coal is shown in Figure 6. The Anvil Rock Sandstone is here shown in its channel phase and has cut down through the Brereton Limestone and lies very close to the coal. Problems expected in these cases would be: (1) water leaking down through the bolt holes in the two entries on the right, (2) potentially weak roof in the two right entries depending on the nature of the bedding and the cementation of the sandstone, and (3) fractures generated by differential compaction in the roof and consequently poor roof conditions in the entry near the lateral margin of the sandstone body (slickensided fractures parallel to the margin of the sandstone body have been frequently observed). In this latter instance, conditions would be particularly bad if the Brereton Limestone were poorly developed or absent. The Anvil Rock Sandstone situation described here is widely developed; the channels may vary in width from only a few hundred feet to as much as two miles. In places the coal has been completely eroded.

## Gray Shale Areas

Both the No. 6 and No. 5 Coals are characterized by gray shale roof areas occurring along with major sandstone hannel trends: Energy Shale and alshville Channel for the No. 6 Coal, and Dykersburg Shale and Galatia Channel for the No. 5 Coal. The principal mining problems are related primarily to such features as interbedding of roof shale with the coal, shale or siltstone splits in the coal, soft sediment deformation structures in the roof shale, sandstone or siltstone-filled channels in the roof shale with attendant fractures, bedding characteristics in the shale, and siltiness of the shale which affects its general hardness.

The general stratigraphic relations on the lateral margin of the Energy Shale belt is shown on Figure 7. The thickest coal is usually found under the gray shale but as the sandstone channel is approached the coal usually splits, the individual tongues separated by silty shale. In some cases major splitting does not occur and the main bench simply thins towards the channel. The coal is minable only up to a limit usually determined by thickness of the split (or splits) being mined. The shale normally becomes finer grained and softer as it decreases in thickness laterally away from the particular channel active at that time.

The zero isopach line for the gray shale is extremely irregular, sometimes extending for 10 miles or so from the main area. In some places gray shale lenses usually less than 10 feet thick appear to be separated from the main body. These shale "pods" disrupt the continuity of the normal roof strata and commonly cause roof problems. As shown in Figure 8 the Brereton Limestone may thin over the gray shale, thus not constituting a competent roof bolt anchor horizon.

## Associated Features

Detailed mapping in several mines by the Coal Section of the Illinois State Geological Survey has demonstrated numerous small faults and slips which are associated with, or parallel to, thickness changes and pinch-outs in the roof strata of the No. 6 Coal, particularly the Anna Shale and Brereton Limestone. Swarms of curving faults or slips have obviously weakened the roof in the areas of pinch out of the Anna Shale.

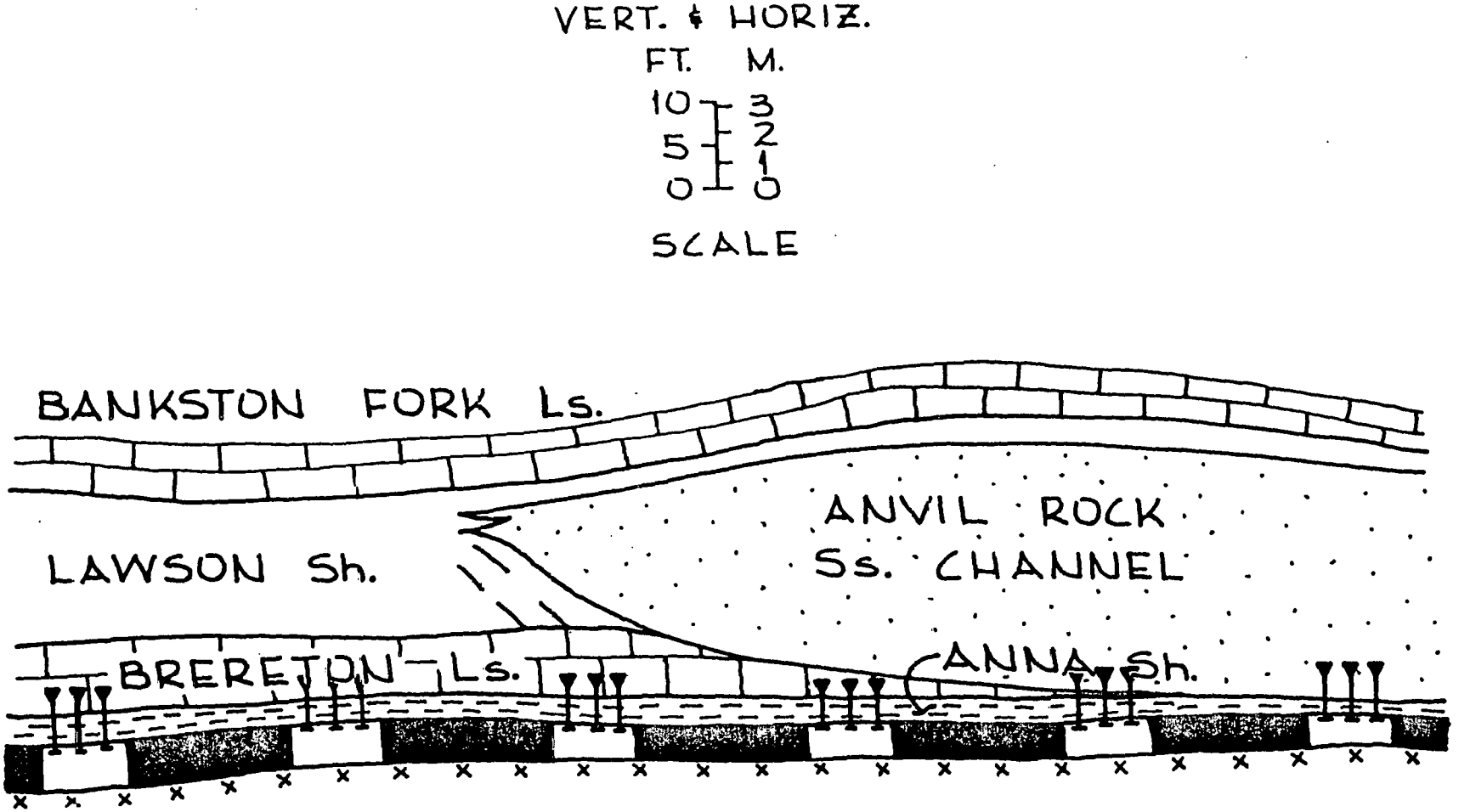


Fig. 6. Cross section showing mining in No. 6 Coal in area of Anvil Rock channel, which interrupts the normal black shale/limestone continuity.

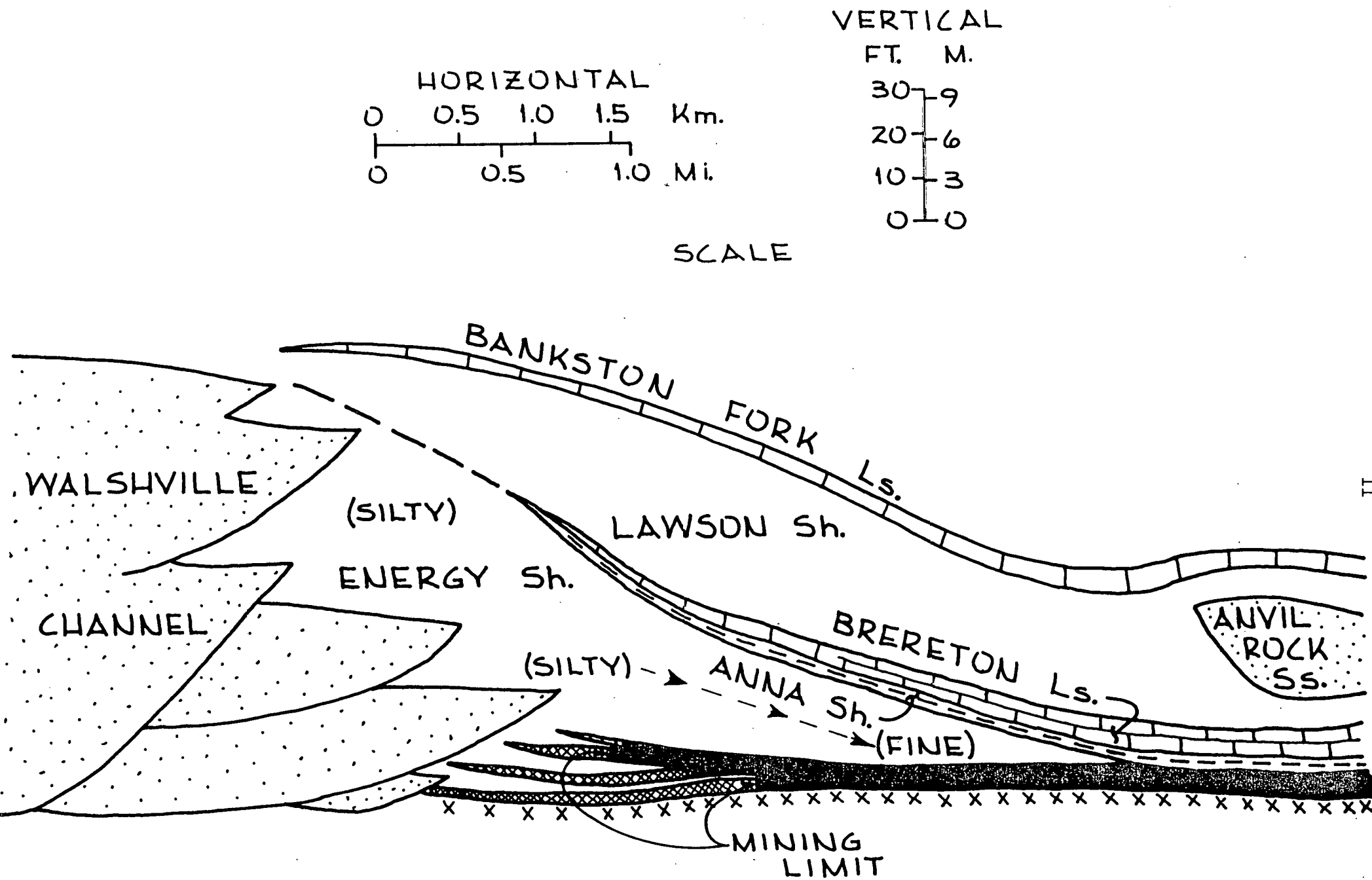


Fig. 7. Cross section showing stratigraphic relations of Energy Shale, Walshville Channel, and normal roof strata of the No. 6 Coal.

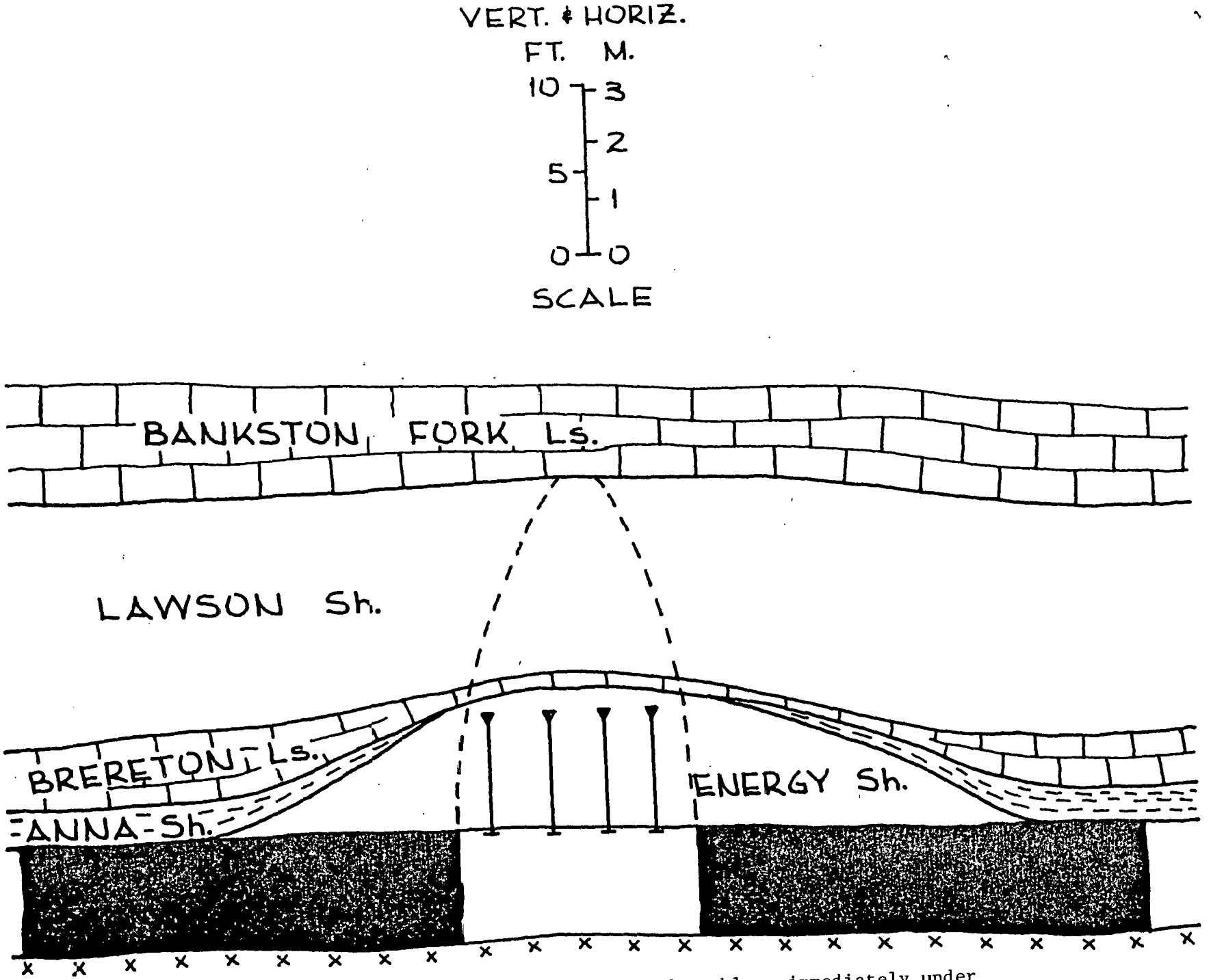


Fig. 8. Cross section showing potential roof problems immediately under a lense of Energy Shale, a condition relatively common adjacent to the major Energy Shale/Walshville Channel trend.

The ISGS has also conducted detailed mapping in No. 6 Coal mines with Energy Shale roofs. Here they demonstrated various features, mostly soft sediment deformation structures, which cause roof instability.

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# EFFECT OF GEOLOGY ON ROOF STABILITY IN ROOM-AND-PILLAR MINES IN THE HERRIN (NO. 6) COAL OF ILLINOIS

Heinz H. Damberger, W. J. Nelson, and H.-F. Krausse

Illinois State Geological Survey
Urbana, Illinois

## ABSTRACT

Roof stability in underground mines in the Herrin (No. 6) Coal of Illinois is dependent upon the lithology and geologic structure of the rocks overlying the coal. Detailed geologic mapping in many mines throughout Illinois has shown that four major roof types exist. They are: (1) the gray shale roof type, composed of gray shale, siltstone, and sandstone associated with crevasse-splay deposits that are from a major contemporaneous channel; (2) the black shale-limestone roof type, consisting primarily of marine fissile black shale and limestone; (3) the wedge-type transitional roof, characterized by large wedge-shaped deposits of gray shale 0 to 25 feet thick and overlapped by black shale-limestone; and (4) the pod-type transitional roof, in which small pods of gray shale less than 10 feet thick are overlapped by black shale-limestone. Characteristic structural features, such as clay dikes, clay-dike faults, and rolls, are associated with each of the four roof types. Within each roof type, however, the lithology and structure tend to be highly variable on the local scale. Exploratory drilling, therefore, cannot be used to map the roof but is only valuable for providing an indication of the range of geologic variability in the roof.

## INTRODUCTION

The lithology and structure of the rocks overlying a coal seam are important to the roof stability of underground mines. Despite this fact, few systematic and well-documented studies of geologic factors in roof stability have been undertaken. The first such detailed investigation at the Illinois State Geological Survey (ISGS) was begun in 1974, and was published recently.[1,2] To date, roof conditions in almost every underground mine in Illinois have been documented, and over 30 sample areas in 11 mines have been studied in detail. In this report we summarize our current knowledge of the effects of geology on roof conditions in the Herrin (No. 6) Coal Member, the principal seam mined underground in Illinois. This report can be considered an updated and revised summary of Krausse et al.[1]

We have always used detailed geologic mapping as our primary method to study mine roofs. Areas averaging about 20 acres are selected for detailed mapping; the usual scale of field mapping is 1:1200, but complex areas may be mapped at 1:600. Within the selected areas we map all accessible headings, and plot data directly on base maps provided by the coal companies. Data plotted include:

lithology of the immediate roof and of overlying units, if visible; geologic structures, such as faults, joints, clay dikes, concretions, and rolls; and mining-induced features, including roof falls, zones of sagging roof, floor heave, and rib rash. Various maps are then prepared from the field maps to display the findings in a useful way.

Our studies have shown that the Herrin (No. 6) Coal has several geologically distinct types of roof. Each type has a characteristic sequence of lithologies and characteristic structural features. In turn, distinctive problems of stability are associated with each roof type.

## GENERAL GEOLOGY

The Herrin (No. 6) Coal Member of the Carbondale Formation accounts for about 42 percent of the delineated coal resources and 80 percent of the current coal production in Illinois.[3] Of the remaining coal resources, 85 to 90 percent lie at depths greater than 150 feet and can be recovered only by underground mining. The distribution of the Herrin Coal in Illinois is shown in figure 1. A schematic cross section of the coal and overlying strata appears in figure 2.

A significant geologic feature of this area is the Walshville channel, which roughly bisects the main Herrin Coal Field from north to south (fig. 1). The Walshville channel is the course of an ancient river system that existed during, and probably before and after the formation of the Herrin Coal. Within the channel there is no coal; instead there are shale, siltstone, and sandstone. Along the margins of the channel the coal commonly is split by layers of clastic material, which probably represent sediments deposited during floods from the main channel.[4,5] The thickest Herrin Coal is closely associated with the Walshville channel, especially in the area of southern Illinois sometimes referred to as the "Quality Circle" (fig. 1).

In several distinct areas adjacent to the Walshville channel, the Herrin Coal is overlain by thick deposits of gray shale, siltstone, and sandstone, known collectively as the Energy Shale Member.[6] Toward the channel, the Energy Shale thickens, locally exceeding 100 feet; upward and toward the channel, the Energy Shale Member coarsens. The gray shale and associated facies show a lobate pattern of distribution along the channel, indicating their origins to be crevasse-splay deposits.[4,6] (A crevasse splay is a wedge of sediments, in effect a small delta, that forms where a stream breaches its natural levee and carries mud, silt, and sand into the surrounding lowlands. Crevasse splays are common in the delta of the Mississippi.[7,8]) The rocks of the Energy Shale Member comprise the *gray shale roof type* cited in Krausse et al.[1,2] and in this paper.

Overlapping the Energy Shale, and lying directly on the coal where the Energy Shale is absent, is a succession of rocks that are dominantly of marine origin. At the base of the succession is the black, generally fissile Anna Shale Member, and above this is the Brereton Limestone Member. Krausse et al.[1,2] called this succession the *black shale-limestone roof type*. The black shale-limestone roof type includes up to seven named members of the Carbondale Formation, from the Anna Shale through the Bankston Fork Limestone Member (fig. 2).

Transitions between the gray shale and the black shale-limestone roof types are widespread. Two broad classes of transitional roof can be defined. One class—the *wedge-type transitional roof*—is characterized by large areas of Energy Shale, 15 to 25 feet thick, which locally wedge out so that black shale and limestone lie directly on the coal. Wedge-type transitional roof occurs around the margins of major lobes of Energy Shale, particularly in the "Quality Circle,"

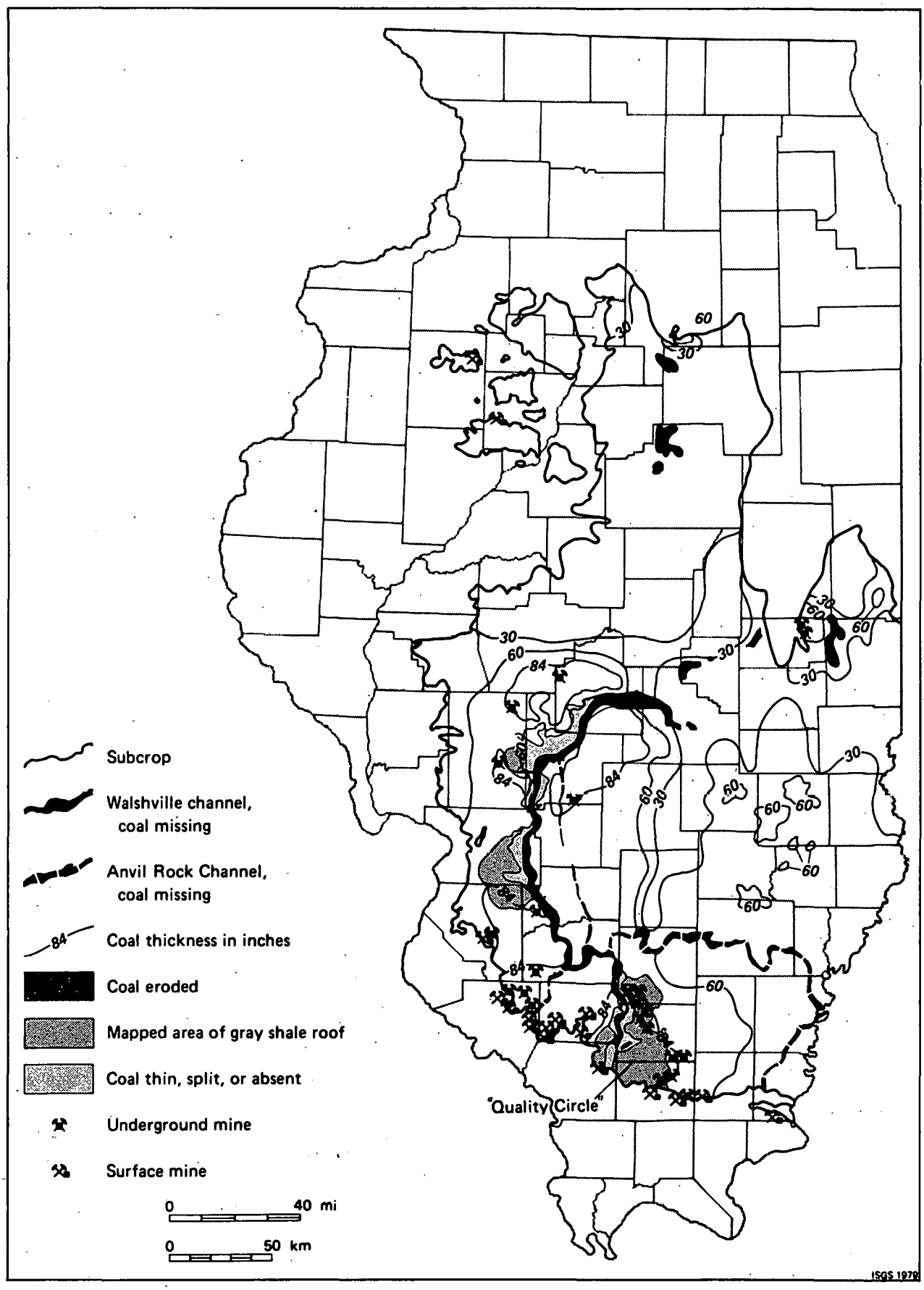


Figure 1. Distribution of the Herrin (No. 6) Coal in Illinois. (From Krausse et al.[1,2])

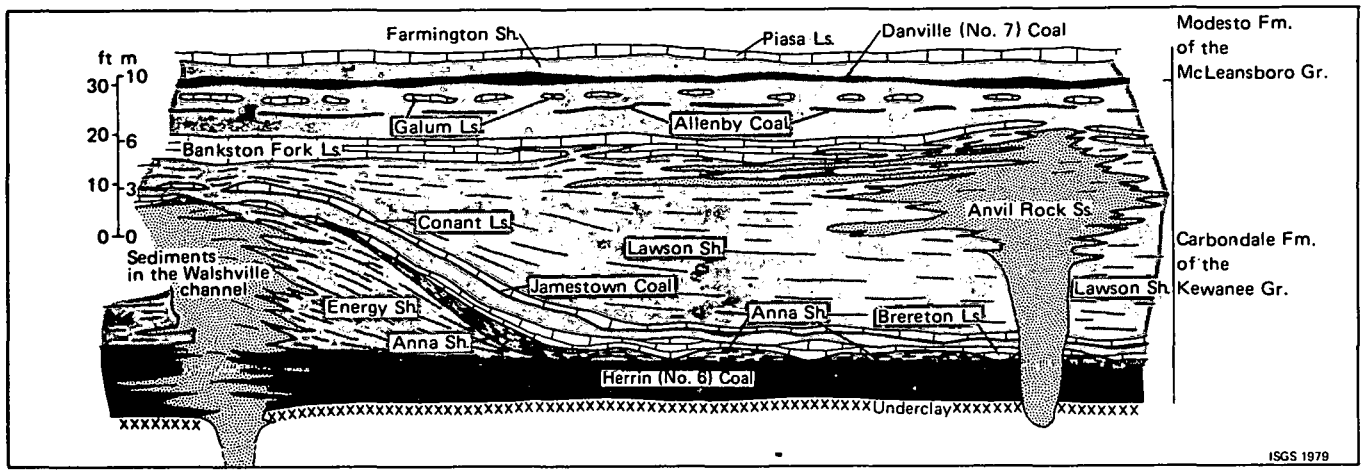


Figure 2. Schematic section of the interval between the Herrin (No. 6) Coal and the Piasa Limestone (from Krausse et al.[1,2] after a sketch by G. J. Allgaier ).

but also in other areas (fig. 3). The second class of transitional roof is the *pod-type transitional roof*, in which Energy Shale occurs in pods that are from a few tens of feet to a few hundred feet across, and vary in thickness from 2 to 10 feet. The pods of gray shale are overlapped by the rock units comprising the black shale-limestone roof type.

Figure 3 shows the general regional distribution of these four roof types of the Herrin Coal in Illinois. The map is based primarily on observations in mines around the edges of the basin and inferences from drill-hole data in other parts; it is subject to future revisions.

## Gray Shale Roof Type

The gray shale roof type, which is composed of Energy Shale, has been best studied in the "Quality Circle" (fig. 3). In this area, four active mines lie completely under this type of roof, and three lie partially under it. Two active mines in Douglas County of east-central Illinois also have roofs of Energy Shale. Still more areas of gray shale roof are known from drilling data and from notes on abandoned mines (fig. 3).

The gray shale roof type includes several distinct lithologies (fig. 4): (1) lenses of dark-gray, thinly laminated shale, (2) large areas of medium-gray, poorly laminated shale, silty to nonsilty, and (3) areas of sandstone and siltstone, often planar-bedded, near the Walshville channel. Other rock types have also been observed in mine roofs, but they are less common than those mentioned above. As we have noted, the Energy Shale coarsens upward and toward the Walshville channel.

Dark-gray shale. Dark-gray, carbonaceous, thinly laminated shale is fairly widespread in the immediate roof of mine B[1,2] and has been observed in several other mines in the "Quality Circle" of southern Illinois. This dark shale is generally less than 5 feet thick and occurs as isolated pods or as irregularly interconnected areas that are usually several hundred feet across (fig. 5). Locally, the dark-gray shale is more than 10 feet thick. Dark-gray shale is variable as a roof material: in some areas it makes sound, stable roof, but elsewhere it is highly prone to slabbing and is removed as "draw slate."

Medium-gray shale. In all mines with the gray shale roof type, medium-gray shale is the dominant lithology. It overlies the dark-gray shale or forms the immediate roof of the coal, and ranges from 20 to as much as 100 feet in thickness, except close to the Walshville channel, where coarser lithologies dominate. The shale exists in various states, from a soft,

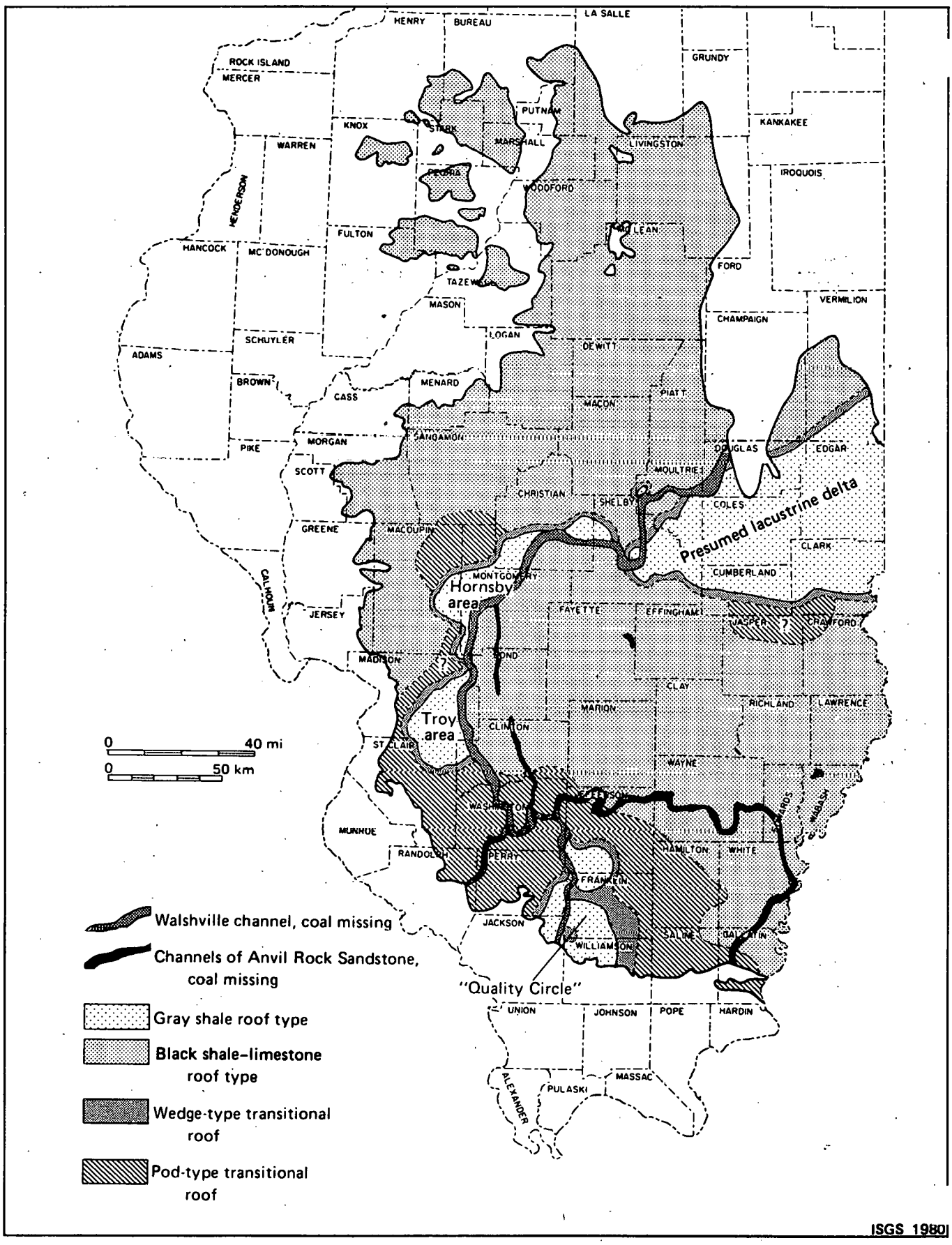


Figure 3. Regional distribution of roof types of the Herrin (No. 6) Coal in Illinois.

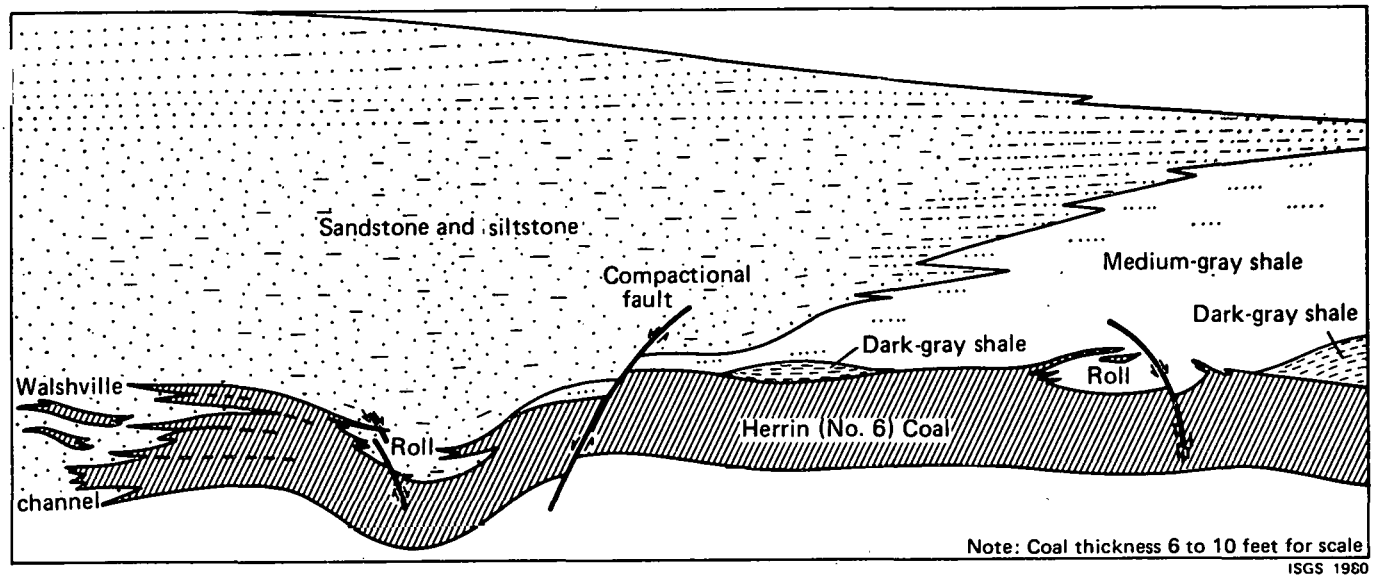


Figure 4. Schematic section of lithologies and structures of gray shale roof type.

silt-free, poorly laminated shale or mudstone, to a massive or faintly laminated siltstone.

In general, the finer grained types of shale are more susceptible to moisture-slaking and fail more readily than the silty shales. Massive falls are common in the softer mudstones, particularly in entries subjected to varying temperatures and humidity. Carbonaceous partings and streaks of coal ("riders") are also blamed for many roof falls. In some mines, however, little trouble has been encountered with the roof where wide areas of the medium-gray shale exist.

Siltstone and sandstone. Coarse-grained clastic rocks are not common in the roofs of most mines except in those close to the Walshville channel, where such rocks may lie close to or directly upon the coal (fig. 4). The texture, composition, and stability of these clastic rocks vary considerably. Well-cemented, massive siltstones and sandstones make excellent roof material, but they are relatively rare in the Energy Shale Member. Thinly laminated and planar-bedded siltstones and sandstones, with abundant mica and carbonaceous plant debris on the bedding surfaces, are more common in this member. This rock separates into slabs along partings, and is notoriously difficult to support. Mechanical bolts with anchors all set to the same height may exacerbate the problem by placing all the weight on a single bedding plane (fig. 6). Thus, it is not uncommon to see massive roof falls in this siltstone or sandstone that propogate upwards after the initial fall (fig. 7). Resin bolts, or mechanical bolts of varying length, or trusses probably would provide better roof control in this kind of planar-bedded rock.

Additional problems appear when the sandstone contains water. Where water-bearing sandstone overlies gray shale, the water may weaken the shale below and cause severe roof instability. Of course, the water creates unpleasant working conditions for miners, and may cause heaving of the underclay in the mine.

Other features hampering mining near the Walshville channel include split coal, steeply dipping coal, large rolls, wash-outs, and compactional faults (fig. 4). More information on these features can be found in Nelson[5] and Krausse et al.[1,2]

Rolls. Characteristic structural features of gray shale roof are rolls—elongate protrusions of roof material into the coal seam. They are generally lens-shaped in cross sections and are separated from the main roof by one or

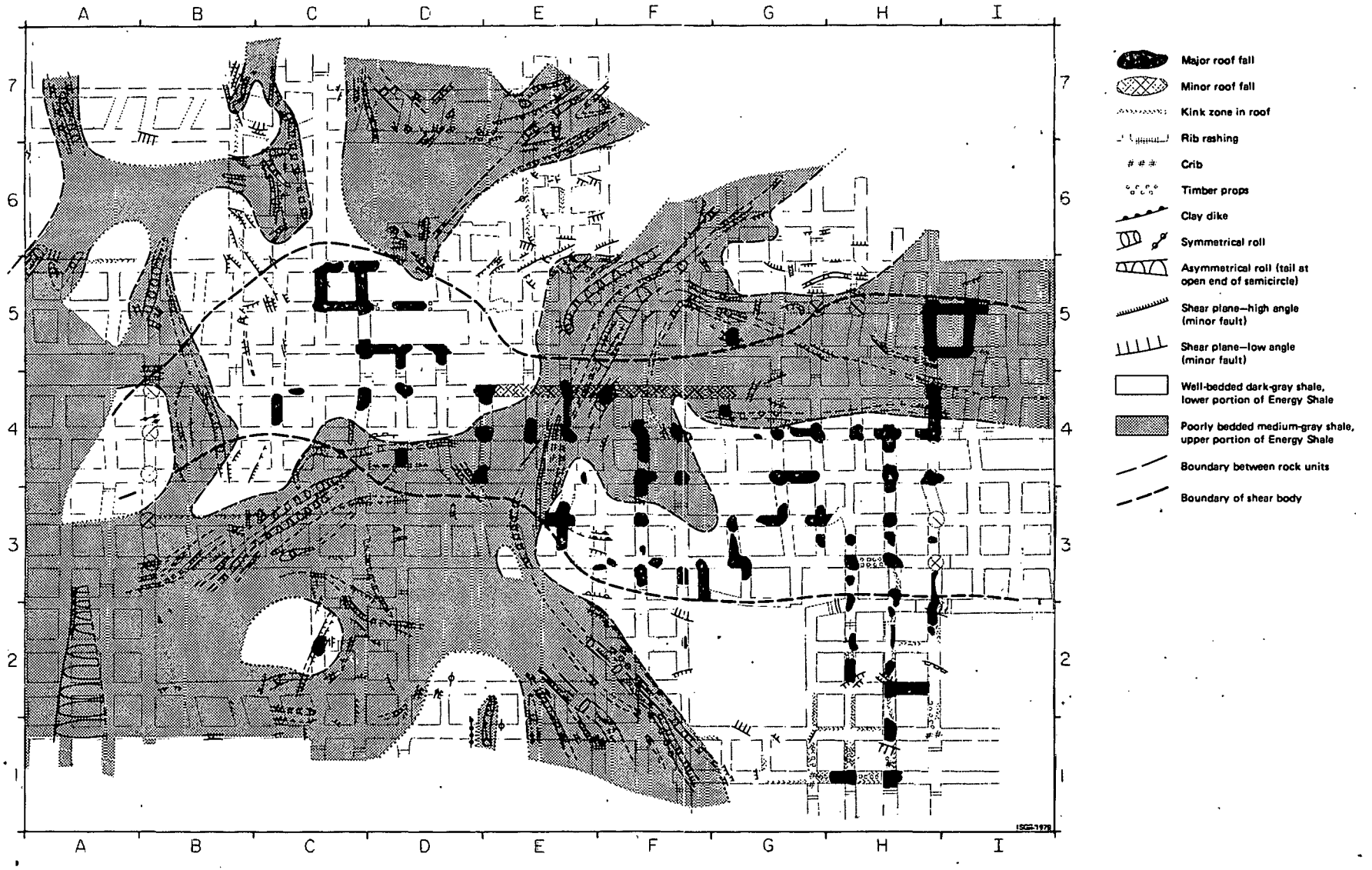


Figure 5. Patchy distribution of dark- and medium-gray shales and occurrence of rolls in a study area within the gray shale roof [illegible] (from Krausse et al.[2]).

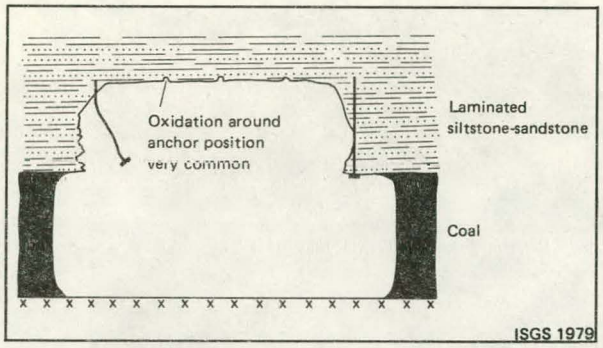


Figure 6. Laminated siltstone-sandstone, often with mica and carbonaceous material on bedding surfaces, may fail along bedding surface at level of anchoring of mechanical bolts (from Krausse et al.[2]).

more coal "riders" that split from the main seam. Most rolls contain compactional faults which, together with the coal "riders," create unstable roof conditions (figs. 4 and 8). The entire mass of a roll can fall out of the roof unless it is promptly supported. It may be desirable to deliberately remove rolls, especially in entries that are needed for extended periods of time.

Rolls are small and rare in dark-gray shale, but numerous in medium-gray shale. In mine B, rolls tend to run parallel with the boundary between dark- and medium-gray shale[1,2,9] (fig. 5) and are predominantly found under the medium-gray shale roof. In mines where the roof is uniformly gray shale or siltstone, rolls appear to be less common. Large and complex rolls occur close to the Walshville channel in silty shale, siltstone, and sandstone[1,2] (fig. 9). Below such rolls the coal may nearly pinch out, or dip so steeply as to be impossible to mine economically.

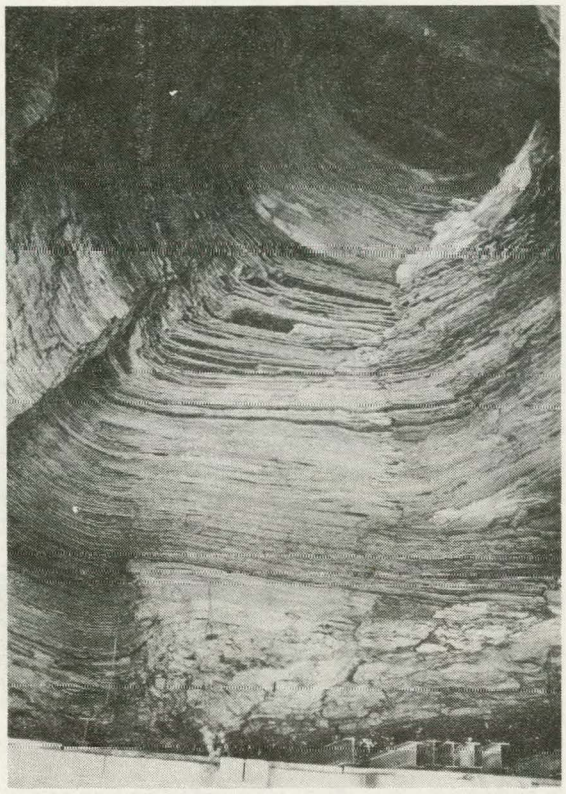

Figure 7. Roof failure in planar-bedded, fine-grained sandstone (from Krausse et al.[2]).

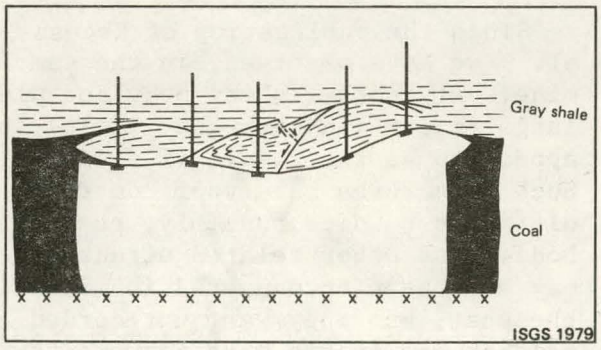


Figure 8. Rolls in roof cause instability and unevenness of roof (from Krausse et al.[2]).

Gravitational slides and slump-faults. In some areas, gray shale roof contains large bodies of intensively sheared, slumped, and disturbed rock that is extremely difficult to support. The best-documented of these occurrences is the "shear body" in mine B.[1,2,10] This shear body is at least 1,800 feet long, and 150 to 500 feet wide; it is composed of deformed shale, siltstone, and other rocks that apparently slid from the upper portion of the Energy Shale Member to a position just above the coal (figs. 10 and 11). The sliding is believed to have occurred before the rocks were completely lithified, and could have been the result of rapid loading by siltstone and

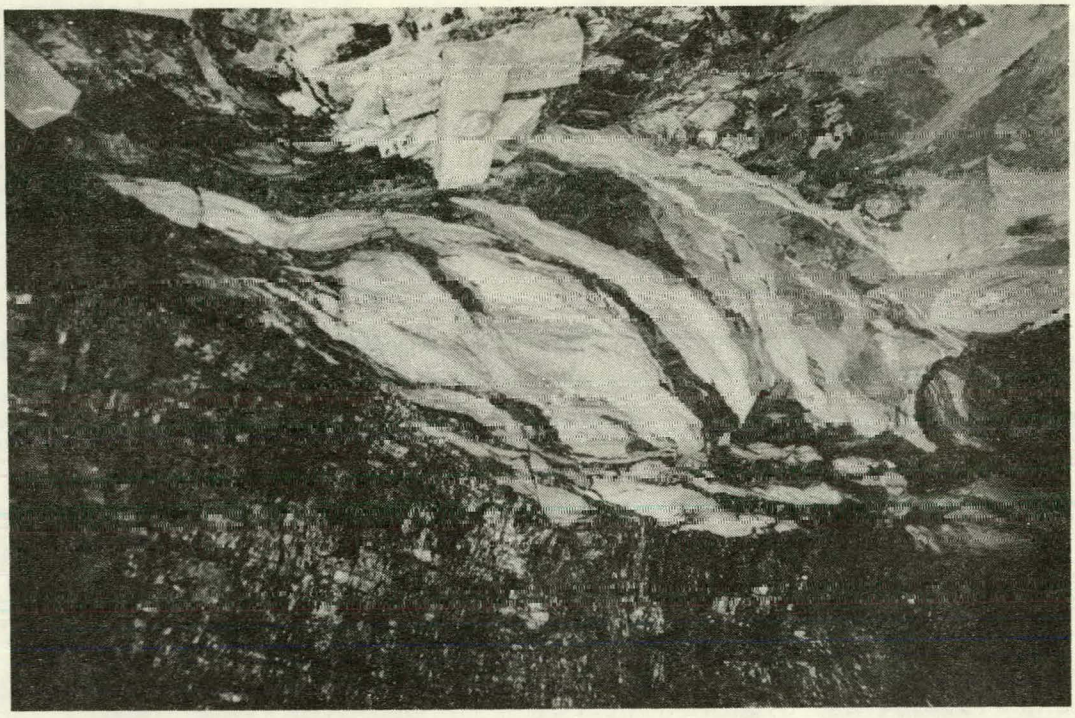

Figure 9. Large complex roll of sandstone and siltstone (from Krausse et al.[2]).

sandstone on the underlying shale. Nelson and Ledvina[10] proposed that the coarser grained sediments may have moved downward, displacing clay that squeezed upward as diapirs (mudlumps).

Since the publication of Krausse et al.[1,2] we have observed, in the same mine, yet another shear body and also large faults (in dark-gray shale) that appear to be the result of slumping. Such structures make roof control difficult. Quite possibly, shear bodies and other related structures may have been encountered in mines in the past, but they went unrecorded because geologists were unaware how such structures make roof control difficult.

### Black Shale-Limestone Roof Type

The black shale-limestone roof type, though probably the most widespread roof type of the Herrin Coal (fig. 3), is currently exposed only in two active mines; a third mine lies partially under this roof type.

Anna Shale. The Anna Shale is the lowest member of the black shale-limestone roof succession (fig. 12). Statewide, the Anna Shale ranges in thickness from 0 to over 6 feet; the normal range is 0 to 4 feet. In much of Illinois the Anna Shale is discontinuous, occurring in pods, lenses, or interconnected irregular areas that are usually several hundred feet across (fig. 13). In southern Illinois, however, the shale appears to be more uniform and continuous.

The lower portion of the Anna Shale (1 to 2 feet) is typically hard, smooth, and fissile, and is prone to break into slabs along joint planes (fig. 14). Oval concretions, often 2 to 3 feet in diameter, pose an additional hazard to miners (fig. 12). Aside from these concretions, however, fissile Anna Shale generally makes fairly sound roof if the overlying strata are competent. Nevertheless, if the upper portion of the Anna Shale is soft, weakly bedded, and mottled—as it is in some mines—it can be difficult to hold in place.[1,2]

Brereton Limestone. The Brereton Limestone overlies the Anna Shale and, like the shale, is locally discontinuous and variable in thickness over much of Illinois (figs. 12 and 13). This unit is the key to stability of the main roof in black shale-limestone areas. Where the limestone is more than 2 to 2½ feet

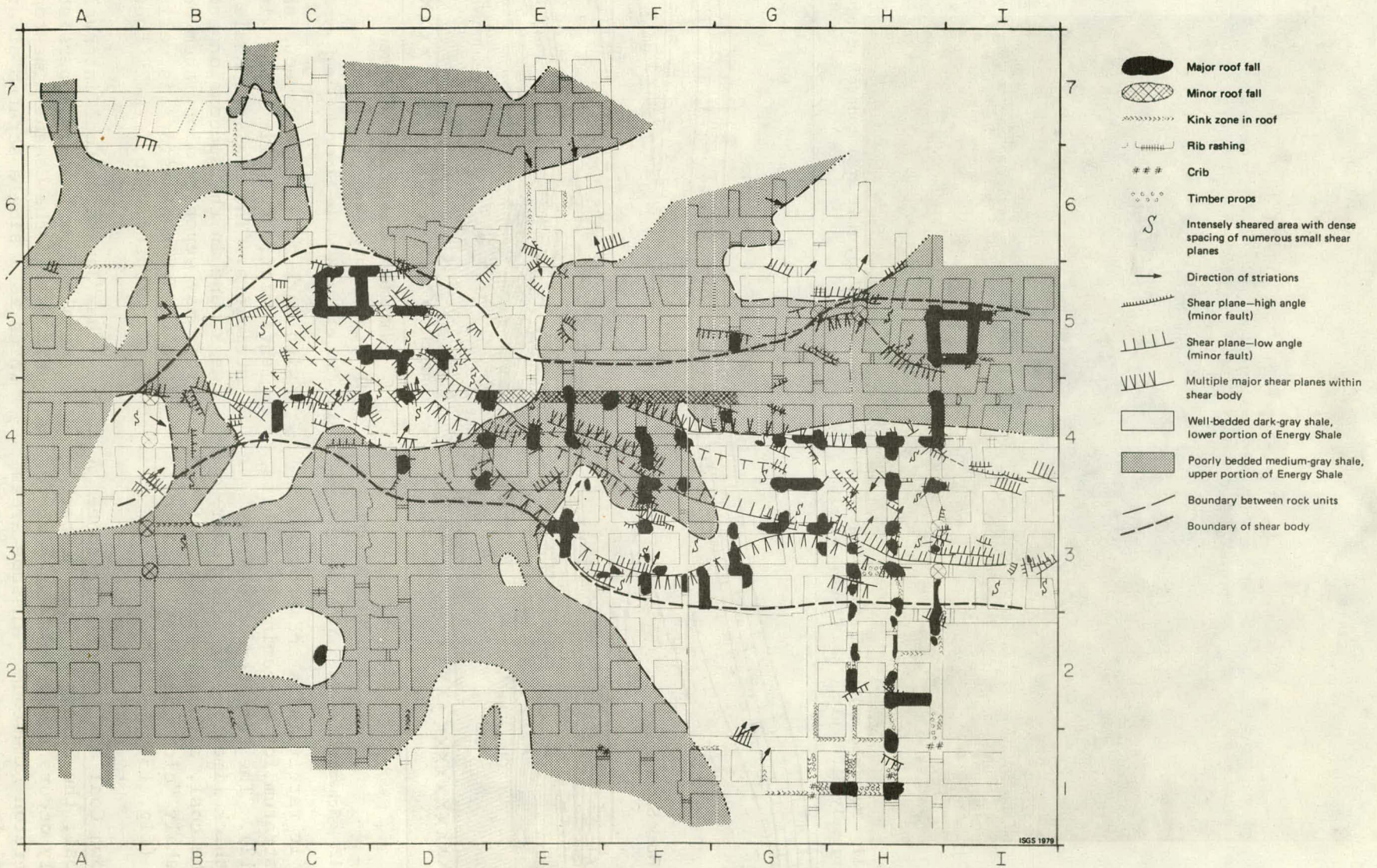


Figure 10. Outline and major shear structures of the shear body in mine B (from Krausse et al.[2]) Numerous roof failures are almost entirely attributable to intensive shearing of rocks within the shear body (compare fig. 5).

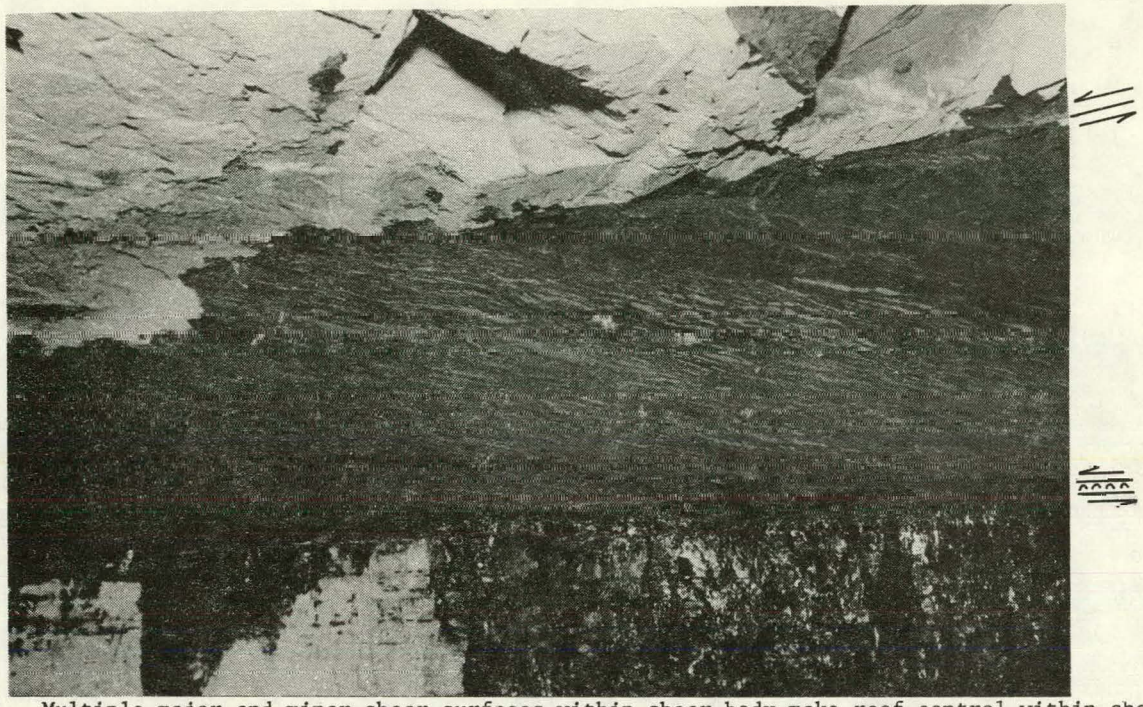

Figure 11. Multiple major and minor shear surfaces within shear body make roof control within shear body extremely difficult (from Krausse et al.[2]).

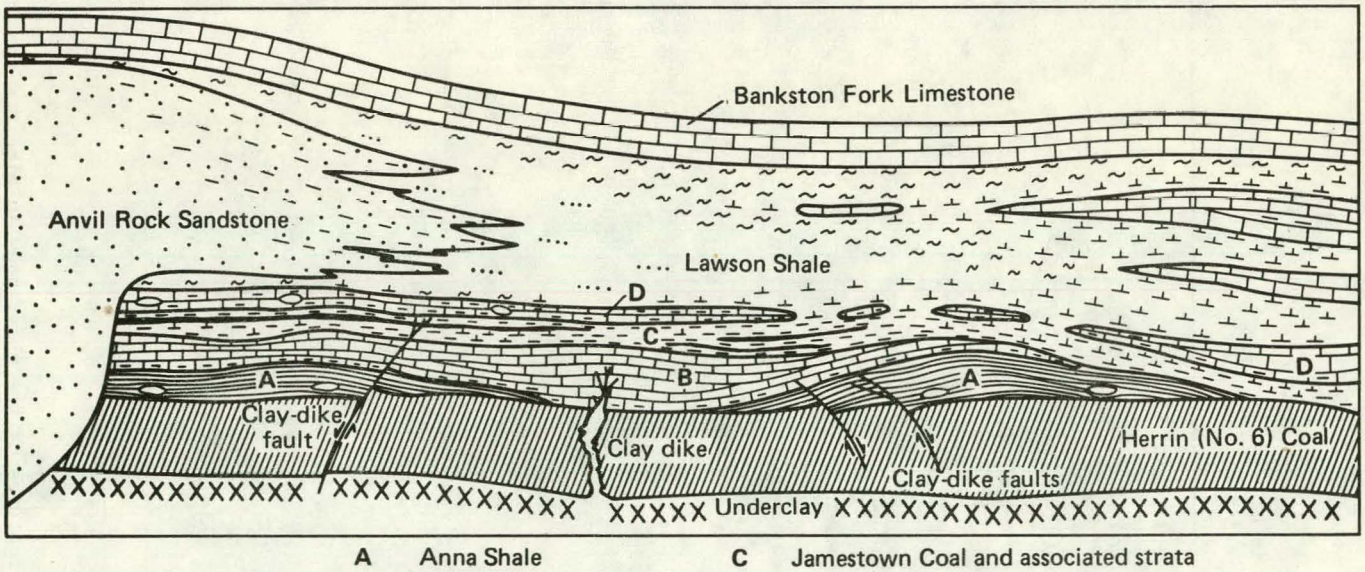


Figure 12. Schematic section of lithologies and structures of black shale-limestone roof type.

thick, it rarely fails and provides a solid stratum for anchoring roof bolts (fig. 12). The best conditions are where thick limestone directly overlies the coal, although slabbing of the basal shaly "clod" may still be a roof hazard (fig. 15).

Jamestown Coal and Conant Limestone. A thin coal, the Jamestown Coal Member, commonly occurs a short distance above the Brereton Limestone. In some areas the coal is replaced by or associated with dark-gray, carbonaceous shale and lenticular, shaly limestone with coal stringers. The coal and shale are a weak zone in the succession and the roof may separate along them unless the Brereton Limestone is well developed (greater than 2 to 2½ feet thick).

The "Jamestown Coal interval" is generally overlain by a persistent unit called the Conant Limestone, which is a single stratum of argillaceous limestone or, in some mines,

calcareous fossiliferous shale. Except in southwestern Illinois, the Conant Limestone is thin (less than 1 foot thick) and adds only limited strength to the roof succession. It is not a secure zone for anchoring roof bolts except in the rare instances where it exceeds 2 feet in thickness.

Lawson Shale. The Lawson Shale is generally 5 to 15 feet thick in west-central and southwestern Illinois west of the Du Quoin Monocline. East of the monocline, the Lawson Shale is 30 to 50 feet thick, and usually consists of laminated medium- to dark-gray, often silty shale and siltstone. Since the Brereton Limestone is persistently thick and competent in this region, the Lawson Shale is rarely exposed in roof falls. To the west, where the Lawson Shale is thinner, it is commonly exposed because of failure of the underlying units. From St. Clair County northward, the usual lithology is dark-gray to greenish, mottled, poorly bedded shale or claystone. This is a weak unit; it often falls from the base of the Bankston Fork Limestone (fig. 16). In Randolph and Perry Counties, the Lawson Shale is well bedded, calcareous, and often interlayered with limestone. This material is strong, and falls are rare even where the Brereton Limestone is thin or missing.

Anvil Rock Sandstone. The Anvil Rock Sandstone is closely related to the Lawson Shale and usually overlies it or interfingers it laterally. It occupies a widespread system of channels that are locally eroded through the Herrin Coal (figs. 1, 2, and 3). A sheet sandstone facies may be found adjacent to the channels. The sheet sandstone facies typically is thinly laminated, planar-bedded siltstone or fine-grained sandstone—lithologies that are similar to certain lithologies of the Energy Shale Member, as described above. Like the sandstones of the Energy Shale, the Anvil Rock Sandstone may contain water that can seep into mines. Both the Anvil Rock Sandstone and the Lawson Shale are overlain by the Bankston Fork Limestone. Rock units above the Lawson Shale and the Anvil Rock Sandstone have little effect on roof control.

Clay dikes and clay-dike faults. Clay dikes are irregular vertical or inclined intrusions of clay along fissures in the coal and overlying strata. Wherever clay-dikes are not vertical, they are associated with faults and called clay-dike faults;[1,2] these clay-dike faults often pass laterally into clay dikes.[1,2,12,13,14]

Clay dikes and clay-dike faults are abundant in most mines with black shale-limestone or pod-type transitional roof. They are less frequent in the gray shale and the transitional roof types of southern Illinois. Clay dikes and clay-dike faults are common in the northwestern, west-central, and southwestern parts of the Illinois Basin Coal Field, but are scarce in the southern and southeastern parts of the field.[12,13] Little is known of their distribution outside of mined areas.

Krausse et al.[1,2] documented that the local distribution and orientation of dikes and clay-dike faults is strongly influenced by lithologic patterns in the immediate roof of the coal (fig. 13). Clay-dike faults and dikes run subparallel with boundaries between rock types. Some large clay-dike faults, such as the fault at H7 to H12 in figure 13 cut across lithologic boundaries; their distribution may be controlled by larger scale variations of lithology that are unrecognizable by in-mine mapping.

Clay-dike faults and clay dikes weaken the roof. They are not likely to cause failure of limestone roof; however, even several feet of limestone have occasionally fallen where penetrated by dikes or faults. Areas that bear special watching are intersections that are crossed by dikes and faults, headings with faults parallel to the rib, or places where several dikes or faults meet in the roof.

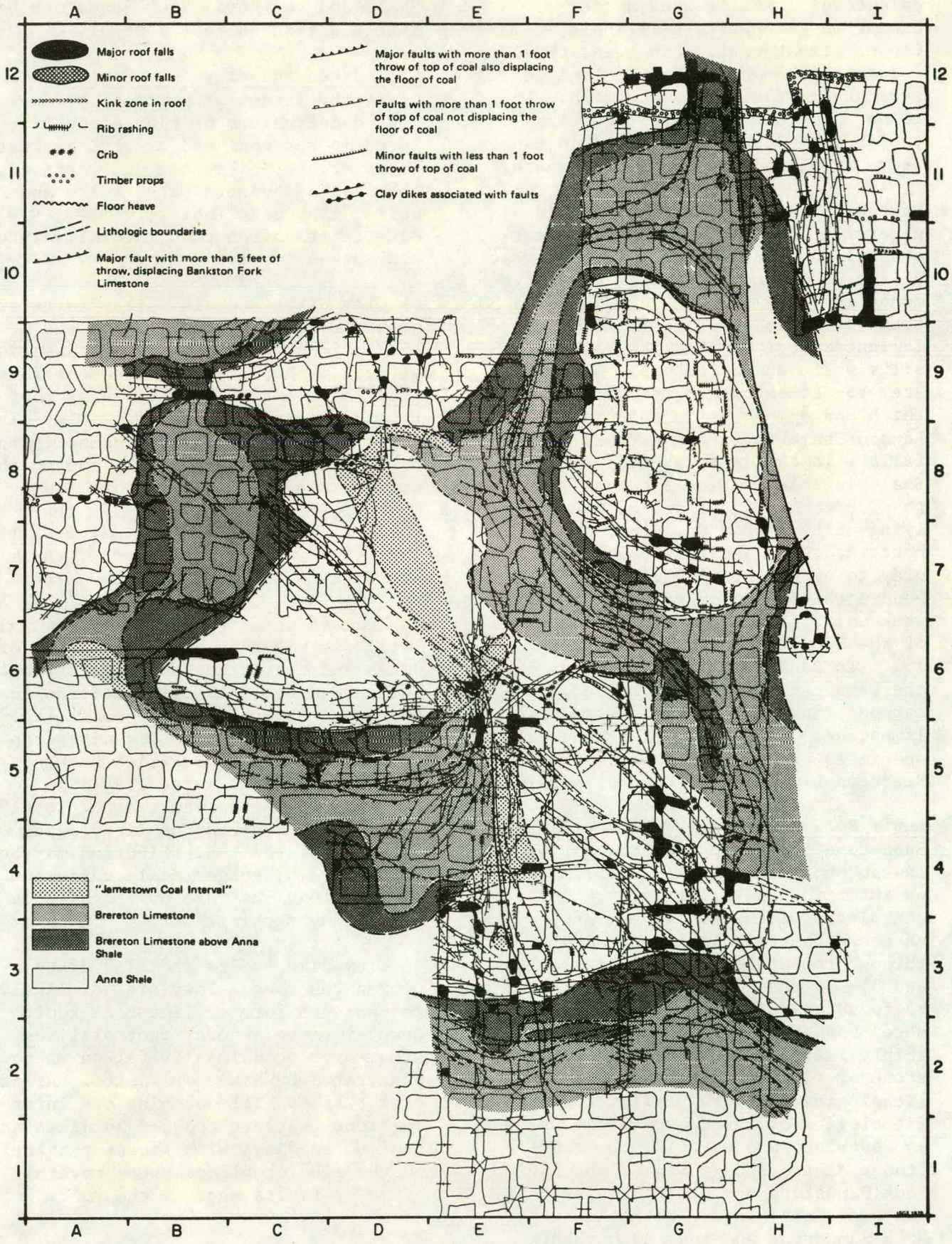


Figure 13. Typical distribution of lithologies, clay dikes, and clay-dike faults in a study area of black shale-limestone roof type (from Krausse et al.[2]).

Figure 14. Prominent joints in lower fissile portion of Anna Shale (from Krausse et al.[2]).

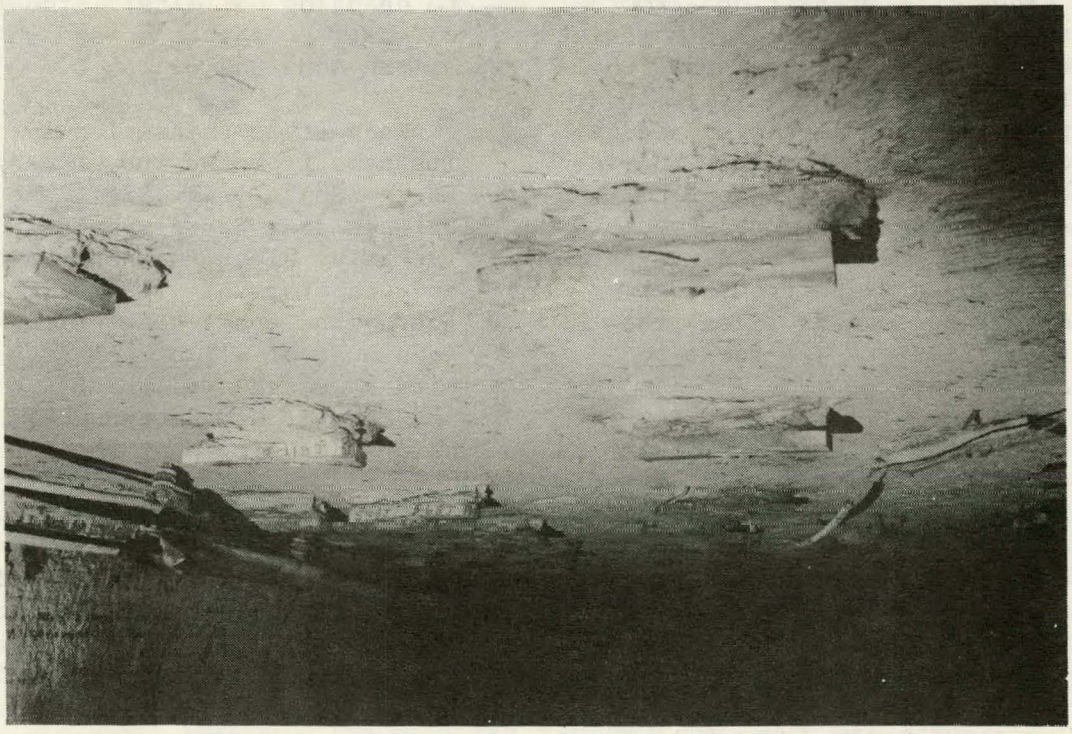

Figure 15. Thick Brereton Limestone forming stable roof in black shale-limestone roof type. A few inches of soft flaky "clod" have fallen, except above header boards (from Krausse et al.[2]).

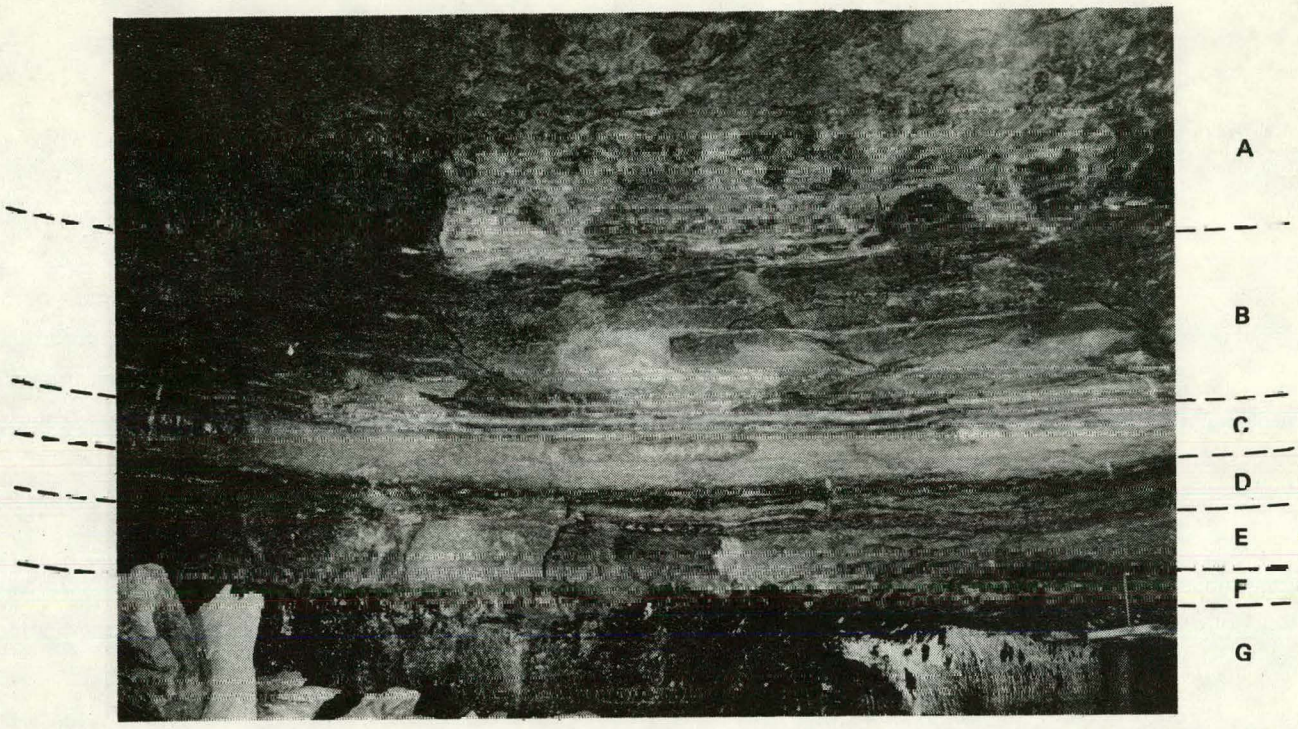


**Figure 16.** Roof failure predominantly in weak mottled Lawson Shale (A and B). Note absence of Anna Shale and lenticular, thin Brereton Limestone (E). The Conant Limestone (C) did not provide enough strength to stabilize the roof below very weak, heavily mottled Lawson Shale (A). D = "Jamestown Coal interval," F = "clod," G = Herrin Coal. (From Krausse et al.[2]).

## Transitional Roof Types

The *wedge-type transitional roof* extends over a small area. Six active mines around the fringes of the "Quality Circle" have this roof type in at least part of their workings. Wedge-type roof probably forms belts dividing the gray shale roof type from black shale-limestone or pod-type of roof (fig. 3). Belts of wedge-type roof may vary in width from a few hundred feet to several miles; the exact boundaries are difficult to delineate as yet.

The wedge-type transitional roof is characterized by the presence of large irregular bodies of Energy Shale, which are several hundred to more than 1000 feet across and mostly 10 to more than 25 feet thick (fig. 17). These bodies of gray shale may or may not be connected with the major lobes of Energy Shale. Medium-gray, poorly bedded, silty or silt-free shale is the typical lithology; lenses of dark-gray shale may be present at the base. Roof control within the gray shale area tends to be difficult, particularly in long term entries subjected to changing humidity; large falls commonly propagate upward to the base of the Anna Shale.

Wedge-type transitional roof takes its name from the abrupt wedging of Energy Shale (fig. 17). The contact between gray Energy Shale and the overlying black Anna Shale is sharp and unconformable. This suggests that Energy Shale may have originally overlain most or all of the area now occupied by wedge-type transitional roof, and that it was subsequently eroded before deposition of the Anna Shale.

Rolls of gray shale are the major structural anomalies of wedge-type roof. Most rolls strike parallel with the black-shale/gray shale boundary. These rolls are filled with medium-gray shale but are overlain by Anna Shale. Like rolls in the gray shale roof type, these structures create difficult roof conditions.[1,2,16]

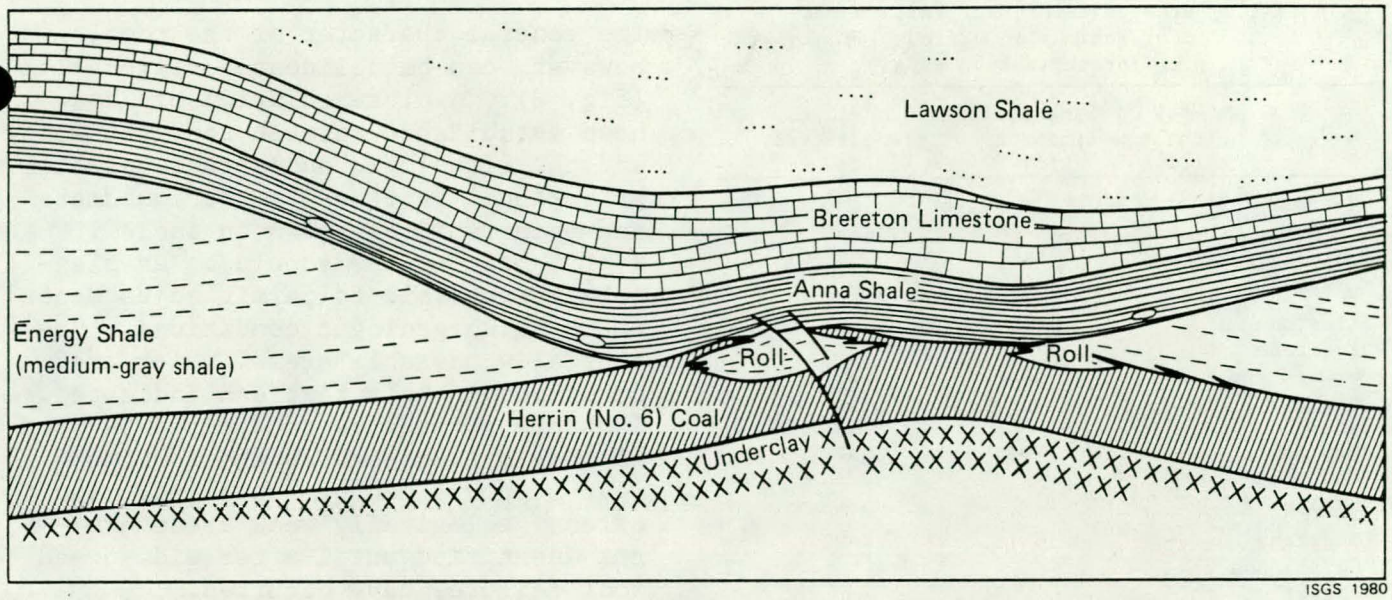


Figure 17. Schematic section of lithologies and structures of wedge-type roof.

The *pod-type transitional roof* is widespread in southern Illinois (fig. 3); seven or eight active mines have this roof type. West of the Walshville channel in central and southwestern Illinois, it is the dominant roof type. It also extends from the border of wedge roof in Williamson County as far east as Gallatin County. Little is known about the presence of gray shale pods in the deeper part of the Illinois Basin, where no mining has yet occurred. Data from drilling, however, indicate that pod-type transitional roof probably occurs in much of Franklin, Jefferson, and Hamilton Counties (fig. 3).

Small irregular pods of medium- to dark-gray Energy Shale below Anna Shale or Brereton Limestone are distinctive features of this roof type. The pods range from a few feet to several hundred feet in diameter, and the gray shale is typically 2 to 5 feet thick, although locally it reaches 10 feet or more. The gray shale is soft, weak, sensitive to moisture, and difficult to support. This shale may be so dark that it is difficult to distinguish from Anna Shale. In many areas, the Brereton Limestone tends to pinch out over pods of gray to dark-gray shale; thus, not only the immediate roof but also the main roof may fail.

In most areas of pod-type transitional roof that have been mapped in detail, Energy Shale covers less than one-fourth of the total area. Outside of the pods, prevailing conditions are comparable to those of the black shale-limestone roof type. The high degree of variability in stability of various lithologies is illustrated in table 1, a compilation of results from several study areas in each roof type. Gray shale that forms pods has the highest incidence of roof failure.

## Other Factors in Roof Stability

Several systems of *tectonic faults* exist in southern Illinois and have important effects on roof stability in mines of the region. An account of these faults and their effect on mining is beyond the scope of this paper, however. The interested reader is referred to Brownfield,[18] Keys and Nelson,[19] Bristol and Treworgy,[20] Krausse et al.,[1,2] and Nelson and Krausse.[21] Several additional manuscripts on faults in the coal fields of Illinois are in preparation at the Illinois State Geological Survey.

*Joints* have a relatively minor influence on roof stability in the mines we have studied. Occasionally, problems develop where well-developed joints run parallel with mine openings.

TABLE 1. Percentages of roof failures at intersections in various lithologies for three roof types*

| | Gray shale (%) | Black shale/ limestone (%) | Pod-type transitional (%) |
|---|---|---|---|
| Dark-gray shale | 4-10 | — | 38-58 |
| Medium-gray shale | 0-1 | — | |
| Siltstone/ sandstone | | | |
| Wet | 16-39 | — | — |
| Dry | 0-9 | — | — |
| Anna Shale, no limestone | — | 19-33 | 28-70 |
| Limestone over Anna Shale | — | 15-16 | 4-22 |
| Limestone as immediate roof | — | 0-1 | 2-3 |
| Other lithology | — | 0-19 | 36-38 |

*Compiled from several study areas at intersections in main entries that were mapped several months or years after mining.

Joints in the Anna Shale and in the dark-gray, laminated facies of the Energy Shale permit slabbing of the immediate roof, but these joints rarely extend more than a foot or two upward and so do not weaken the main roof. Other lithologies rarely have joints.

The mechanical properties of rocks have a major influence on the stability of mine openings. These properties are geological factors in that they are determined by the depositional and deformational history of the rocks. Research on this subject is underway at the Survey, and the results will be reserved for future reports. Findings by Ganow,[22] Coulson,[23] Mesri and Gibala,[24] (1971) and others are summarized in Krausse et al.,[1] and Bauer[25,26] has provided index numbers from tests on cores.

## CONCLUSIONS

Our studies demonstrate that roof conditions in underground mines are highly variable on the local scale. The general character of the roof, however, can be delineated regionally (fig. 3). A close relationship has been established between roof stability and the lithologic and structural distributional patterns in the immediate and main roof, as shown in table 1. Roof control plans should be as flexible as possible to permit adjustments to changing geologic conditions. Potentially unstable areas, which are described in this text and indicated in table 1, must be recognized as soon as they are exposed so that optimum roof supports can be installed. Too often, geologically weak areas are not identified until after widespread roof failures have occurred.

We have also found that during exploration, one cannot hope to delineate the intricate local distributional patterns of roof lithologies. Through drilling, however, the roof type and the range in lithologic variability within the prospect can be determined, and these findings will have statistical validity if a sufficient number of cores are drilled. Furthermore, drilling can be used to select geologically optimum locations for key installations such as shaft bottoms, belt transfer points, and workshops.

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# Premining Investigations and Their Use in Planning Ground Control in the Illinois Basin Coal Mines

William M. Kester*, Yoginder P. Chugh**

*Manager, Underground Mine Planning
AMAX Coal Company, Indianapolis, IN

**Department of Mining Engineering
Southern Illinois University, Carbondale, IL

Abstract. Over the past five years the authors have conducted premining investigations for four virgin areas and studied ground control problems in several operating coal mines in the basin. Premining investigations involved detailed geologic descriptions of 2-inch, 4-inch, or 6-inch diameter cores and preparation of maps pertinent from ground control point-of-view. Specifically, facies changes in the immediate roof, interval to limestone bed, if any, thickness of limestone bed and thickness and nature of shales were mapped. Moisture sensitivity of shales (slaking and swelling indices), thickness and nature (chemical composition) of underclays, presence of channels, etc., were also considered from ground control point-of-view. Based on geologic description and physical property data on roof, coal, and floor rocks, the authors have developed rating scales for delineating areas with ground control problems overlying Herrin seam.

## Introduction

Coal exploration and development drilling for underground coal mines in the Illinois basin has been considerably advanced in the past one decade. Considerable attention is now paid to planning of drill programs, drilling procedures, geologic description and geotechnical investigation of rock cores, and preparation of geologic maps to illustrate anticipated and/or existing conditions. This has been the result of more difficult mining conditions (depth, geological disturbances, methane content, old workings), increasing capital requirements for a mining venture, and higher operating costs with a general decline in underground mine productivity. The mine operators have found it advantageous to learn more about the geological environment of the reserves to minimize the risks of development. Premining investigations can provide useful information in mine design, equipment selection, estimation of mine productivity, and analysis of ground control problems. Some of the data obtained can be utilized to improve feasibility studies and financial analysis.

Subsidence control plans required under the Federal Surface Mining and Land Reclamation Act, 1977, would further increase emphasis on premining investigations. Data on aquifers and geotechnical properties of rocks overlying the coal seams will be required to preserve the quality and quantity of ground water resources and anticipate subsidence.

The authors have been actively engaged in premining investigations in the basin coal properties for the past five years with primary emphasis on planning ground control.

This paper describes premining investigations, geologic and geotechnical evaluation of rock cores and geologic mapping to evaluate ground control problems. The rating scales for delineating areas with ground control problems were developed and are discussed.

## Core Drilling Programs

Premining investigations in the past frequently consisted of limited number of core holes (1-inch or 2-inch diameter) in large reserves. The primary objectives of these were to indicate the presence or absence of coal seam/s, determine coal seam thickness to estimate reserves, recover coal cores for washability tests and mine product quality estimates, and geologic evaluation of cores for roof control. The holes were rarely drilled below the coal seam and little emphasis was placed on the geotechnical properties of the coal seam itself.

Subsequent development of the reserves indicated that lateral geologic variations in strata conditions were not delineated during exploration which led to costly mistakes including abandonment of mines. It is not unusual for the roof and floor strata properties to exhibit greater lateral variability than seam thickness, seam quality and washability characteristics. Thus, more core holes are required to evaluate roof and floor conditions to desired confidence levels.

The authors have utilized a geometric pattern of core holes, Figure 1, for

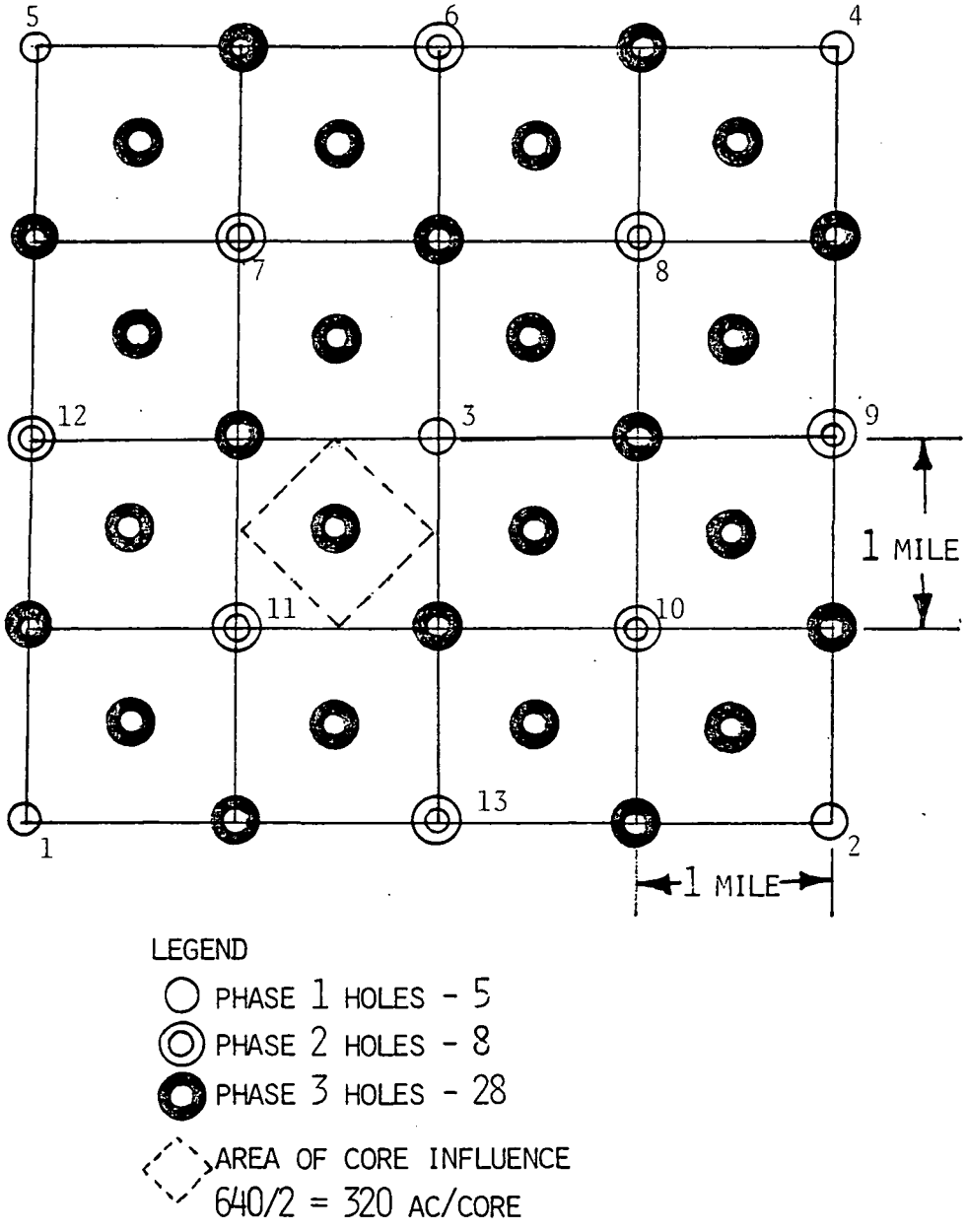



FIGURE 1. Core Drilling Pattern for Illinois Basin Coal Seams.

evaluating roof, coal, and floor variability over virgin reserve areas. The drilling program consists of three phases. In Phase I, five (5) NX-cores are drilled over a sixteen square mile area. Additional eight (8) NX-size holes are drilled in Phase II if Phase I data are encouraging. After evaluation of data in Phases I and II, either the program is terminated or Phase III is initiated to bring the area to development status. If the area has fairly consistent conditions, the 28 core holes in Phase III consist of NX-size cores and 5-5/8-inch cores for detailed geologic and geotechnical evaluation. If conditions are fairly variable, Phase III can be completed by NX-size coring with supplemental six-inch cores drilled in Phase IV to evaluate questionable areas. This is generally necessary in southeastern portion of Illinois basin where No. 5 seam is characterized by faults, seam splitting, seam thinning and highly variable roof strata due to contemporaneous and post depositional channels.

Small diameter cores (less than 3-inch) are generally not suited for geotechnical studies where roof and floor strata are friable and may deteriorate with circulating fluid. Several coal companies are not utilizing 5-5/8-inch diameter cores for rock mechanics tests. Such cores may be tested directly or smaller cores may be drilled in the laboratory. Recent studies (Chugh, 1978, 1979) using large diameter cores have provided good data in evaluating No. 6 seam in central and southern Illinois.

The authors have utilized 25-30 feet of core in the roof, core of coal seam, and 10-15 feet of core in the floor or to the first competent bed for rock mechanics tests. Holland (1962) suggested that all strata above and below a coal bed for a distance of twice the entry width should be studied for geotechnical properties. It should be emphasized that roof, coal, and floor strata cores should be analyzed for planning ground control.

There is a common myth in the basin that No. 6 and No. 5 coal seams have been adequately drilled and explored. This is a bad assumption and it is strongly recommended that each reserve area be adequately drilled for information on coal reserves, coal quality, geological disturbances, and ground control problems.

## Analysis and Description of Rock Cores

Colored photographs of NX-size cores in open boxes containing five 2-foot sections permit good records. The cores should be wiped clean of drill mud and photographed carefully so that fractures, natural bed separations, clay dikes and weak zones are easily identifiable. Permanent core descriptions, photographs, and geophysical logs are very valuable and may be useful in obtaining the OSM subsidence control plan permit.

A columnar record of the core at a scale of 1"=1 or 2 feet has been found very helpful for evaluating ground control problems. Pertinent descriptive data on the bed is presented on the left and significant geotechnical properties of each bed are presented on the right. Natural fractures, joint patterns, and changes in the lithology of the bed are also shown on this columnar record as shown in Figure 2.

## Evaluation of Roof Strata

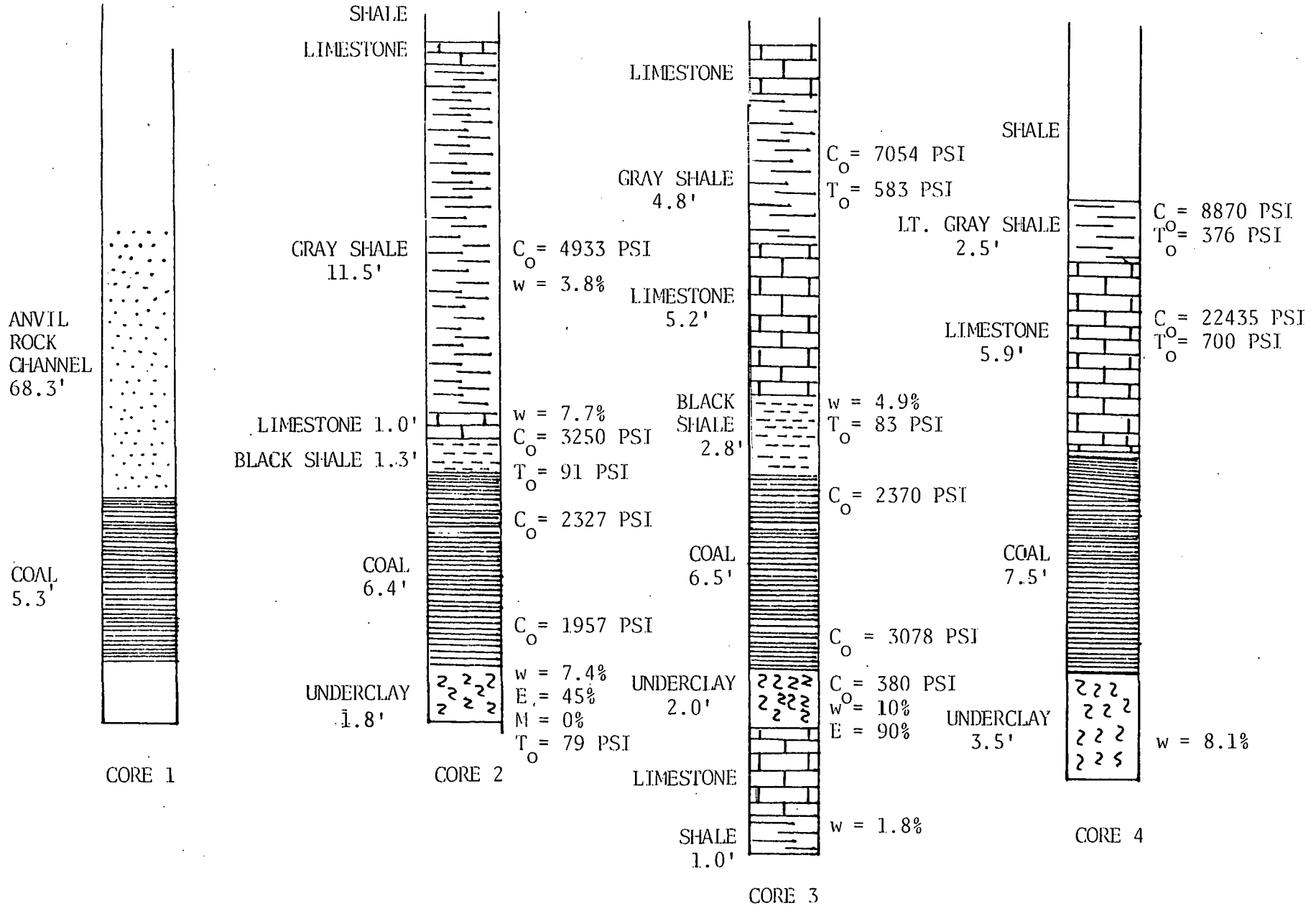
### Geologic Evaluation

-Assessment of the number and thickness of individual roof beds in the immediate roof is important. Thinly bedded strata of diverse character or interlaminated coals and shales are indicators of potential roof control difficulties.

-Pronounced bedding planes are indicative of inherent weakness in both No. 6 and No. 5 seam roof strata. Such bedding planes generally occur in roof strata affected by channel facies.

-Natural fractures, their inclination, and fracture density should be carefully studied in the immediate roof strata as well as in the potential anchor zone. Slickensided zones should be carefully mapped.

-Location and thickness of anomalous, thick, silty, or sandy shales and sandstone strata indicating the




Legend: $C_o$ - Unconfined Compressive Strength
w - Moisture Content
$T_o$ - Brazil Strength Parallel to Bed
E - Expandable Minerals in Clay
M - Montmorillonite in Clays

Figure 2. Typical No. 6 Seam Roof Strata Southern Illinois

presence of channels or channel facies should be carefully noted. Increased water problems and poor roof conditions are common in such areas.

-Qualitative tests of water sensitivity of shales and argillaceous laminations in limestones should be made in the field. Roof strata above No. 6 and No. 5 coal seams may be highly susceptible to moisture effects.

-Relative strength and hardness of roof strata should be evaluated in the field and marked on columnar sections. Judgements should be made of the quality of roof bolt anchor zones, bolt lengths, and recorded.

-Height of the first limestone bed above the coal seam is important in evaluating No. 6 seam roof conditions. Anchorage into limestone bed is customary to reduce roof control problems where the shale is fractured or extremely sensitive to water.

-Anchor horizon should be relatively free of argillaceous laminations and fractures. The bed in which the bolt is anchored should be of adequate thickness (at least 2 feet).

-The beds should be properly identified. Coloration and texture of shales, density and jointing should be utilized to differentiate between the typical Anna shale and atypical Energy shale related to the Pennsylvania channel facies.

-The existence of natural fractures, their inclination, presence of filling and its nature should be carefully noted. Clay dike faults are very common in the basin and can significantly influence roof control.

-The contact between the coal seam and immediate roof rock should be carefully noted. Significant inclination may be indicative of rolls, the presence of clod may indicate roof control and dilution problems.

-The thickness and nature of Lawson shale is important. Where the ratio of the thickness of Lawson shale to underlying Brereton limestone is large, roof control may present problems.

Rock Mechanics Studies

The following rock mechanics tests have been utilized on roof rock strata: 1) unconfined compressive strength with modulus of elasticity and Poisson's ratio, 2) indirect tensile strength along and across bedding planes, 3) modulus of rupture for immediate roof rock, 4) moisture content, 5) slaking and swelling indices of shales, 6) anchorage capacity tests.

The geotechnical data can be utilized for designing safe span of mine openings, size of coal pillars and planning ground control. The utilization of rock mechanics data in evaluating ground control problems in basin coal mines is illustrated by means of a currently ongoing research project (Chugh, 1979). The columnar sections showing immediate roof, No. 6 coal, and floor strata for four 6-inch diameter holes are shown in Figure 2.

Core 4 depicts Brereton limestone in the immediate roof above the seam. The limestone has high compressive (22,435 psi) and tensile (700 psi) strengths. It is of adequate thickness so that three-foot long conventional bolts can be installed and should provide adequate roof control. The gray limey shale above limestone is moderately strong with 8,870 psi compressive strength and 376 psi tensile strength. Very few roof control problems should be anticipated in the area and productivity should be relatively high.

Core 3 shows 2.8 feet of black Anna shale immediately above the coal seam. The shale has two pronounced set of joints and has very low tensile strength (83 psi). It has, however, low swelling index and high slake durability. The limestone bed is unfractured. It is proposed to install 5-foot long bolts anchored in limestone in this area. The grey shale above limestone has moderate strength with

7,054 psi compressive strength and 583 psi tensile strength. The key to roof control in this area would be black Anna shale. Roof bolts should be installed immediately after mining.

Core 2 indicates area with possible roof control problems. The immediate bed in the roof is 1.3 feet of Anna shale with an unconfined compressive strength of 3,250 psi and relatively high moisture content of 7.7%. The Brereton limestone is poorly developed and is only one foot in thickness. Lawson shale is thick and relatively weak. The roof bolts will have to be anchored in this shale. Narrow entries with 6-7-foot long bolts in conjunction with header boards or resin bolts would be recommended in this area. The increased moisture content and decreased strength in core 2 are probably due to the presence of a nearby channel.

Core 1 indicates the presence of the Anvil rock sandstone channel lying immediately above the coal seam. The coal seam has been partially cut-off by the channel. The channel area is expected to contain water and at the interface between the channel and the marine facies, roof control will be extremely difficult. Therefore, the area west of core 2 has been excluded from the current reserve assessment.

East of Core 4, local lenticular pods of Energy shale lying below Anna shale were observed. Although Energy shale is not laminated and has reasonably good strength, the interface between the two shales is abrupt. The Energy shale tends to separate from Anna shale. Maps have been developed showing the thickness of shale lenses. The development of this shale has its origin to a channel which had both contemporaneous and post coal deposition phases.

These evaluations are utilized to develop roof evaluation maps from ground control point-of-view. These have been found very helpful in planning mine development and estimating productivity in those areas. The roof evaluation maps consist of: 1) glacial drift isopach, 2) total shale isopach (top of coal to base of limestone in No. 6 seam), 3) Energy shale isopach (No. 6 seam), 4) facies change maps, 5) bolt length isogram, 6) cross-sections adjacent to channels, 7) roof rating isogram, and 8) height of isogram of limestone bed (if any) above the coal seam.

Based on geologic and geotechnical studies discussed earlier, a roof core rating scheme has been developed for roof rocks above No. 6 coal seam and is shown in Table 1. A rating factor ranging from Zero for bad roof conditions to Four (4) for excellent conditions is utilized for each parameter. Each parameter is evaluated individually to introduce objectivity. Overall roof core rating is obtained by dividing total points by the number of parameters. A weighting factor can be assigned to each parameter and multiplied by rating points to arrive at weighted points for each category. Points for all categories are then totalled and divided by 100% to get overall numerical weighted roof rating from 0-4.

Evaluation of critical parameters individually eliminates subjectivity in core evaluation and provides more consistent evaluation from core to core in a given reserve or for comparing one reserve area against another.

## Evaluation of the Coal Seam

Coal seam is shown on the columnar section on a scale of 1"=2' along with immediate roof and floor strata. In evaluating coal seam, particular emphasis should be placed on the following:

- -abnormal splits in the seam which may pose mining problems and be indicative of channel facies in the roof
- -extreme variations in seam thickness which may be indicative of the presence of channels
- -presence of clay seams which can act as weak planes along which movement in coal pillars can occur

-severe fracturing of the coal in conjunction with disturbance in roof strata may indicate proximity of faulting or slips in the coal seam
-sudden change in the structural elevation of the seam may relate to extreme undulations and/or local fault disturbances
-angular coal bed-roof stratum contact may be indicative of the presence of rolls

Rock mechanics studies include unconfined compressive strength with elastic moduli at two or three different horizons in the coal seam. Two or three-inch cubes or cylindrical samples may be utilized for testing. The authors have observed significant strength variation (5:1) in different horizons in No. 5 coal seam. This can lead to rib failures at different horizons as is experienced in several mines in the basin. The strength data are utilized for computing size of coal pillars.

With regard to evaluation of the coal seam, several maps are proposed including coal seam section with splits, gross seam isopachs, mineable section isopachs, structural elevation isopach at the base of coal seam, cross-section in area of seam splits. It is important to recognize that weak immediate roof and rider coal seam may have to be mined with the main seam in some cases to obtain good roof control. This considerably increases dilution of the ROM coal.

## Evaluation of Floor Strata

Until in the recent years, floor strata were generally not cored and ignored in evaluation of ground control problems. It should be emphasized that floor strata are as important in evaluating ground control problems as roof strata and coal seam and, therefore, should form an integral part of the evaluation.

The NX and smaller size cores are generally unsuited for evaluation and rock mechanics studies because they are generally broken and disturbed by drilling fluid. This is especially true of the underclays. The authors have utilized 5-5/8" diameter cores with considerable success. It has bee possible to study strength characteristics of underclays either on cores or prismatic specimens prepared from cores.

The following points should be particularly noted in evaluating floor strata:

-thickness of immediate soft floor clays
-thickness of strata to the first competent stratum which can carry load
-nature of sub-strata below the competent load-carrying member; presence of weak underclays below the first competent layer can be detrimental
-clay mineralogy of underclays and weak sub-strata

Particular attention should be focused on high concentration of montmorillonite and mixed layer clays below nodular limestones.

Rock mechanics studies include unconfined and confined compressive strength with elastic moduli for immediate floor strata, moisture content of underclays and weak sub-strata below competent load-carrying bed, Atterberg limits, and clay mineralogy.

The following maps are most commonly utilized in evaluating floor control problems: 1) underclay isopach, 2) isopach of interval from base of coal to competent bedrock, 3) floor rating isogram, 4) cross-sections showing vertical variability in strength, moisture, and clay mineralogy.

Based on geologic and geotechnical studies of large number of reserve areas in the basin, a rating table for floor cores is proposed in Table 2. The extreme variability of the No. 6 seam floor both vertically and laterally and the presence of occasionally weak strata below the typical nodular limestone bed makes the correlations with the rating table difficult.

The poor floor and/or poor coal seam conditions may not only reflect as floor heave or rib sloughing problems but may also lead to roof falls. The interaction among the roof, coal, and floor structural elements is very important from ground control point-of-view and should be considered in

planning ground control.

## Concluding Remarks

Premining investigations involving adequate drilling, geologic evaluation of cores, and geotechnical studies can be effectively utilized to predict possible ground control problems during mining. The mining layouts, pillar sizes, and roof control plans can be varied based on these studies to provide good ground control, reduce production losses, increase coal recovery, and reduce mining costs. The Federal Surface Mining and Land Reclamation Act of 1977 will require more emphasis on premining investigations to minimize and predict subsidence and reduce adverse effects on hydrology in underground mineable reserves.

It is recognized that limited number of boreholes during premining investigations cannot provide complete information on what is buried under the earth over large areas. However, the geologic and geotechnical data obtained from these holes can provide meaningful information on the anticipated ground control problems. The costs of premining investigations are generally very small in comparison to the development costs and, therefore, should not be deterrant to undertaking them.

It should be emphasized that geotechnical data (strength, swelling properties, modulus of rupture, etc.) alone are not adequate in evaluating ground control problems. It should be considered along with the geologic description of rock cores and stratigraphy in order to obtain a true picture.

Table 1. Rating Table for Roof Cores - Illinois No. 6 Seam

| Evaluation Factor | Bad - 0 | Poor - 1 | Fair - 2 | Good - 3 | Excellent - 4 |
|---|---|---|---|---|---|
| Interval to base of limestone | >15 ft. | 8-12 ft. | 4-8 ft. | 2-4 ft. | 0-2 ft. |
| Number of beds in interval to limestone | numerous laminated | multiple | two | one | one |
| Relative strength of beds | all weak - no strong beds <3,000 psi | alternating strong and weak 3,000-5,000 psi | moderately strong or strong lower bed with weaker intermediate beds 5,000-8,000 psi | moderately strong 8,000-10,000 psi | strong >10,000 psi |
| Fractures or bedding in interval to limestone | highly fractured or severe bedding separation | moderate fractures or bed separation | few fractures moderately strong on bedding | occasional fractures difficult to separate on bedding | no fractures no bedding separation |
| Roof shale sensitivity to water (Percent swell strain perpendicular to bedding) | highly sensitive >4.0 | sensitive 1.5-4.0 | moderately sensitive 0.5-1.5 | slightly sensitive 0.2-0.5 | insensitive 0.0-0.2 |
| Unbroken limestone anchor thickness | no lime within boltable height | 0-18" | 18-24" | 24-36" | >36" |
| Limestone character | none within boltable height | numerous fractures or weak shale lamination | occasional fractures or shale laminations | no fractures, occasional small shale laminae | no fractures or laminations |
| Projected bolt length | no anchor | 10-14 ft. | 6-10 ft. | 4-6 ft. | 3-4 ft. min. |

Table 2. Rating Table for Floor Cores - Illinois No. 6 Seam

| Evaluation Factor | Bad - 0 | Poor - 1 | Fair - 2 | Good - 3 | Excellent - 4 |
|---|---|---|---|---|---|
| Floor clay thickness | >5 ft. | 3-5 ft. | 2-3 ft. | 1-2 ft. | 0-1 ft. |
| Floor clay sensitivity to water | highly sensitive | sensitive | moderately sensitive | slightly sensitive | insensitive |
| Floor clay hardness or strength | very soft >100 psi | soft 100-300 psi | firm 300-700 psi | medium hard 700-1500 psi | hard >1500 psi |
| Interval base coal to first competent strata | >10 ft. | >6 ft. | 4-6 ft. | 2-4 ft. | <2 ft. |
| Composition of first competent strata | none, reworked clays, slickensided, few lime nodules | thin interbedded strata, fractured, <25% lime nodules in claystone or mudstone matrix, slickensided | limestone nodules 25-50% by volume at least 5' thick in claystone matrix, moderate fracturing and slickensides | shales and limestone, nodules 50-75% by volume at least 3' thick in claystone matrix, few fractures slickensides | shales, limestone, nodular lime 75% by volume in claystone matrix |
| Substrata below first competent bed | all weak members | strong beds with weak beds below | beds increase in strength with depth | moderately strong beds below nodular lime for at least 5 feet | strong beds immediately below nodular lime |
| Moisture content substrata below first competent member | >10% | 8-10% | 6-8% | 5-6% | ≤5% |
| Clay mineralogy of weaker strata within 5 ft. below first competent bed | >70% montmorillonite | 0-25% montmorillonite 75-100% mixed layer | no montmorillonite <75% mixed layer | no montmorillonite <50% mixed layer | no montmorillonite <40% mixed layer |

# SESSION II

## GROUND CONTROL IN THE ROOM-AND-PILLAR MINING SYSTEM

Session Co-Chairpersons:

Dr. N. Aughenbaugh, Department of Mining, Geological and Petroleum Engineering
University of Missouri at Rolla

Mr. R. Caudle, Carbondale Mining Technology Center
U.S. Department of Energy

# DESIGN OF ROOM AND PILLAR MINING SYSTEMS

by

James R. Aggson*

Abstract. Recent investigations into ground control problems in underground coal mines have shown a strong correlation between the results of theoretical structural analyses of coal mine openings and observed failures in operating mines. This paper presents a general overview of the various loading conditions that can be expected underground and the response of the mine roof to those loading conditions. Potential ground control problems and recommended solutions are discussed for several roof types under various loading conditions.

## Introduction

The stability of any underground opening is a complex function of numerous geotechnical parameters. The in situ stress field is one of the basic parameters that has a direct effect on underground structural stability. A mine is like a bridge or any other structure in that the integrity of the final product depends on the forces to which the system is subjected. This system of underground forces, or the in situ stress field, is, of course, partially due to the gravity load of the overburden. In addition to the gravity load, an underground site may be acted upon by horizontal stresses that are in excess of those produced by gravity loading. When a new underground mine is developed, the underground stress distribution should be determined and taken into consideration in the design of underground openings. This paper discusses the stress distributions that have been determined by the Bureau of Mines at several locations in the United States and the effect that various types of loading conditions can have on the stability of a mine roof.

## The Stress Field Underground

The vertical stress underground is dependent on the depth of overburden and the density of the overlying strata. The gravitational component of the horizontal stress distribution is a function of Poisson's ratio of the material and the vertical stress. The following equations can be used to calculate the stresses due to the gravity load of the overburden:

$$\sigma_v = p\ g\ h \qquad (a)$$

$$\sigma_h = \sigma_v \left(\frac{\nu}{1-\nu}\right) \qquad (b)$$

where $\sigma_v$ = vertical normal stress,
$\sigma_h$ = horizontal normal stress due to gravity load,
$p$ = density of overburden,
$g$ = gravitational acceleration,
$h$ = depth of overburden,
and $\nu$ = Poisson's ratio.

The derivation of equation (b) is based on the assumption that the underground site has lateral confinement. If the underground site is not confined laterally (i.e., above the drainage level of the immediate area), equation (b) may not apply and the horizontal stress may be less than predicted by equation (b). Equation (a), however, applies in most underground situations and the vertical stress calculated by equation (a) is usually the only component of the vertical stress.

Although equation (b) does represent the normal horizontal stress due to overburden at an underground site, it cannot be assumed that this is the total horizontal stress field. The total horizontal normal stress in a particular direction at an underground site can be given by the equation:

$$\sigma_H = \sigma_h + \sigma_e \qquad (c)$$

where $\sigma_H$ = in situ horizontal stress,
and $\sigma_e$ = excess horizontal stress.

The excess horizontal stress is defined as the portion of the in situ horizontal stress field that is in excess of that gravitational component

$$(\sigma_e = \sigma_H - \sigma_h).$$

Excess horizontal stress distributions have been determined to exist in a number of different mining environments throughout the United States. A summary of stress measurement results that have been determined by the Bureau of Mines can be found in the literature (1). Experience has shown that the magnitude and orientation of the horizontal secondary principal stresses at any particular location are closely related to the regional and the local geologic structure. The reasons for the existence of excess horizontal stress in the earth's crust are not well understood or agreed on in the scientific community. This understanding, however, is not required so long as it is recognized that the earth is not a static, gravity-loaded sphere and that, in order to be stable, an underground opening must support the actual, in situ stress distribution, not just the load of the overburden.

An estimate of the vertical stress at any particular location underground is roughly 1 psi for every foot of overburden. The horizontal stress due to gravity load is on the order of one-third of the vertical stress. The excess horizontal stress can have any magnitude from zero on up to several times larger than the vertical stress.

As an example of a coal mine located in an area that contains excess horizontal stress, we can look at an underground coal mine in the Beckley seam in southern West Virginia. The stress distribution was determined using the borehole deformation gage and overcoring techniques developed by the Bureau of Mines. The stress information was used as part of the input to the analysis of the severe ground control problems that the mine was experiencing. The compressive stresses in the horizontal plane were determined to be 3,300 psi bearing N69°E and 1,700 psi bearing N21°W. The mine was laid out such that the orientations of the main entries and the cross cuts were N25°W and N65°E respectively. Thus, the N25°W main entries were virtually perpendicular to the maximum horizontal compressive stress of 3,300 psi. This combination of mine layout and stress orientation produced a variety of roof and floor

* icist, Mine Structure Design, Denver Mining Re ch Center, Bureau of Mines, U.S. Department of the Interior, Denver, Colorado

problems in the main entries. These ground control problems can be seen in Figure 1, which shows a portion of the developed area of the mine.

The roof and floor problems that were occurring in this mine are a direct result of the fact that the horizontal stresses were by far the largest stresses acting on the mine openings, and that these stresses were not anticipated or allowed for in the mine design. The manner in which horizontal stresses affect stability will be discussed in detail in another section.

## Geologic Roof Structure

The response of the mine roof to the in situ loading conditions depends on the particular type of roof involved. This discussion will be limited to two general roof types. These are the massive roof and the thin-bedded, or laminated, roof.

Stratified rocks are considered massive if they occur in thick beds and are free of joints. For this discussion, thick-bedded rocks are considered massive if the spacing between joints, partings, faults, etc. is comparable to, or larger than, the critical dimensions of the opening (approximately one-half the opening width), or if the strength of the bond across partings or joints is comparable to the rock strength. A thin-bedded or laminated roof differs from a massive roof in that the bonds between the sedimentary beds are weak, and these weak bonds form parallel planes of weakness. Also for this discussion, the classification of bedding in the roof as thick or thin depends on the width of the opening. A 2-foot-thick bed in the roof may be considered thick in an opening 10 feet wide, but thin in an opening that was 60 feet wide.

## The Roof Failure Mechanism

The following discussion is a general summary of the results obtained from detailed theoretical (1) and numerical (2) evaluations of specific ground control problems in underground mines.

If a coal mine entry with a massive roof is subjected to horizontal stresses that are significantly larger than the vertical stress, and these compressive stresses are of sufficient magnitude to produce shear stresses in the roof that exceed the shear strength of the material, the most probable ground control problem is an arch-shaped shear failure in the roof. The shear failure starts near the corner formed by the roof and the pillar. The height of the arch, or the angle of propagation of the failure surface, depends on the relative magnitudes of the horizontal and the vertical stress. If the horizontal stress is much larger than the vertical stress, the failure surface will be at a low angle (relative to the horizontal). Figure 2 represents the type of failure produced by high horizontal stress in a massive roof. High horizontal stress loading conditions produce compression in the entire roof member. Even the center of the roof span (point A) is in compression.

If the horizontal and vertical stresses that produce the failure depicted in Figure 2 become more nearly equal, then the failure surface will rotate toward the vertical and the arch will gain height. If the horizontal and vertical stresses are equal and of sufficient magnitude to produce shear stresses that are higher than the shear strength of the roof rock, then the most probable failure will be a vertical shear failure near the rib. This type of failure may be the so-called "cutter roof." The failures produced in a massive roof by equal horizontal and vertical stresses are shown in Figure 3. Again under these loading conditions the entire roof member, including point A, is in compression.

If the horizontal stress is considerably less than the vertical stress, as would be the case in a gravity-loaded situation, the entire roof member is not in compression. Maximum tension is in the center of the roof span. Since the tensile strength of most rocks is quite low, the most probable ground control problem in a gravity-loaded situation would be a vertical tension crack in the center of the span, as shown in Figure 4. This assumes that the tensile stresses are larger than the tensile strength of the material.

If an entry is created in a mine that has a thin-bedded or laminated roof with weak bedding plane bonds, the roof failure mechanism is quite different than that in a mine with a massive roof. The first thing that happens to a mine roof when an entry is created is a release of stored vertical strain in the immediate roof. This release of stored vertical strain is aided by gravity sag of the immediate roof and produces a downward deflection of the roof member. Maximum deflection will occur at the center of the roof span, as depicted by location A in Figure 5. The deflection of the immediate roof member creates shear stresses that cause the weak bedding plane bond between the first and second roof members to fail. The failure of this bond will prevent the still competent upper roof from aiding in the support of the lower member. Thus the immediate roof member can deform independently of the upper portion of the roof.

The horizontal stress in the immediate roof has stored horizontal strain in the immediate roof member. This immediate roof member can now be looked at as a curved beam with a bending moment (produced by the vertical deflection). The stored horizontal strain in the immediate roof member will be relieved by continued bending into the entry. This situation is similar to a long, coiled spring in axial compression. Without lateral restrictions, the spring will bend and elongate to its uncompressed length with the application of the slightest bending moment. An analysis of the roof member as a curved loaded column shows that tensile stresses develop at points B, B', and A. If the horizontal stress is high enough or if the tensile stress of the roof material is low enough, failure will occur at points B, B', and A.

As the immediate roof member fails, the second roof bed loses its lateral confinement and is free to fail into the entry in the same manner. This failure process of the laminated roof members continues, as shown in Figure 6, until a thicker member is reached or until the failing roof establishes a stable geometry.

The laminated roof failure process that has been described here has been identified as the floor heave failure mechanism in a coal mine with a thinly bedded, laminated floor (1).

## Design Recommendations

The mine operator has very little control over the in situ stress distributions and the strengths of the various rock types encountered underground. The operator can, however, vary such things as bolting patterns, entry geometries, entry orientation, and pillar geometries in a manner that will minimize ground control problems. The operator should not randomly initiate changes in the mine design to try to improve ground control. Rather, the operator should have a good understanding of the various conditions causi the problem(s) and a systematic plan that addresse the particular failure mechanism(s) involved. The most cost-effective mine design step that can

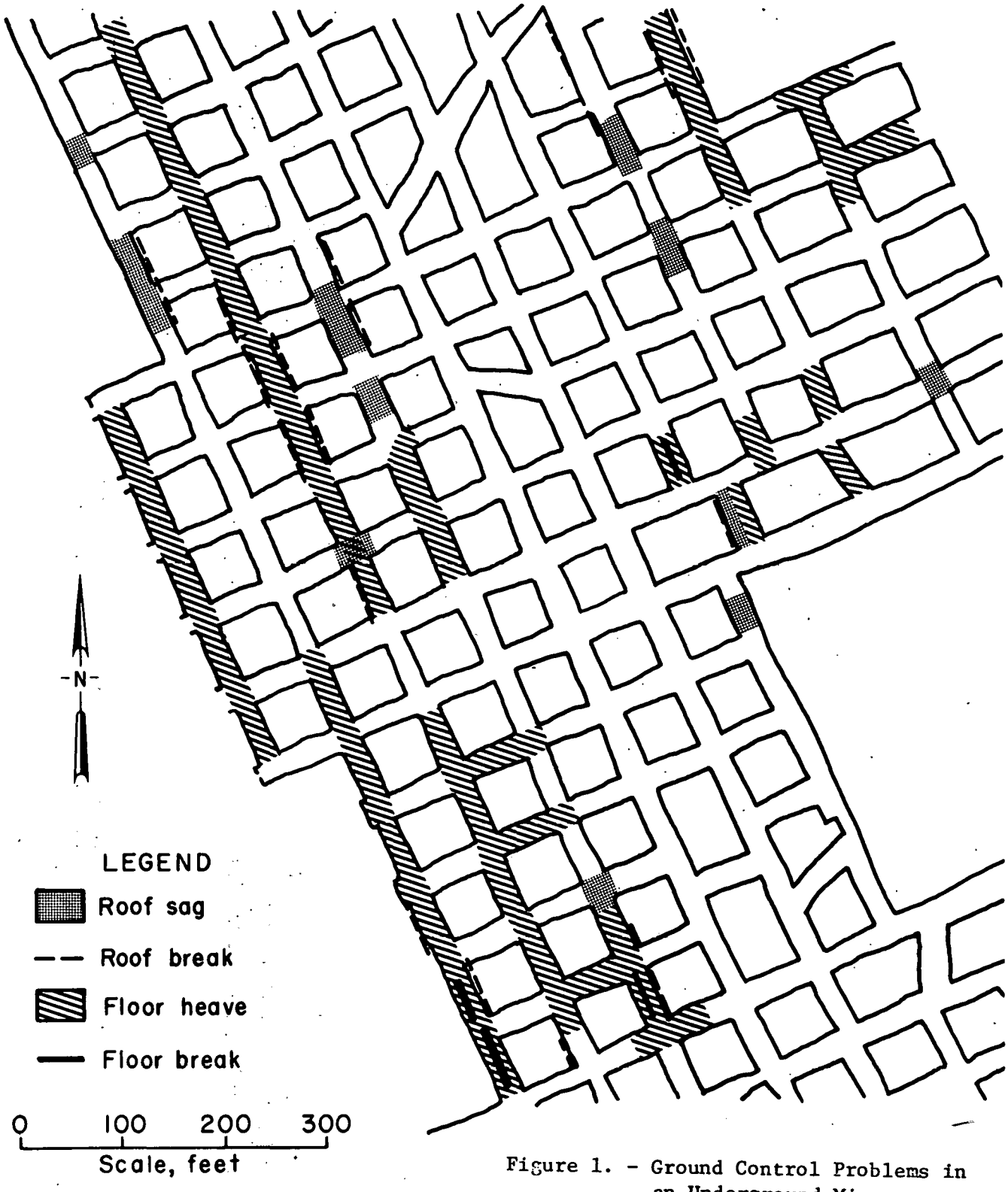



Figure 1. - Ground Control Problems in an Underground Mine.

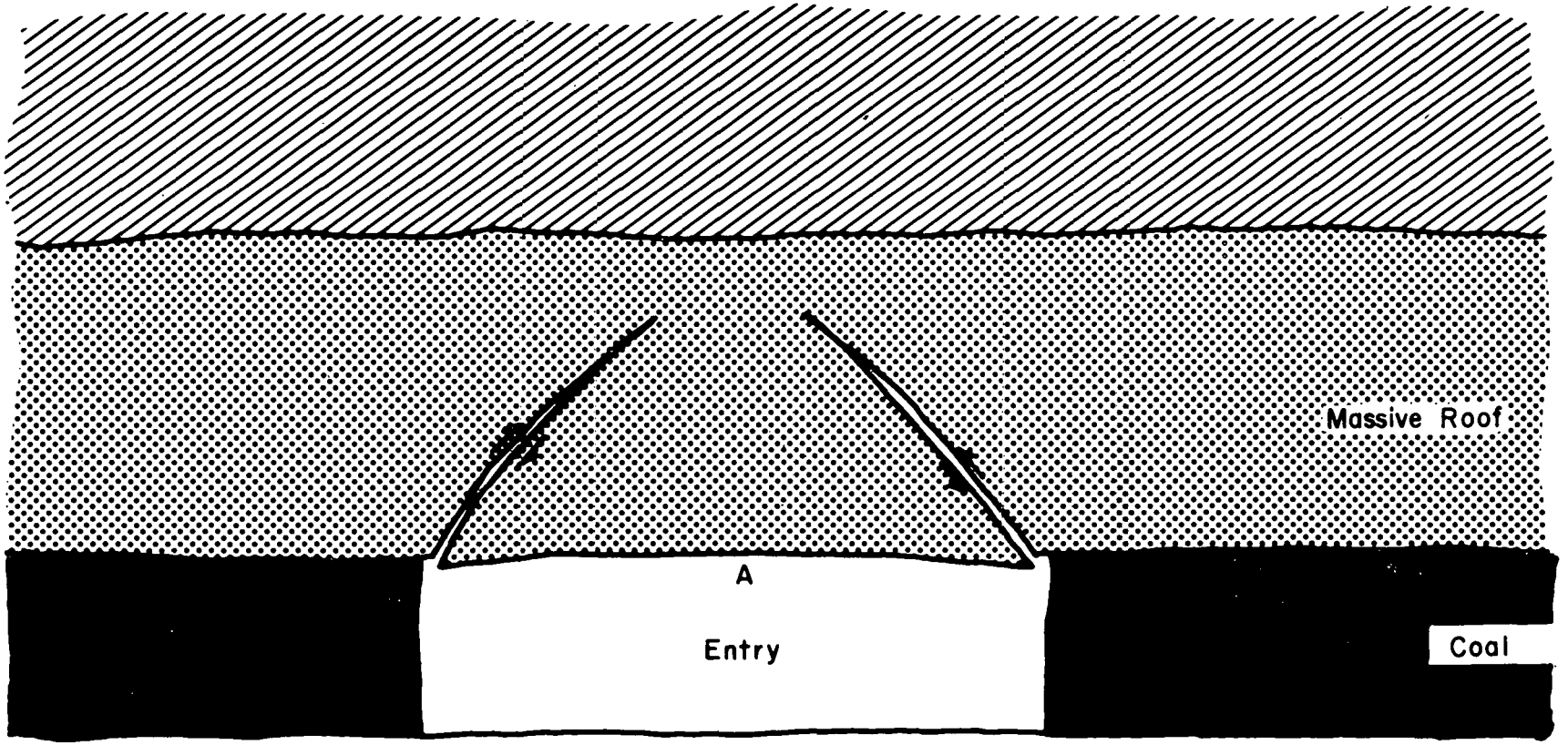



Figure 2. - Shear Failure in Massive Roof Due to High Horizontal Stress.

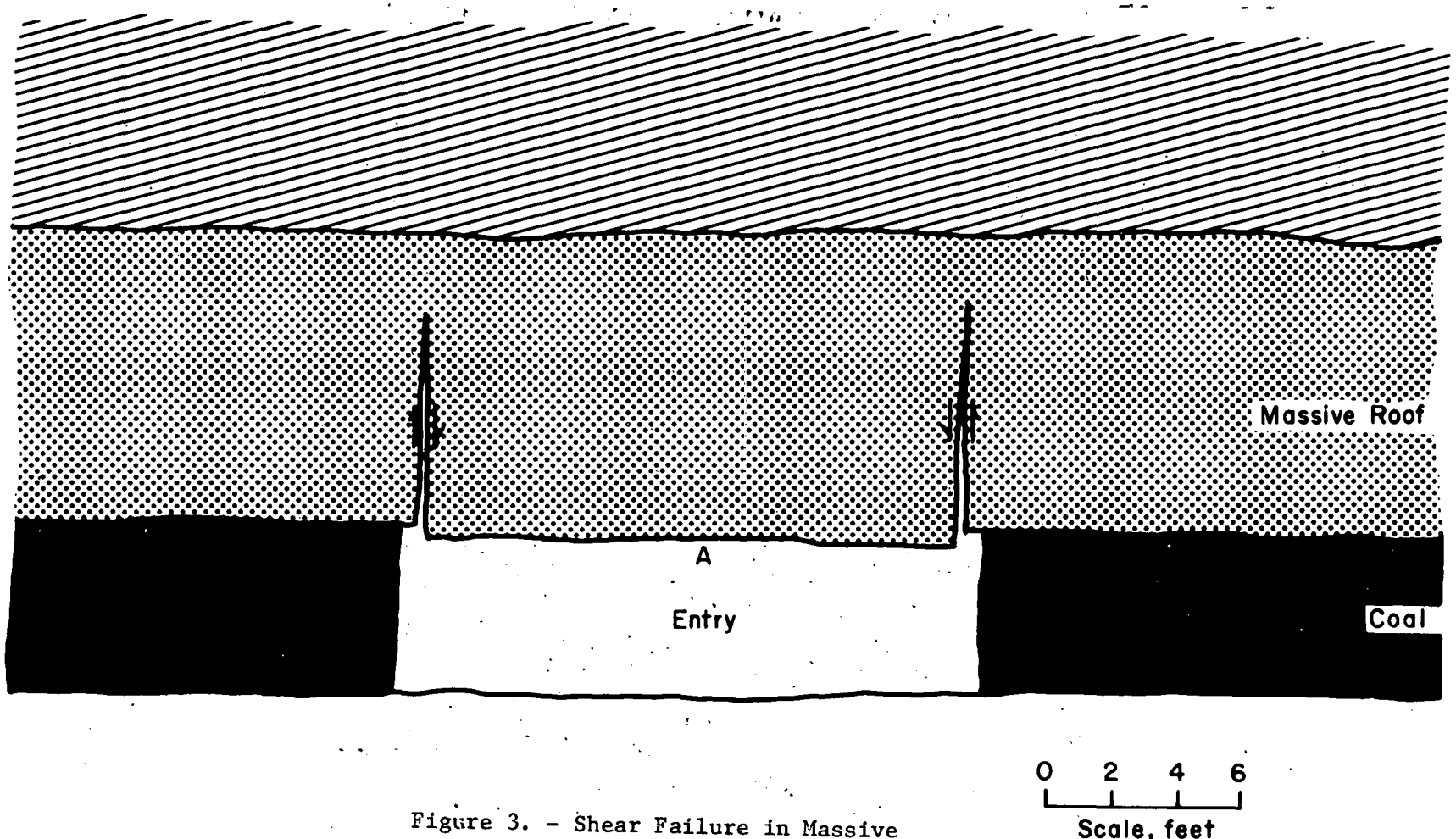



Figure 3. - Shear Failure in Massive Roof Due to Equal Horizontal and Vertical Stressess.

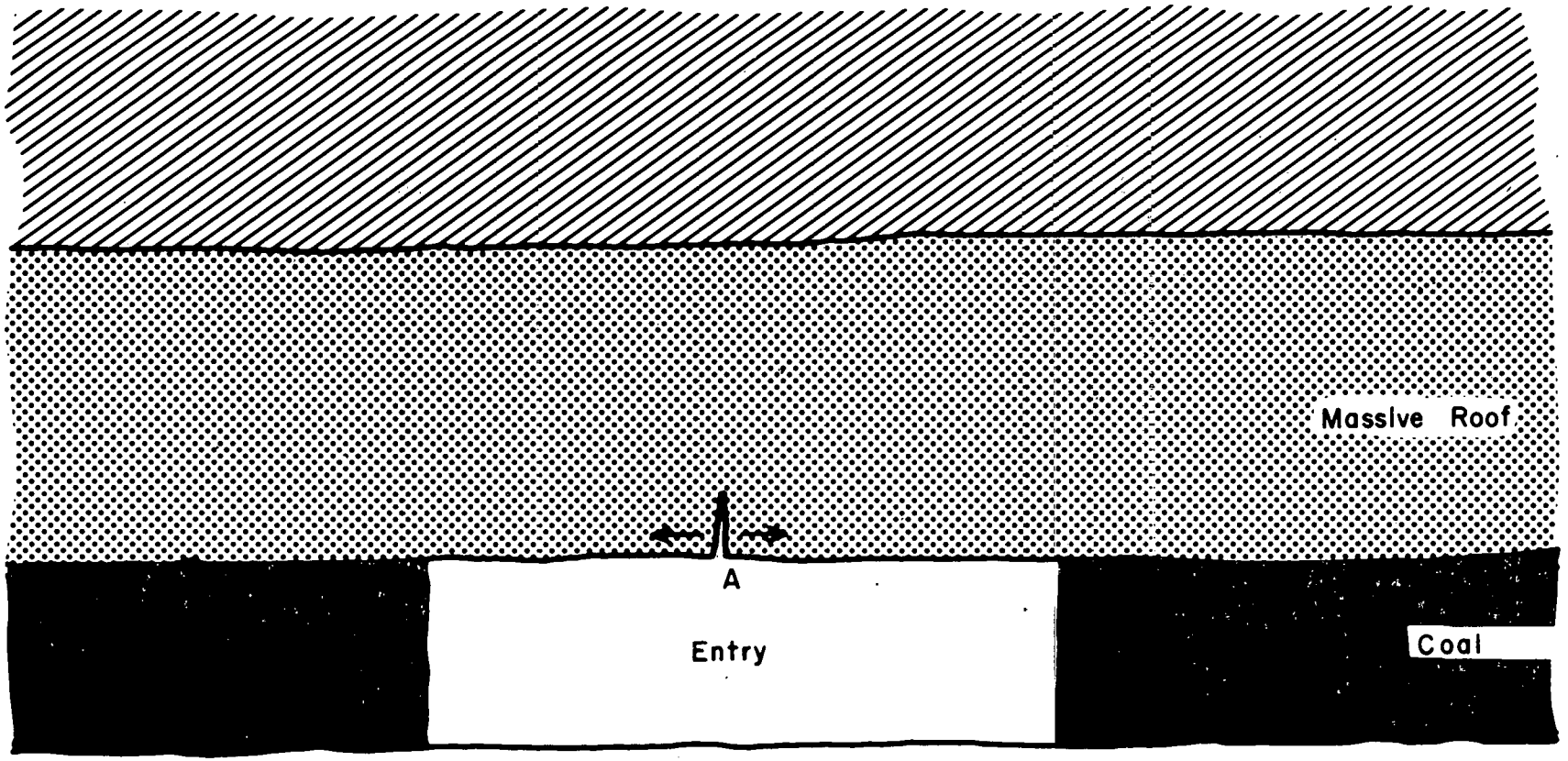



Figure 4. - Tensile Failure in Massive Roof due to High Vertical Stress.

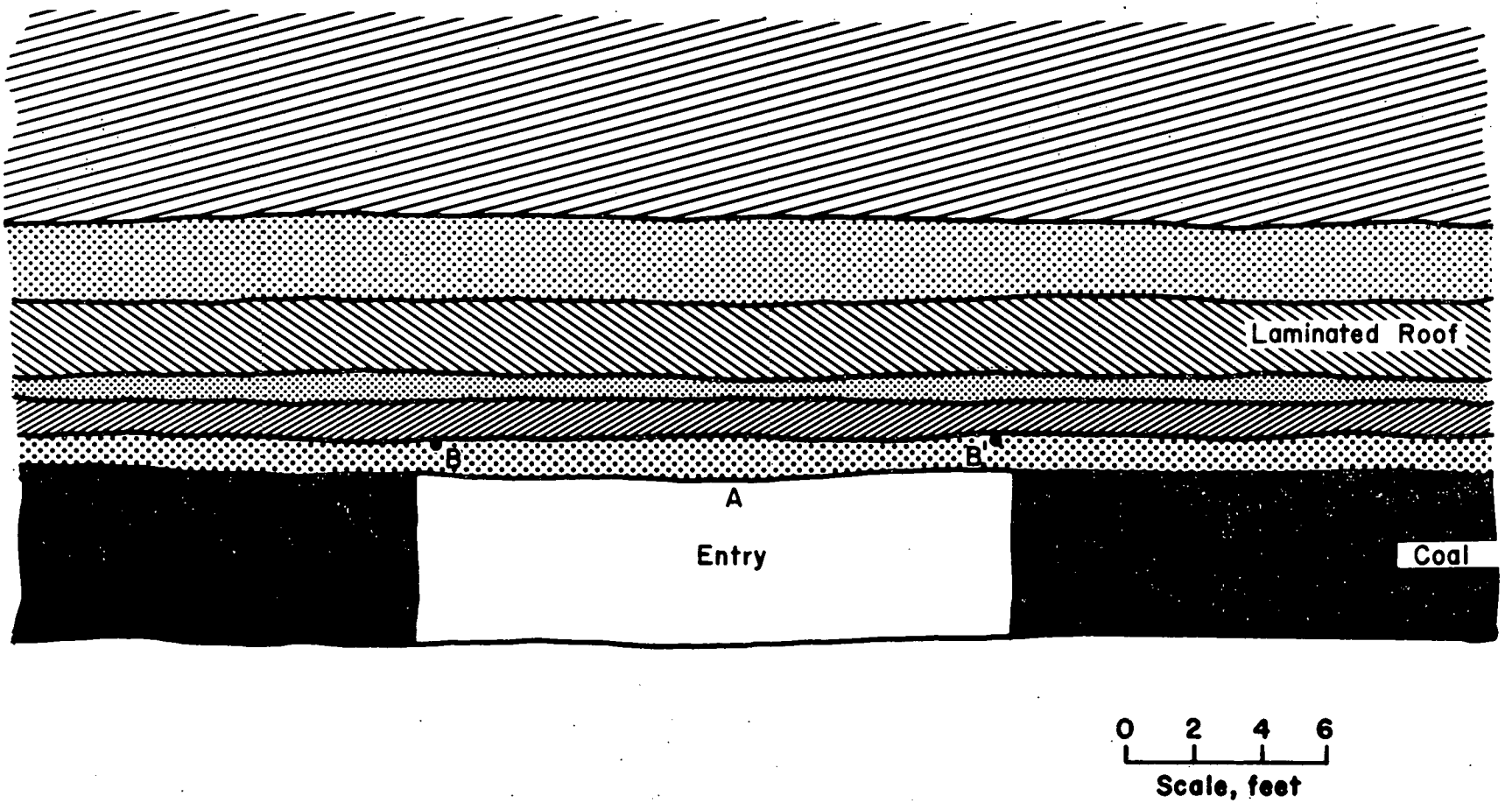



Figure 5. - Mine Entry with Laminated Roof.

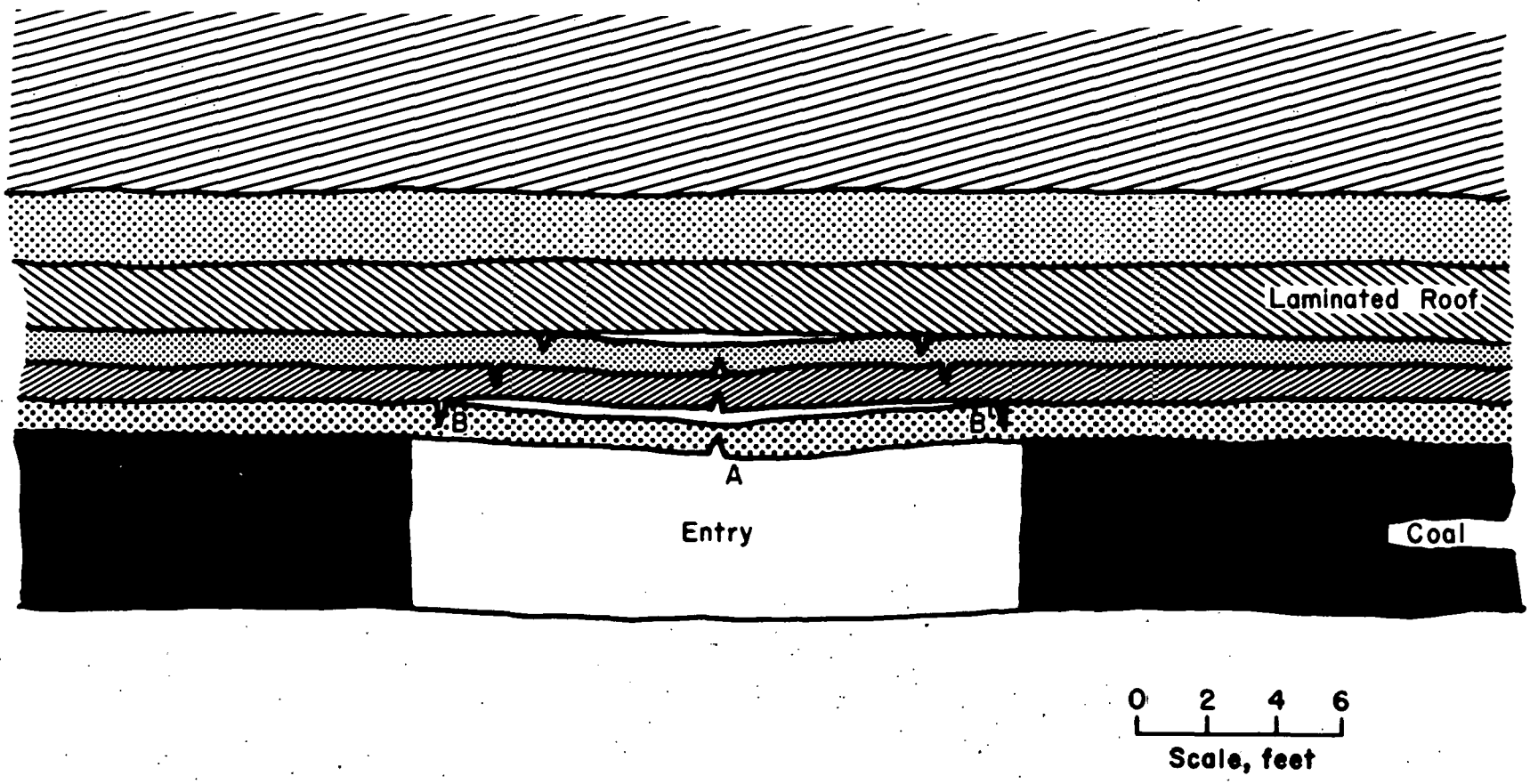



Figure 6. - Laminated Roof Failure.

be made to deal with in situ stress distributions is to select entry orientations so as to minimize problems in haulage, ventilation, and other entries that will be required for long periods. This requires a k edge of the stress field early in the mine's d opment. For once development has begun, changes in the mine's layout are costly and can be temporarily counterproductive. Entry orientations should be selected so as to reduce the probability of roof failure and minimize the requirements for and the cost of artificial support. If the most probable roof failure mechanism(s) can be identified for a particular mine design with a specified stress distribution and known physical properties, then the operator can evaluate the particular type and the cost of the artificial support that will be required to make the mine productive and safe.

As an example of the type of design recommendations that might result from a specific set of mining conditions, we may consider a mine with a laminated roof in an area that has a high biaxial horizontal stress distribution. The main entries should be laid out parallel to the maximum component of the compressive horizontal stress field. This will minimize the horizontal stress in the roof perpendicular to the axes of the main entries. To avoid the failure of the laminated roof in the cross cuts, the cross cut width should be small. Reducing the width of the cross cut effectively reduces the slenderness ratio (the length to width ratio) of the laminated roof members and therefore increases their stability. The optimum pillar geometry depends on the stress distribution, the strength of the coal, and the amount of overburden that must be supported. It is possible that a mine may be faced with conditions so severe that a combination of entry orientation, room size, pillar geometry, etc. cannot be specified that will produce a safe, productive, and economic operation. An operator faced with such a condition may have to consider a change from the more conventional mining techniques to a system that stress relieves large areas of the mine. Horizontal stresses tend to strengthen pillars by providing additional confining pressure in the pillar core.

A comprehensive set of design recommendations that covers all possible underground conditions is impossible to create. Each mine is unique, and each property should be evaluated in terms of its own loading conditions, physical properties, and individual characteristics. The resulting mine design should minimize the probability of failure, the required artificial support, and the danger to the miner while maximizing productivity and efficiency.

## Summary

The stress distribution that exists underground cannot, in general, be assumed to be solely the result of the gravity load of the overburden. Investigations in underground mines have shown that horizontal stresses in excess of those produced by gravity loading can exist, and that such stress distributions can directly affect the structural stability of underground openings.

The predicted roof failures discussed in this paper for the various loading conditions do not include the effects of geological discontinuities or weaknesses that can exist in mine roof rocks. The various roof members are considered to be continuous l eneous members, and are analyzed as such. This ach, however, appears to be reasonable in that the observed roof failures in existing underground mines with known stress distributions are exactly the failures that are predicted by a detailed analysis. If geological discontinuities do exist in a particular mine roof, these discontinuities will most likely be involved in the failure mechanism.

The most logical approach to avoiding ground control problems is that of careful evaluation of conditions and proper planning early in a mine's development. If ground control problems are to be understood and eliminated in an existing mine, a knowledge of the in situ loading conditions is essential. Changes in the basic mine design that are intended to improve roof stability or ground control should only be attempted after the loading conditions, physical properties of the materials involved, and failure mechanisms are well understood.

## References

1. Aggson, James R., Coal Mine Floor Heave in the Beckley Coalbed, An Analysis. BuMines RI 8274, 1978, 32 pp.

2. Aggson, James R., Stress-Induced Failures in Mine Roof. BuMines RI 8338, 1978, 18 pp.

# OBSERVATIONS IN MINES WHICH ARE INDICATIVE OF GROUND CONTROL PROBLEMS IN THE ILLINOIS BASIN

David L. Stritzel

Mining Engineer and Supervisory Coal Mine Technical Specialist
Mine Safety and Health Administration
Benton, Illinois

## INTRODUCTION

In order that we may understand the theoretical concepts of roof control more clearly, we would like to present actual case studies of situations depicting serious ground control problems in the Illinois Basin. As you will see, many concepts of Rock Mechanics can be applied in our daily routine of practical observations. Many times, we find ourselves wrapped up in complex situations which in reality, have relatively simple solutions. If we understand the basics of what causes ground control problems, we can use our experience in conjunction with theory to prevent such occurrences, thereby providing safer and more efficiently operated coal mines.

## CASE STUDY NO. 1

The strata overlying the coal bed in this particular mine consists of approximately 80 to 90 feet of shale and 110 feet of unconsolidated glacial material. The underclay is very soft and ranges in thickness from 5 to 6 feet and more. The coal itself is moderately soft, showing very little rib rash, and is approximately 7 feet thick.

The area being mined in this case was located in a "glaciar-age delta" which contained several channels which had cut into the rock strata overlying the coal bed. The problem area of this mine laid directly below one of these channels. To better illustrate this condition, Figure No. 1 is provided.

The mining plan consisted of a 9-entry checkerboard style system with 75-foot centers (55-foot pillars) and 20-foot wide openings. The entries were numbered 1 thru 9, left to right.

The original plan for roof control called for 5-foot conventional-type bolts as a sole means of support.

The section was developing main entries when numerous roof falls began occurring in No. 1 and No. 8 entries. In was evident after investigating these falls, that a shearing action of the bedding planes was occurring. Therefore, to try and stabilize this shearing motion, the mine operator installed resin bolts, which appeared to be the logical solution. However, the roof continued to fall in these two entries as the section advanced. In addition to these falls, a very pronounced heaving of the bottoms was beginning to occur in the No. 6 and No. 7 entries.

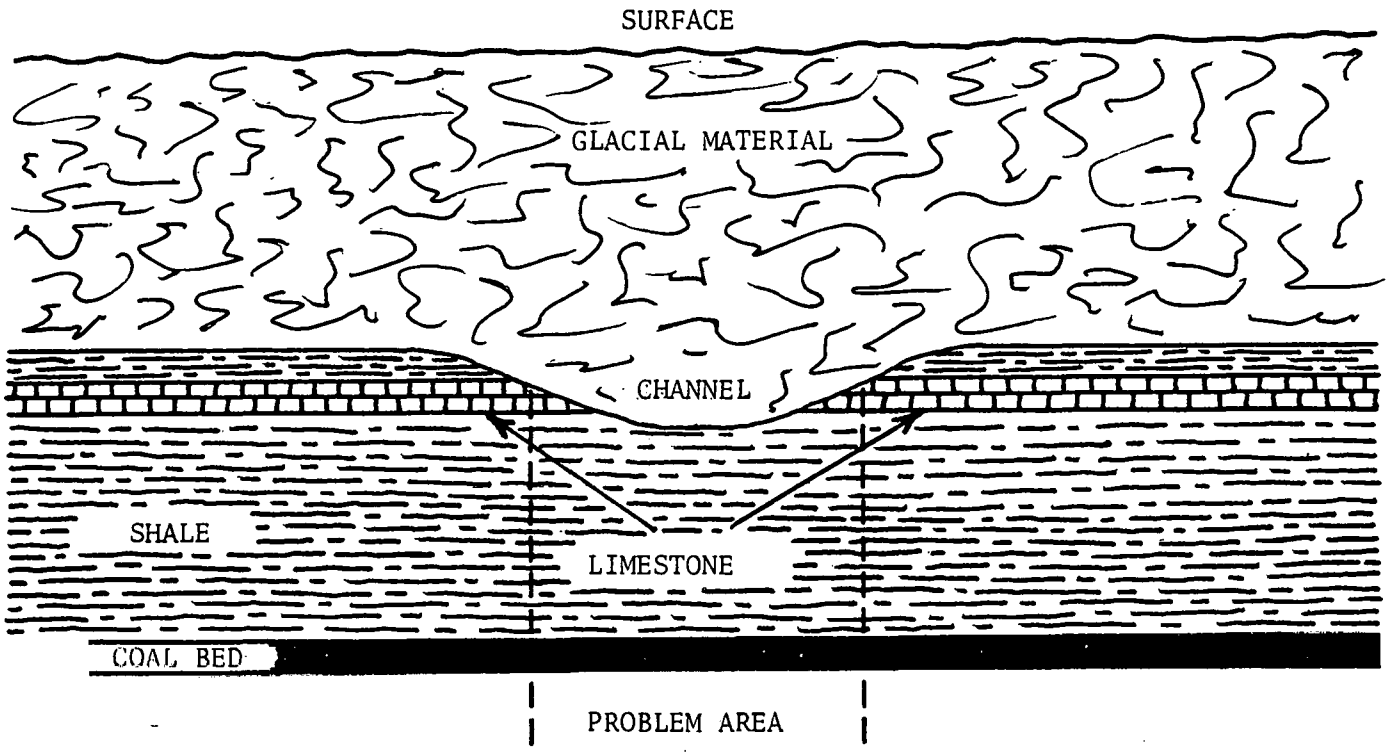



FIGURE NO. 1

The mine operator then chose to set cribs and timbers in these entries to try and support the roof, but to no avail. However, the solution to this problem was not to provide additional roof support, but rather to redesign the mining plan to allow for larger pillars, thus, supporting the bottoms. It was now evident that unstable roof was not the problem, but rather, the entire section was being squeezed. In this particular case however, it was too late for adjustments, the section was lost.

What caused the squeeze was an under-design of the pillar size. The glacial material acted as "dead weight" on the pillars since there is no strength to unconsolidated material. The shale was bending under the excessive weight resulting in a fixed-beam effect and eventually fractured in the No. 1 and No. 8 entries. Once this occurred, water seepage became evident and was allowed to accumulate and penetrate the underclay, thereby weakening it further. Consequently, the small pillars simply pushed into the underclay until the openings were completely sealed tight.

The load distribution across the section was indeterminate, but assuming that it was uniform, the forces on the beams would be as depicted in Figure No. 2. As indicated, the cross section of each beam rotate about their own axis, thus causing slippage on the plane between the two beams, which explains the reason for the shearing action of the planes experienced in the strata. In addition, the excessive loads caused by the glacial material created a bending moment at each fixed end of the beams. As the pillars gradually sank into the underclay, the forces of these bending moments became more evident as experienced by eventual failure of the roof in the No. 1 and No. 8 entries.

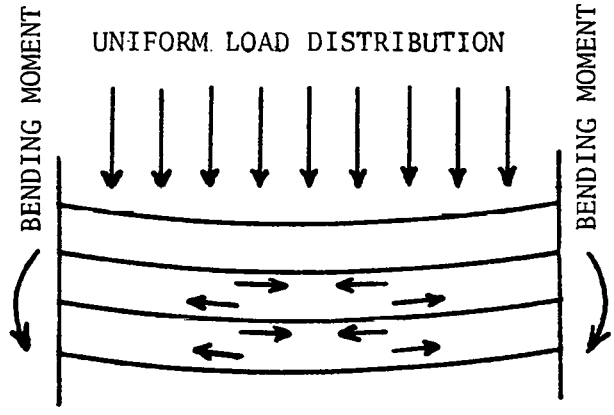



FIGURE NO. 2

CASE STUDY NO. 2

The coal bed in this particular mine is covered by approximately 85 to 90 feet of overburden which varied in thickness with the rolling surface terrain. The immediate roof consists of 30 inches of firm shale, above which is a soft clay or mud ranging in thickness from 5 to 18 feet. Above this mud is a bed of limestone which ranges in thickness from 6 inches to 2 feet, and varies from 8 to 20 feet above the top of the coal bed. Kettlebottoms are common in the immediate roof, and the coal bed tends to roll with the terrain, as indicated by Figure No. 3.

The mining plan consists of a 5 and 6 entry checkerboard style system, with 50 foot centers (30 X 40 foot pillars), and 16 to 20 foot openings.

Originally, 4 foot resin bolts and various lengths of conventional bolts were used to support the roof.

However, sporatic roof falls began occurring in areas where the strata was of an anticline nature as shown in Figure No. 3. The falls were steeply pitching in that they seemed to break vertically upward from the rib, and the soft clay or mud vein was thicker than usual, while the limestone was thinner than usual in these particular roof fall areas. Conventional roof bolts would not properly torque and therefore would not anchor securely in the soft clay or mud. Since the distance from the limestone to the coal varied greatly, it was not practical to try and anchor into the limestone. For example, the limestone in some areas was as much as 20 feet or more above the coal bed.

Four foot resin bolts failed to provide proper anchorage at all times because the resin would push into the soft strata, filling cracks and would therefore leave the space around the rod void of resin.

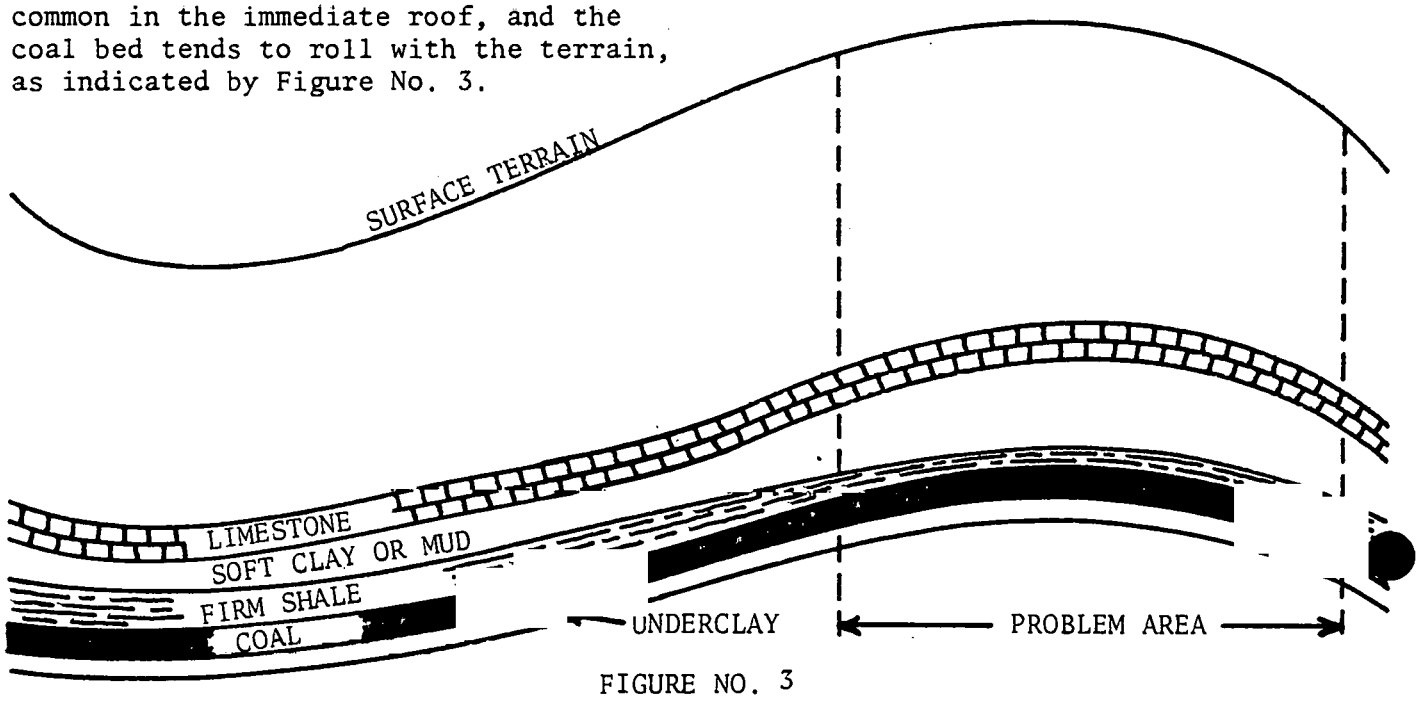



FIGURE NO. 3

Of course, the obvious problem in this case is how to provide proper anchorage in roof with such unconsolidated properties? In this particular case, the problem was solved by using shorter resin roof bolts, thereby anchoring into the immediate firm shale which created a beam effect, and subsequently supported the weak overlying strata.

CASE STUDY NO. 3

In the past several years, coal companies in the Illinois Basin have tried longwall mining in the Herrin No. 6 Seam, but to no avail. Mining would progress without significant problems until the longwall face had advanced approximately 165 feet. At this point, an extreme weight over-riding effect would take place causing fracturing of the coal and roof strata ahead of the working face. Once this occurred, the excessive weight would simply push the chocks down into the soft underclay.

The geology of the overburden is typical of the strata overlying the Herrin No. 6 Seam. The overburden is approximately 700 feet thick and a soft underclay lies beneath the coal seam. The immediate roof consists of shale, while a thick layer of limestone lays approximately 80 feet above the coal.

Various systems were tried to control the over-ride, such as lengthening the face, shortening the face, changing equipment, use of cribs for support, etcetera, but none of these controlled the problem. The fractures would still occur after 165 feet of advancement, and the chocks would sink into the soft underclay.

This problem was not an easy one to solve, mainly because it could not be determined what was causing the over-ride. What in fact, actually was happening can be best illustrated in Figure No. 4.

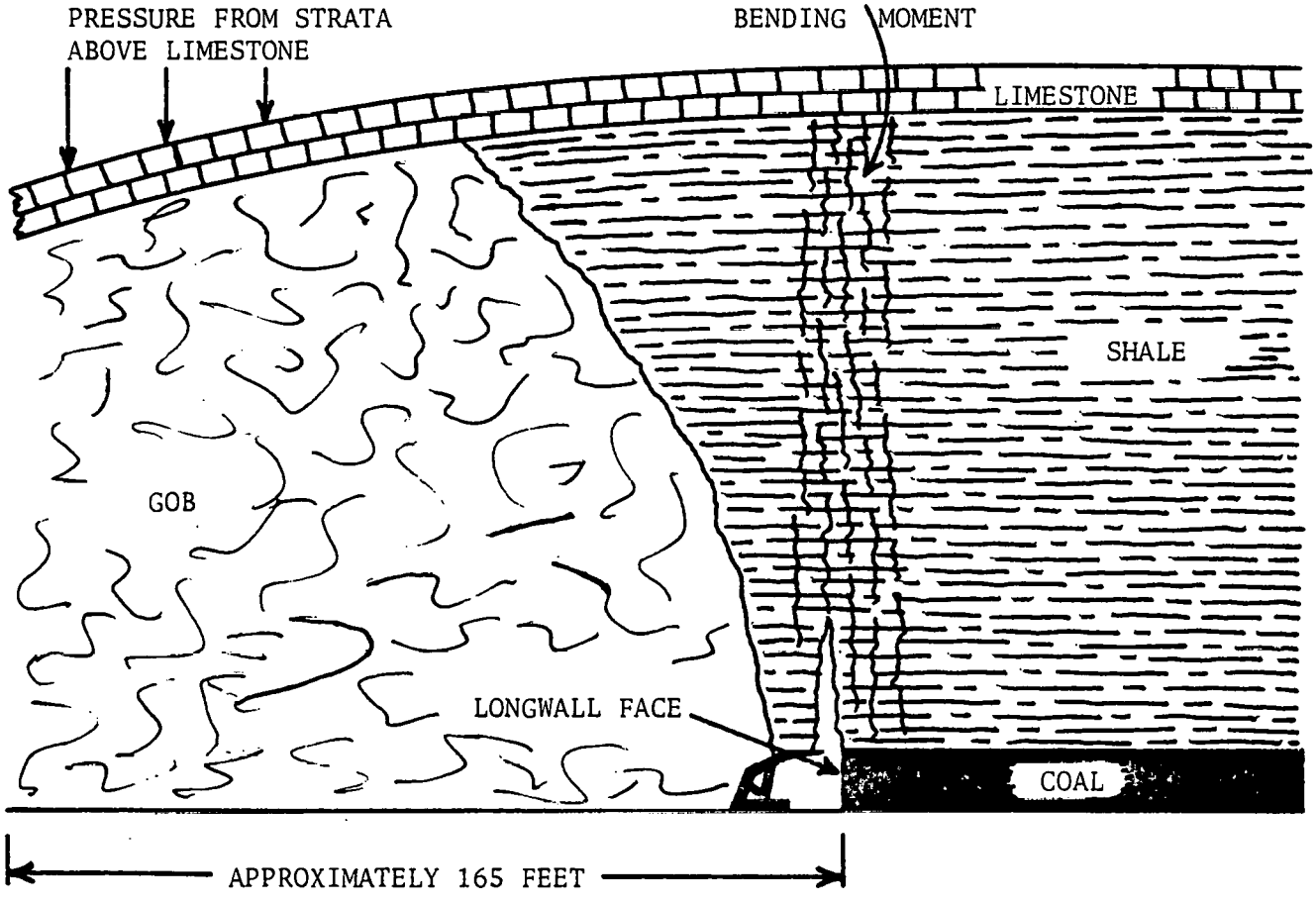



FIGURE NO. 4

As you can see, we were experiencing a cantilever beam effect created by the exceptionally strong band of limestone. The pressures on the limestone band created a bending moment at the fixed end. This in turn caused excessive pressures on the strata at that point, resulting in crushing and fracturing of the coal and rock strata.

As a result, rock began falling between the chocks and the coal face, thereby preventing further advancement of the longwall face.

This particular problem has apparently been solved by the use of very large shields in place of the smaller chocks. The fracturing still occurs, but the large shields are capable of advancing under the fractured strata until the limestone eventually breaks, thus relieving the pressure. It appears that, once this happens, normal mining can progress without incident.

Presently, however, the same problem is being experienced on existing longwalls in the Illinois Basin, but not at the coal face. Instead, the pillar of coal between the belt entry and the supply entry (which is adjacent to the solid block) is being extracted as depicted by Figure No. 5 and is not large enough to support the overlying massive shale and limestone. Consequently, a bending moment is created at this point, resulting in crushing and fracturing of the strata and coal pillar. Thus, the pillar is pushed into the underclay by the excessive pressures created by the bending moment and the mine roof is collapsing in both the belt and supply entries. Therefore, it appears that the immediate solution to this problem is to cease extraction of this pillar, move the conveyor belt to the entry adjacent to the solid block, and mine only the solid block. This would create a uniform load distribution across the section, thus, relieving the excessive pressures on these particular openings and pillar.

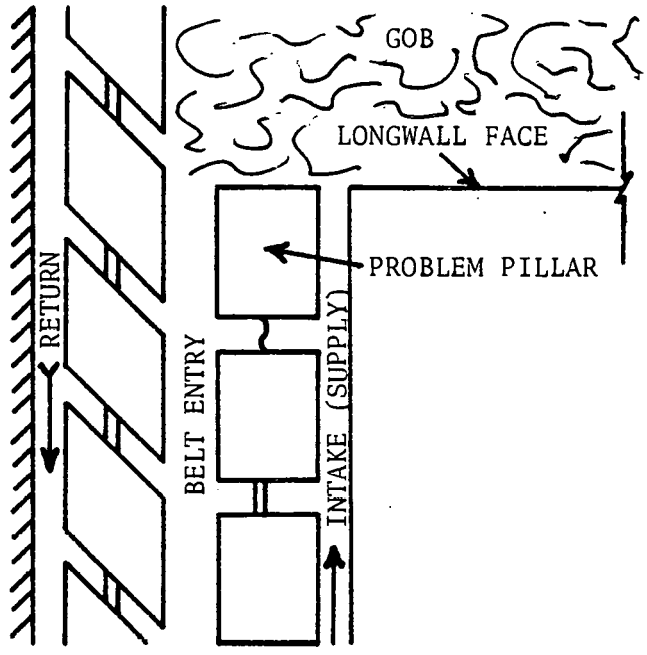



FIGURE NO. 5

## CONCLUSIONS

Every unintentional roof fall occurring in a mine should be investigated thoroughly to determine the cause so as to prevent reoccurrences. If several falls occur in any one given area, or a pattern of roof falls exist, it is wise to review the overall situation from a broad overview, and utilize all available expertise in Rock Mechanics to solve the problem as soon as possible. As Case Study No. 1 illustrates, by not following such procedures can result in costly consequences.

When investigating roof falls, a few basic determinations should be made before drawing any conclusions. Such determinations are as follows:

1. The shape of the fall, such as low angle or high angle arch, etc.

2. Location of fall (at an intersection, crosscut, face, etc.)

3. The dimensions of the fall, entries. crosscuts, and intersections (draw a sketch in detail and record on a mine map)

4. Location of roof failure (e.g. above anchorage zone, below anchorage zone, etc.)

5. Type of roof support used (resin bolts, conventional bolts, trusses, etc)

6. Type of supplementary roof support used and effectiveness (e.g. timbers, crossbars, rails, cribs, etc.)

7. Evaluation for defective material (e.g. bolt, expansion shell, bolt head, etc.)

8. Determination of mining practices and whether approved roof control plan was followed

9. Length of time roof was standing before failure was first evidenced

10. Types of warning observed prior to failure (e.g. sounds of fracturing in the roof, dribbling or flaking of the roof in the center of the entry, along the rib, etc.), and length of time the warning lasted.

11. Unusual geologic features such as slips, rolls, joints, faults, soft material in roof, thin-bedded strata, etc.

12. Was fall localized or was it a part of a series of falls consisting of a certain pattern within a particular area of the mine

13. Evidence of any high stresses in the area and their location

14. Evidence of localized heaving of bottoms in the area

15. Load distribution across the entire area including excessive pressures and their location. Plot these forces on a cross section of the strata covering the area and determine their effect on the situation.

In summary, investigations of unintentional roof falls should include as much detailed information as possible regarding the situation, a correlation of the data, and a thorough analysis of the results. Ignoring a roof control problem can be extremely costly in both lives and money. Therefore, a professional approach to roof control problems can result in improved mining conditions in the Illinois Basin, thereby benefiting all persons concerned. Basically, it's just good mining practice.

# EVALUATION OF THE EFFECTS OF WEAK UNDERCLAYS ON THE SUPPORT OF COAL PILLARS IN ILLINOIS BASIN MINES

John D. Rockaway
Richard W. Stephenson

Professor of Geological Engineering
Associate Professor of Civil Engineering

## SUMMARY

Failure of the mine floor supporting coal pillars frequently occurs when the subcoal strata include "underclays" or other low strength strata. The failure process has been studied to define the response of these materials to applied coal pillar loads and to determine the applicability of bearing capacity analysis to evaluate unsafe conditions.

The study has been accomplished through an investigation of both the intact and insitu geotechnical properties of the subcoal strata and through the installation of floor movement monitoring instrumentation. The results of these studies have indicated that both the shear strength of the underclay and the stratigraphy of the subcoal strata must be considered in the analysis. The shear strength was the major controlling factor determining the ultimate load carrying ability of the materials, but the thickness and stratigraphic position of the weak layers influenced the mechanism of the failure process.

The method of bearing capacity analysis developed by Vesic was used to define pillar stability. This method includes provisions for taking into consideration a two-layer system such as assumed for mine conditions where a weak underclay overlies firmer strata. The results of this analysis compared very well with observed mine conditions, indicating marginal conditions in areas of the mine which later experienced pillar support failure.

## DEFINITION OF PROBLEM

In coal mine stability studies, the subcoal strata, coal pillar and roof strata are a structural unit with varying relative strengths; therefore, the failure of one component can initiate the failure of the entire system. Such a condition may exist when, as a result of increased depths of mining or increased extraction ratios, the stresses carried by the pillars and transmitted to the mine floor exceed the stresses that the mine floor can support. When this occurs, the mine floor fails and the subcoal material is displaced outward and upward into the mined-out opening (Figure 1). This is accompanied by a downward increment of the pillars, a redistribution of stress in the roof, differential deflection and possible

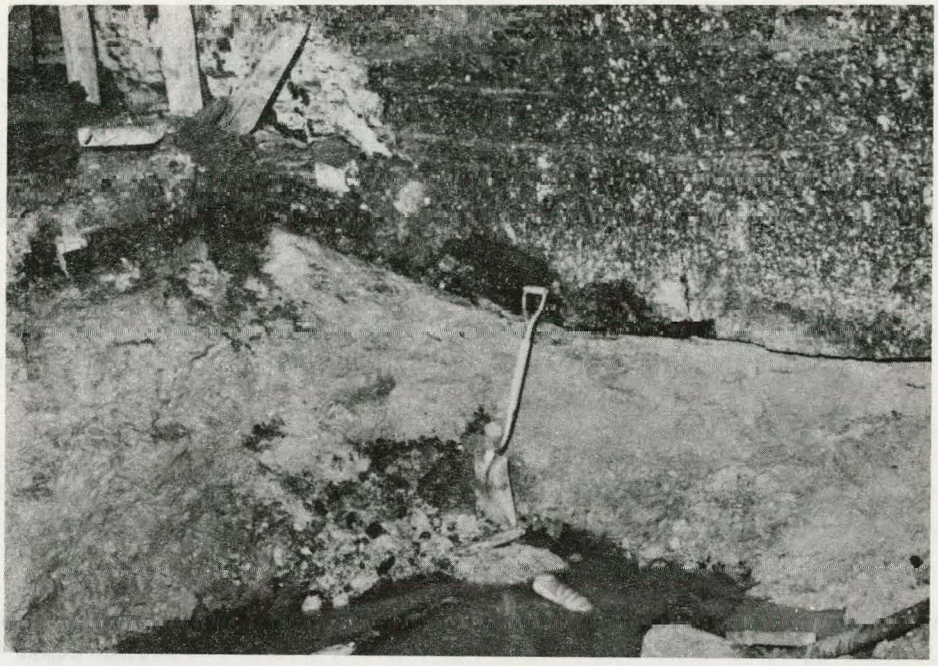


Figure 1 DISPLACEMENT OF UNDERCLAY FROM BENEATH A COAL PILLAR

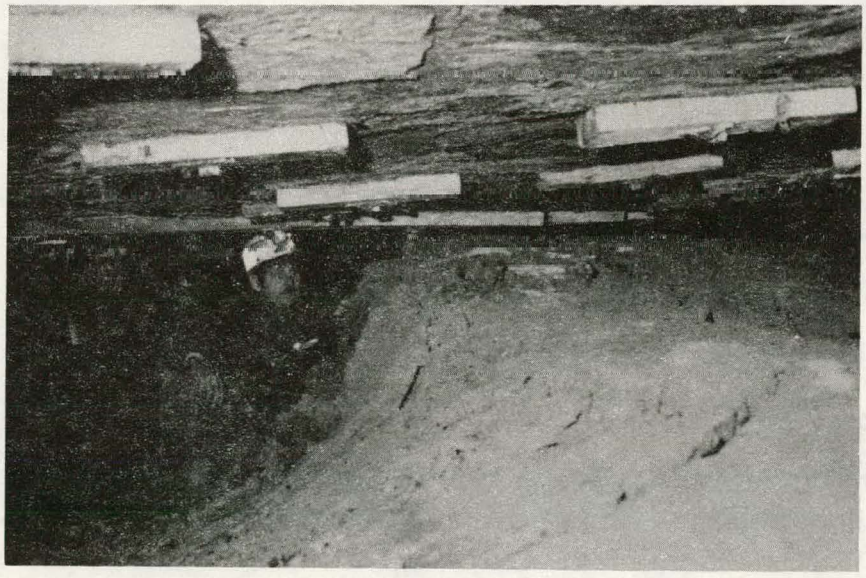


Figure 2 MOVEMENT OF UNDERCLAY INTO MINED-OUT OPENING ORIGINALLY TWO METERS HIGH

failure of the other components. If severe, floor failure in one area can spread through-out the mine since, when one pillar fails the overburden load must be redistributed to surrounding pillars which in turn over-load their supporting floor materials.

The effects of floor failure are well documented within the Illinois Coal Basin. Equipment has been trapped or destroyed by the closure of the floor, panels have been incompletely mined, and large blocks of coal have been left unrecovered after severe floor heave has closed entryways. In addition, roof falls often have been associated with the differential movement of the pillars and it has been suggested that surface subsidence might have ultimately developed as a result of the failure process.

This paper presents the results of a study carried out by the University of Missouri-Rolla with the support of the Interior Department's Bureau of Mines to conduct a geotechnical analysis of coal pillar support. The study was conducted from data collected within the United States mid-continent coal field, chiefly from that of Pennsylvanian deposits of the Illinois Basin. The results undoubtedly reflect the influence of the regional stratigraphic and tectonic history, since the conditions contributing to the failure mechanics in the Illinois Basin are distinct from those in other major coal fields. The residual stresses contributed from past tectonic activity do not compare with those of in the coal fields of the Appalachian or Rocky Mountains and the depth of mining is relatively shallow, generally from 200 to 600 feet. Since extraction ratios are comparable with those of other coal fields, excessive vertical or horizontal loading cannot be considered as the principal course of failure, thus the strength of the subcoal strata supporting the pillars must be responsible.

## DESCRIPTION OF SUBCOAL STRATA

Subcoal strata range from materials which are quite hard and strong to those which are very soft and deform readily. Some of the strata exhibit a high degree of plasticity, other are highly elastic. In addition, a lack of homogeneity in both vertical and lateral directions is a feature common to almost all coal mine floor strata. This variation in material properties, including some very weak members, plus the degradational changes in physical properties that accompany the mining process, are major factors that contribute to the potential instability of these bearing members that must support the coal pillars.

It is the member immediately beneath the coal that is important in coal mine pillar support studies. This strata is referred to by many terms, "seat earth" or "seat rock", "ganister", "fire clay", and "underclay" among them. However, the term "underclay" is commonly used in the mid-continent coal field in the United States. Underclay typically is described as being a seat rock associated with a coal bed, composed mainly of clay minerals, generally nonbedded and irregularly fractured.

The lack of bedding of structure and the presence of a random pattern of fissures or fractures is a characteristic of most underclay deposits. These fractures, considered to have formed during consolidation and tectonic movements, are randomly oriented, discontinuous and seldom more than a meter in length. Additional recent fractures probably have occurred as a result of post-mining stress relief. The presence of these fractures promote displacements even under low stress, since these surfaces offer very little resistance to renewed movement. Thus, when the underclay is overloaded by the pillars, the material beneath and immediately adjacent to the pillar is displaced outward and upward along the path of least resistance toward the mined-out area (Figure 2).

The form of movement that occurs,

however, will depend not only upon the presence of the fractures and the loads applied by the pillar, but also upon the thickness of the weak layers and the stratification of weak and firm layers beneath the coal. There are apparently three distinct mechanisms of floor movement that can develop, depending upon various relationships among the subcoal strata. All mechanisms, however, can be considered as direct examples or modifications of several forms of failure that are described by classical bearing capacity theory.

One failure mode occurs when the underclay is reasonably strong but the pillar is reduced in size by over-mining to such an extent that the loads applied by the pillar exceed the strength of the floor material. This has been referred to as a general shear failure and the result will be the development of a failure surface and an extrusion effect from under a solid pillar and bulging of the mine floor.

A second form of failure occurs when the floor material is relatively soft or loses strength as a result of the presence of moisture. Under these circumstances the pillar can punch into the soft material without formation of a full failure surface. When such penetration of the pillars into the floor occurs, cracks are visible at the pillar-mine floor junction and ridges of displaced material build up along these junctions parallel to the rib.

A third failure mode occurs when a hard strata is located at a relatively shallow depth below the mine floor, and is in turn underlain by a softer strata. If the pillar loads, transmitted through the firm strata to the weaker strata below, are sufficient to cause failure of this material, then a failure surface is developed and the softer materials are displaced upward and outward from beneath the pillars. When this occurs, the stresses act vertically upward on the stronger layer of the floor until the floor buckles as a result of tension.

## GEOTECHNICAL INVESTIGATION

The geotechnical properties of the underclay and other subcoal strata were evaluated by both laboratory and insitu testing procedures. Intact samples for laboratory analysis were obtained using a portable, electrically operated core drill capable of drilling more than 30 feet below the mine floor. Laboratory tests included Atterberg limits, clay mineralogy, grain size distribution and both unconfined and triaxial strength. All tests were conducted according to established ASTM procedures.

The selection of the samples for the specific tests was based to a certain degree on lithologic variation, in that an attempt was made to test specimens representative of all lithologic units. However, first priority was given to allocating pieces of core at least three inches in length to triaxial or unconfined compressive strength testing. Some difficulty was encountered in this respect for the underclay samples however, since much of this core was fissured to such an extent that it was not suitable for laboratory testing. In addition, other initially sound samples would, in a relatively short of period of time, open along microfissures as a result of stress relief and deteriorate too badly for testing. As a result of these limiting conditions, the strength tests were conducted only on intact samples that were relatively sound and could be prepared for testing, thus the data is undoubtedly biased toward the strong side. After failure in strength testing, the broken samples were submitted to testing of other physical properties.

The insitu evaluation program consisted of two phases, one phase the monitoring of the floor heave process, the other a series of plate-bearing tests to provide data for comparison of field strengths with laboratory-

derived strengths.

The object of the monitoring program was to determine not only the rate and direction of movement of the floor and subcoal materials, but also the nature of the deformation mechanism with respect to the depth of movement and relative displacement between strata.

The monitoring program was carried out of two sites of anticipated floor heave in separate portions of two coal mines. The program consisted of a set of inclinometers and extensometers installed to detect horizontal and vertical floor movements. Both types of monitoring instruments were anchored at a depth of 27 feet below the bottom of the mine floor, below the zone of potential floor deformation. All movement data was considered relative to this "fixed point." Each instrumented station generally included two inclinometers, one approximately at the centerline of the entryway and the other approximately halfway between this inclinometer and a coal pillar (Figure 3). The extensometers were positioned equidistantly across the same area and the anchor depths were selected so that any relative movement or displacement between critical strata could be defined as well as the total overall displacement relative to the fixed anchor point (Figure 4). Exact location of all instrumentation however, was subject to suitable drilling conditions and stable roof.

The principal test for collection of insitu strength data was the plate bearing test. Plate sizes and depth of testing were determined from analysis of the core logs and the test program consisted of three tests carried out with either a 12-inch diameter or 18-inch diameter plate conducted at depths varying from 0 to 18 inches below the underclay surface. Additional testing with a 24-inch plate was attempted, but loads high enough to cause failure could not safely be applied under existing conditions. Both stress levels at failure and mode of failure of the subcoal strata materials were documented.

## RESULTS

Core analysis indicated that the underclay at the selected drilling sites varied in thickness from 30 to 80 inches and, significantly, was thickest in the areas experiencing the greatest amount of floor squeezing. Analysis of the borings also showed that, of the mines sampled, the characteristics of the underclay varied greatly both horizontally and vertically. This variation occurred at all scales and gradational changes in the physical properties were characteristic of all deposits. It was evident that significant changes in thickness, lithology and induration take place over short distances and that this non-homogeneity is even more pronounced on a larger scale, such as between mines.

The extent of this variation was evident in the results of the tests defining physical properties. For example, triaxial strength tests were run on 85 samples of underclay and other subcoal strata and a wide distribution of strength values was obtained. Many of the underclay samples had triaxial strength values less than 200 psi, and most of these were less than 100 psi. However, several of the underclay samples had considerably more strength and some of the samples actually had triaxial strength values over 1000 psi. All of these samples were from the lower portions of the underclay and contained limestone nodules of sufficient size to influence the strength testing. Considering the difficulties of collection and preparation of weak or highly fissured samples however, it can be expected that the lower values probably were more representative of most underclays than would be indicat by their frequency of occurrence in the test data. It can also be expected that the higher values, particularly

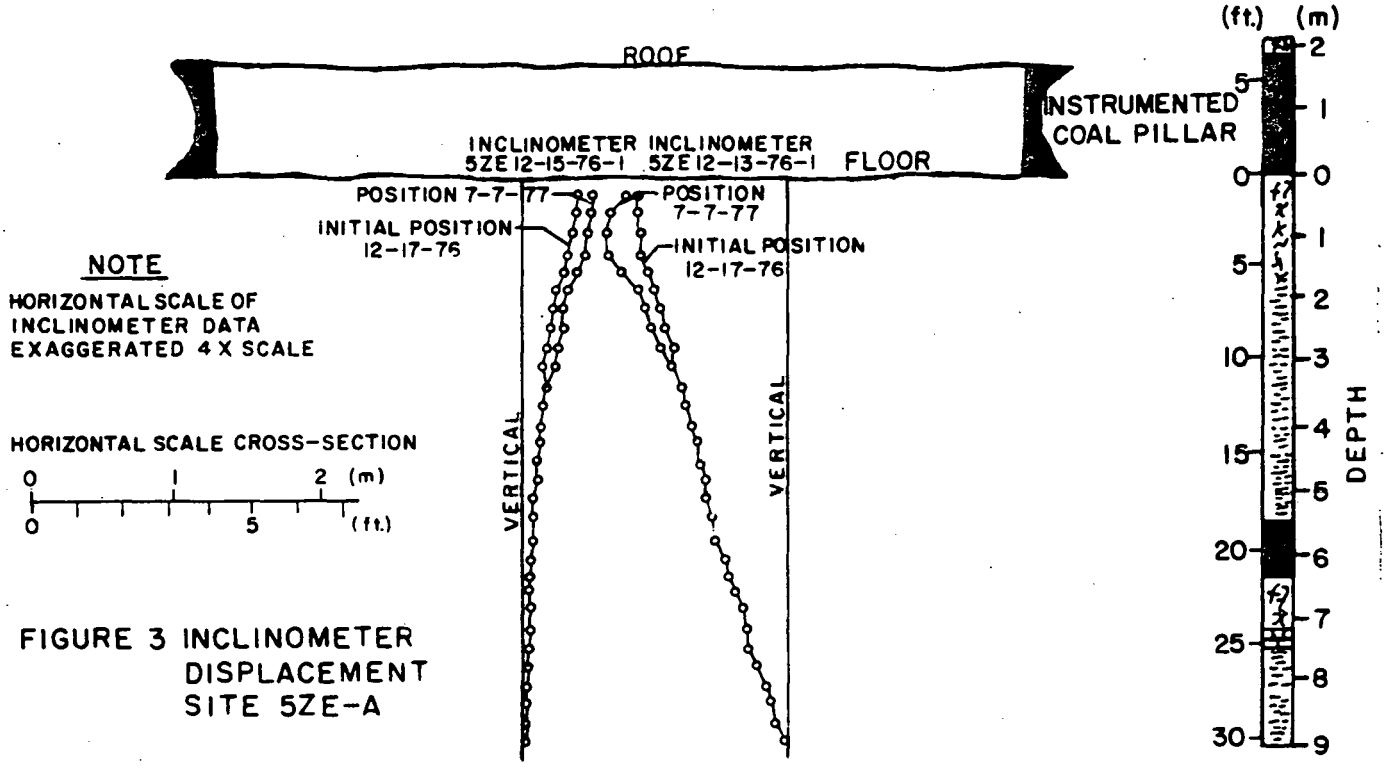



FIGURE 3 INCLINOMETER
DISPLACEMENT
SITE 5ZE-A

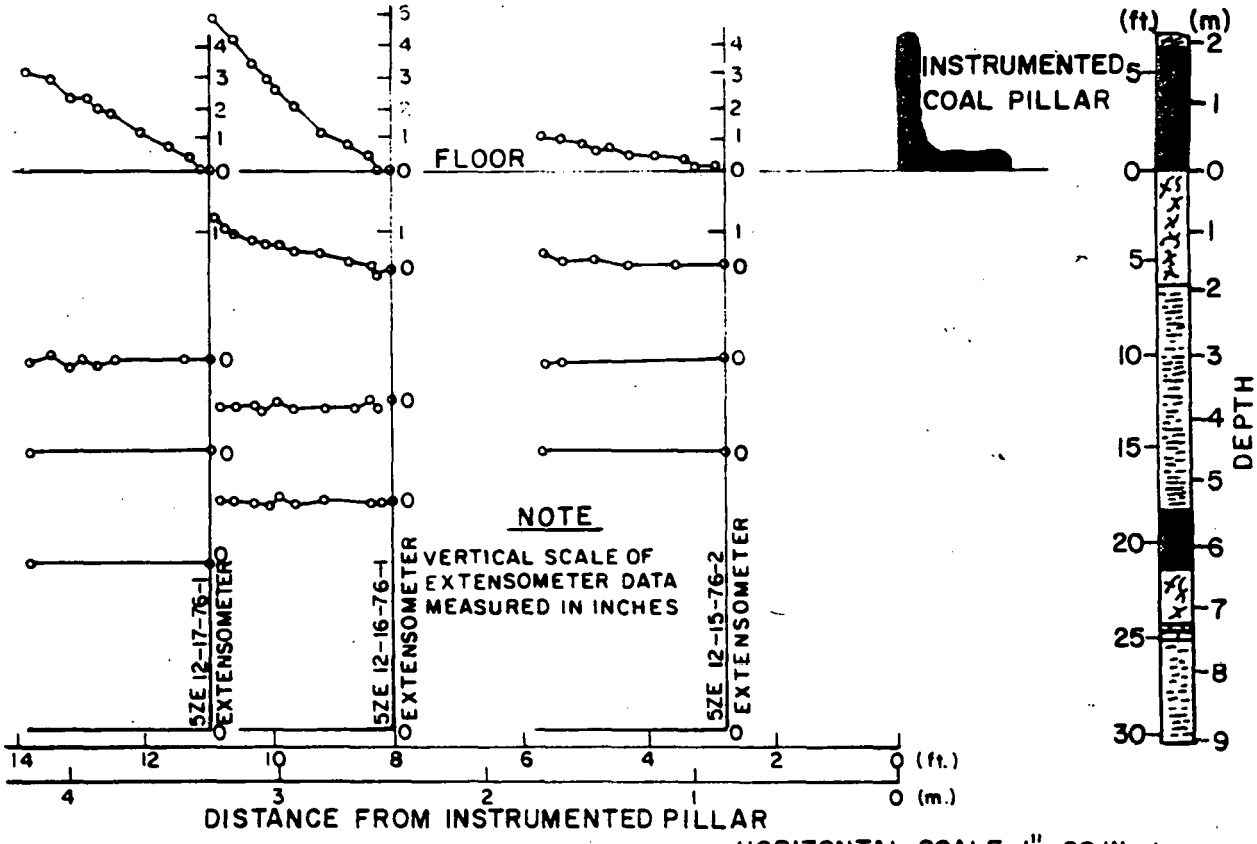



FIGURE 4 EXTENSOMETER
DISPLACEMENT
SITE 5ZE-A

those over 250 psi, were obtained from samples with the properties of a claystone or from those which contained limestone nodules in the matrix.

The potential relationships between triaxial strength and the other physical properties tested were analyzed using a multiple linear regression model to identify and define the form and degree of association that existed among the data. Of these, the parameter that correlated most highly with triaxial strength was natural water content, which had an inverse relationship and a correlation coefficient of -0.70. As a general rule, the underclays were characterized by relatively high water contents and low strengths, whereas the claystones were associated with lower water contents, but higher strength values (Figure 5).

Evaluation of the data from insitu monitoring stations illustrated the significance of the underclay strata in the failure mechanism. In this study all of the sites monitored had a variable, but relatively thick strata of underclay, underlain by firmer claystone and limestone. Under these conditions, the mode of failure expected would be a general shear failure in which there is an extrusion effect from under the pillar base and a bulging of the mine floor. This was indicated very clearly by the inclinometer and extensometer data for these sites. As illustrated, (Figures 3 and 4), the magnitude of this upward heave was more pronounced toward the middle of the entryway and was of a smaller magnitude near the coal pillar. It is apparent from Figures 1, 2, 3, and 4, also, that the direction of this movement is upward and toward the center of the mined-out area, away from the loaded pillars. In addition, it may be observed from Figures 3 and 4, that most of this movement occurred at relatively shallow depths and for the most part, was confined to the strata immediately beneath the coal.

## ANALYSIS OF BEARING CAPACITY

It is necessary to consider the sequence of subcoal strata, as well as the relative thickness of the individual strata in the bearing capacity analysis for pillar support, since all of the members participate in the distribution of stresses applied by the pillars.

The classical bearing capacity equation, developed by Karl Terzaghi is:

$$q_{ult} = cN_c + \gamma D_f N_q + \frac{1}{2} B\gamma N_\gamma \qquad (1)$$

where: $q_{ult}$ = ultimate bearing capacity

$N_c$, $N_q$ and $N_\gamma$ = bearing capacity coefficients

c= cohesion

$\gamma$= unit weight of material

$D_f$= depth of foundation

B= dimension of foundation

However, the assumptions of an infinitely thick bearing stratum and an isotropic and homogeneous material required for this equation obviously were not applicable to the analysis of underclays and subcoal strata, and therefore the equation had to be modified.

One approach to reduce the limitations caused by these assumptions, developed by Vesic (1975), evaluates the influence of the bearing capacity of a soft soil which overlies a firm soil. For this purpose Vesic has proposed the following equation:

$$q_{ult} = c_1 N_m + q \qquad (2)$$

where: $c_1$ = undrained shear strength of upper layer

$N_m$ = modified bearing capacity factor which depends upon the ratio of the shear strengths of the two layers

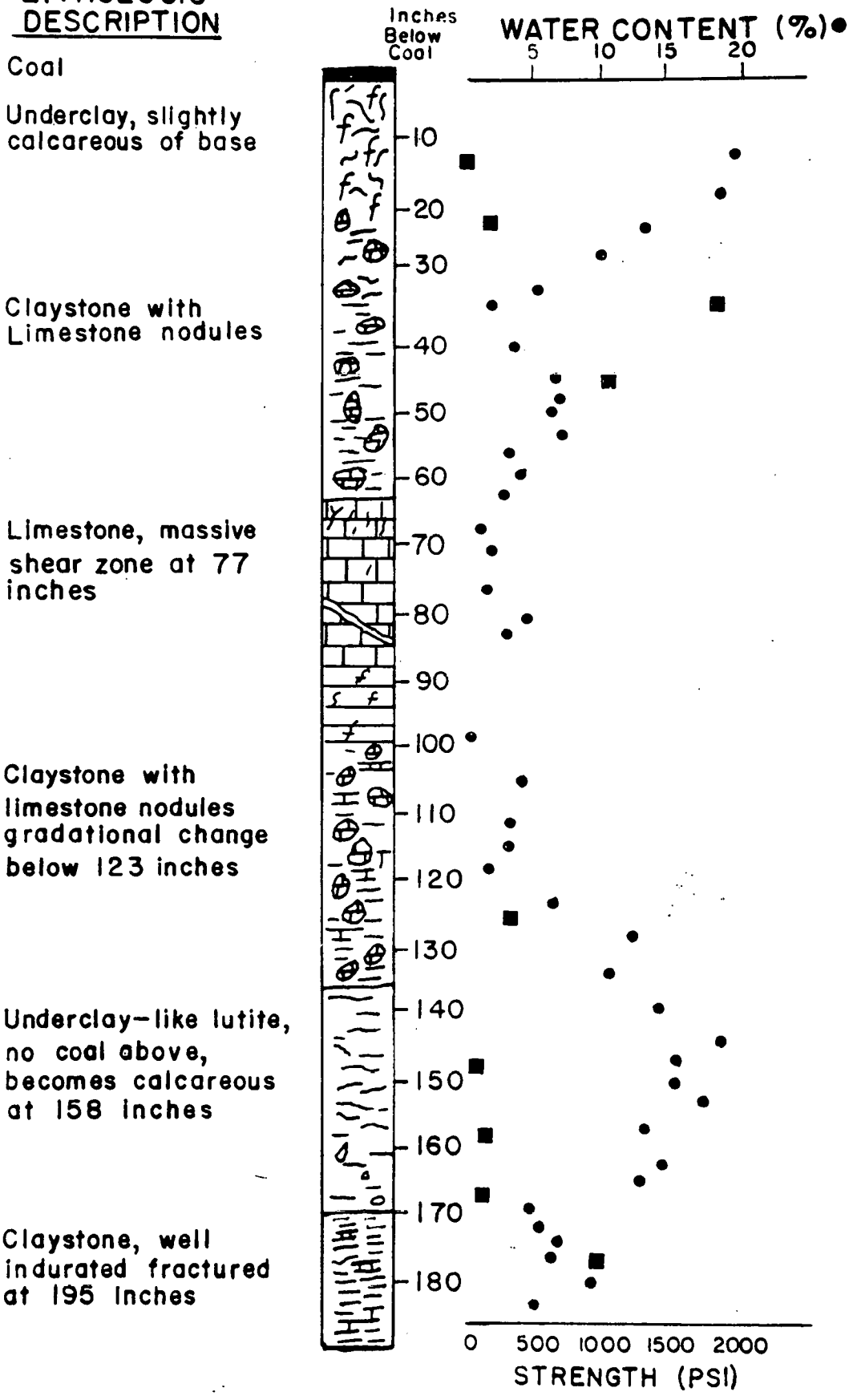



Figure 5 Relationship between Water Content, Triaxial Strength and Lithology. SITE 5ZE–A

ratio of the shear strength of the two layers

$k = c_2/c_1$,

q= surcharge

The highly fissured nature of the overconsolidated underclays complicated the analysis of bearing capacity however, since measurements of the shear strength of underclay material generally give the intact strength. The influence of fissuring must be taken into account either by relative large scale insitu testing or by utilizing theories of bearing capacity of nonhomogeneous soil conditions. Such theories employ a reduction of the intact material strength (operational strength) to take into account the influence of fissuring.

The bearing capacity analysis was carried out for three mines that were studied in detail during the course of the investigation. In addition, two areas of apparently different subcoal conditions were selected for analysis in one of the mines. The data considered included depth of coal below the surface, pillar size and extraction ratios, underclay thickness, sequence of subcoal strata lithology, and the shear strength of the materials.

The results of the analysis are given in Table I. A comparison of the calculated factors of safety with observed mine floor stability indicated that the method provided a satisfactory analysis of existing insitu conditions. For example, of the mines investigated, Mine B was experiencing the most severe floor failure problems whereas Mine A had no indication of floor problems. In Mine C, the area C-1 had more significant floor heaving than area C-2.

TABLE I
SUMMARY OF BEARING CAPACITY ANALYSIS

| | MINE | | | |
|---|---|---|---|---|
| | A | B | C-1 | C-2 |
| Factor of Safety by Vesic Procedure | 1.5 | .79 | 1.3 | 1.5 |

The most significant parameter controlling the magnitude of the factor of safety was the shear strength of the underclay. The shear strengths used for analysis were derived from the results of the laboratory testing phase but the laboratory values were reduced to account for the influence of fissuring the sample selection bias. This reduction factor was determined from insitu plate loading tests conducted at the sampling sites. However, even though the selection of the appropriate shearing strength of the underclay was the most critical factor, the comparative evaluation also illustrated several significant points regarding effect of underclay and subcoal lithology on ultimate bearing capacity. For example, the bearing capacity analysis results for Mine A and B indicated a relationship between the underclay thickness and the pillar stability factor of safety. The underclay thickness in the unstable mine (Mine B) was more than 9 feet, whereas in the stable comparative mine (Mine A), the thickness was 4 feet or about half that of the unstable mine. This difference in thickness influenced the distribution of stress beneath the pillar, which was reflected in the calculation of the maximum bearing capacity $q_{ult}$.

This association was not as obvious in Mine C, but this mine had a more complex subcoal profile that may have obscured the relationship. In Mine C, the underclay graded transitionally into a firm limestone layer at a relatively shallow depth, but beneath the firm layer was another soft underclay-like material with very low strength values. This three-layer system resulted in a failure mode that was a combination of the simplified types discussed previously and

illustrated the significance of the position of the low strength strata in the subcoal profile on the selection of the appropriate form of bearing capacity analysis.

## CONCLUSIONS AND RECOMMENDATIONS

The underclays of Mine B were significantly weaker than those of Mine A or Mine C. This fact was the most important reason for the comparatively low factors of safety. Where the thickest deposit of the underclays occurred, the computed factors of safety against bearing capacity failure were the smallest. The actual mechanism of failure was governed by the stratigraphic position of the weak layer(s). Where a two-layer system existed with the low strength layer overlying the higher strength layer, the failure mode was one of general shear failure with large quantities of the subcoal strata being displaced from beneath the pillar, outward and upward into the mined-out opening. Where there was an additional weak strata at depth, the failure mechanism was more complex and was dependent upon the relative thickness and depth of the weaker zone.

The results of this study indicated that the failure of the strata supporting coal pillars can be evaluated using the procedure of bearing capacity analysis developed by Vesic for two-layer systems. In addition, the investigation illustrated that both the lithology and stratigraphy of the subcoal strata must also be investigated to evaluate coal mine floor stability since, a) the shear strength is a function of the mineralogy and texture and fabric of the clay; b) water content affects the development of strength and correlates very closely with both lithology and strength values; and c) underclay thickness and the stratigraphic position of the low strength layers influence the ultimate pillar support and mechanism of floor failure.

The application of these conclusions to pillar support studies in existing or future mines within the Illinois Basin lead to the following recommendations:

a) During the exploration phase of coal mine development, the subsurface borings should be extended through the underclay into the stronger subcoal strata. The lithologies and thicknesses of the prospective mine floor units should be very carefully determined and contour mapped since each foot of increase in underclay thickness (up to approximately 6 feet) signifies a relatively large corresponding decrease in mine floor bearing capacity. The basic physical properties (Atterberg limits, natural water content, triaxial strength) should be determined as quickly as possible. If laboratory strength testing is not possible for all samples, a relationship between natural water content and triaxial strength should be derived to estimate future underclay strength values.

b) The bearing capacity of the mine floor should be calculated prior to mine development. State-of-the-art bearing capacity procedures for two layer systems such as developed by Vesic are most appropriate for this analysis. Average triaxial compressive strength values, modified compressive strengths, and/or strengths estimated from natural water contents obtained from the sampling and testing program provide representative data for these calculations. This provides a best estimate of the anticipated bearing capacity from which the initial coal pillars may be designed. These designs may later be modified if necessary as additional insitu data becomes available and a knowledge of "operational" floor bearing capacity is gained. Areas of low bearing capacity should be defined

and mapped. These maps, combined with strength, natural water content, and thickness data of the subcoal strata, could be used as input to mine development plans. Potentially unstable areas might be best developed after other areas have been mined or, at a minimum, be rejected as future sites of shaft construction or permanently maintained entryways.

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# Effects of Moisture on Strata Control in Coal Mines

Yoginder P. Chugh, Roger A. Missavage

Department of Mining Engineering
Southern Illinois University, Carbondale, IL

Abstract. Moisture has been known to influence strata failures in coal mines for a long time. However, it is only in the last decade that researchers have attempted to quantify this effect. In the laboratory, attempts have been made to study the effects of moisture on the strength of coal bearing rocks, moisture gain in shales as a function of time and pressures generated due to swelling, slaking and swelling indices and their possible correlation to strata control problems, and effect of humidity variations on anchor creep and bearing plate performance in conjunction with conventional bolts. In the field, researchers have attempted to correlate humidity variations with the incidence and frequency of roof falls and roof convergence, and effects of air tempering on reducing humidity variations in the mine.

A concise review of what is known about effects of moisture on strata control with emphasis on the Illinois basin coal mines is presented in this paper. Some approaches to mitigate the effects of moisture--leaving coal in the roof, guniting and shotcreting, application of sealants--are then briefly discussed.

## Introduction

Ground failures (roof, face, sides and floor) account for about 50 percent of the fatal and non-fatal injuries in the U.S. coal mines. In addition these failures cause substantial loss of production, poor image of the industry, and labor turnover. In Illinois alone, over 100 falls of roof, face, and sides are reported annually.

Effects of environment (moisture, barometric pressure, and temperature) on strata control in coal mines have been studied in some depth in the past one decade. Such effects become very important where shales, which are known to be sensitive to moisture, occur immediately above or below a coal seam being mined as in the Illinois coal basin. Several studies, Stateham and Radcliffe, 1976; Aughenbaugh and Bruzenwski, 1973; Haynes, 1975; Hartman and Greenwald, 1941 among others, have shown that moisture has a significant effect on shales and stability of coal mine roof and floor. The variation in freqency of roof falls, covergence and roof bolt loads can be accounted for by seasonal humidity changes. The weathering of shale mine roofs so commonly observed in Illinois coal mines is due to low durability of shales in humid enviroments. Squeezing ground conditions observed in some areas of the basin may be related to moisture absorption in claystones.

This paper presents results of the past laboratory and field studies to understand and evaluate the effects of moisture on strata control. Some measures to mitigate the effects of moisture on strata control are then discussed.

The discussion has been organized under the following sections: 1) evidence of moisture effects on shales and roof stability, 2) variation of strength and deformation properties of shales as a function of moisture content, 3) slaking and swelling properties of shales, 4) field studies on the effect of moisture on strata control in mines, and 5) measures to reduce moisture effects in coal mine ground control.

## Evidence of Moisture on Strata Control

The coal miners have long observed the effects of changing seasons on roof stability. The terms "summer roof" and "winter roof" have resulted from these observations. Several researchers (Hartman and Greenwald, 1941; Parker, 1966, 1970; Haynes, 1975; Aughenbaugh and Bruzewski, 1973; Stateham and Radcliffe, 1976) observed correlation between roof falls and environmental conditions. Hartman et al (1941) studied in the laboratory the effects of moisture on shales overlying Pittsburgh coal bed. Parker (1966) correlated monthly frequency of roof falls with seasonal humidity changes in White Pine copper mine in Michigan. He also observed relationship between lateral extent of roof falls and fogginess (100% humidity) in a section. Stateham and Radcliffe (1976) have observed, based on four year data (1972-1975) on roof falls in the U. S. coal mines, that the frequen-

cy of roof falls follows the yearly absolute humidity cycle, Figure 1, and lags it by about 14 days. Aughenbaugh and Bruzewski (1973) have shown that the rate of convergence in Illinois coal mines follows the humidity cycle and is high during the spring and summer months and low during the fall and winter months. They also observed, based on roof bolt load studies, that most roof bolt slipping occurred during high humidity months. Haynes (1975) studied effects of moisture on roof shales in three Alabama coal mines and found that moisture had an adverse effect on roof stability. Both Haynes and Hartman et al found that the temperature variations had negligible effect on strata stability as compared to humidity variations.

Moisture gain and moisture loss in mine rock is related to seasonal changes in absolute humidity of mine air. In summer, absolute humidity of surface air is high and air temperature above those of rocks underground. When the hot surface air, drawn into the mine for ventilation, is cooled by rocks the air loses its moisture on rocks (sweating). In winter, surface air temperature and humidity are lower than rocks underground and air traveling through the mine gains temperature and humidity. This causes mine rocks to lose moisture and thus has the drying effect.

## Strength and Deformation Properties of Shales

Several investigators, Hartman and Greenwald, 1941; Haynes, 1975; Aughenbaugh and Bruzewski (1973, 1976), Chenevert (1970), Eeckhout and Peng (1975), Parker (1966), Chugh (1977), Conroy (1973), Chugh, Okunola, Hall (1980), have investigated the changes in the mechanical behavior of shales as a function of humidity and moisture content. These studies involved wetting the shale specimens over a period of time, or placing specimens at different humidities, or drying specimens to remove natural moisture content. Some pertinent comments with regard to these studies follow here:

1) Without exception, the compressive strength ($C_o$) and modulus of elasticity (E) decrease with increasing moisture content. Some of the data from different investigators are compiled in Table 1 and are shown in Figures 2-3. The $C_o$ values immersed in water or placed in 100% relative humidity (R·H) for a period of at least 24 hours is decreased 50-60 percent as compared to its natural moisture content. The ratio of strength for oven dry to completely wet specimens ranges from 2-10.
2) Eeckhout and Peng, and Chugh observed increased compliance (decrease in the value of E and increase in the value of Poisson's ratio ($\mu$). The rate change of compliance was lower at low R·H and increased sharply beyond 50% R·H. The compliance across bedding plane at high R·H increased very rapidly as compared to compliance along the bedding plane.
3) Average sclerscope hardness and fracture toughness decreased with increasing R·H (Aughenbaugh and Bruzewski, 1973)
4) Moisture absorption in shales is a surface phenomenon and therefore depends upon the size and shape of specimens.
5) Moisture absorption rate is much larger along the bedding planes than across the bedding planes. Thus if the immediate roof is fractured, the moisture migration can occur along the bedding planes increasing the susceptibility of shale deterioration.

The effects of decreased strength and deformation properties are to decrease safe mine opening span, increase roof convergence and decrease size of intersections.

## Moisture Absorption in Shales

The natural moisture content of shales in the basin is highly variable and some available data are summarized in Table 2. Shales gain or lose moisture when placed in chambers with different relative humidities. Moisture gain occurs if the humidity is above their current level or lose moisture if

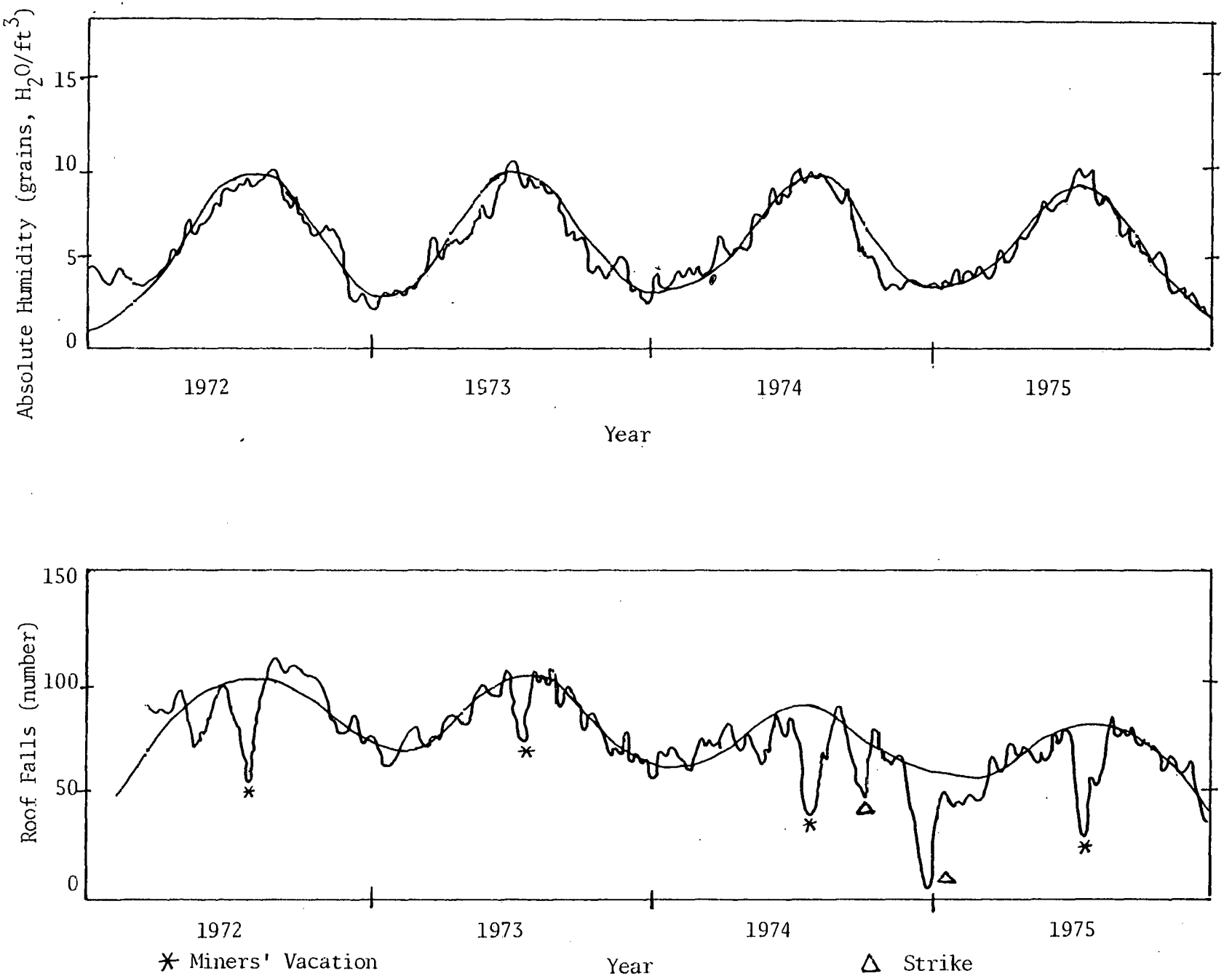


Figure 1. A Comparison of Total Roof Falls and Absolute Humidity (Stateham and Radcliffe, 1976)

Table 1. Effects of Moisture on Strength and Deformation Properties of Rocks

| Type of Rock | Unconfined Compressive Strength (PSI) | | | Modulus of Elasticity (PSI x $10^6$) | | | Unconfined Tensile Strength (PSI) | | | Reference |
|---|---|---|---|---|---|---|---|---|---|---|
| | Dry | Wet | Wet/Dry | Dry | Wet | Wet/Dry | Dry | Wet | Wet/Dry | |
| Siltstone | 29700 | 10200 | .34 | -- | -- | -- | 2200 | 1360 | .62 | Parker, |
| Sandstone | 19500 | 10500 | .54 | -- | -- | -- | 1500 | 850 | .56 | 1970 |
| Marble | -- | -- | .95 | -- | -- | -- | -- | -- | -- | |
| Limestone | -- | -- | .72-.83 | -- | -- | -- | -- | -- | -- | |
| Granite | -- | -- | .86 | -- | -- | -- | -- | -- | -- | |
| Sandstone | -- | -- | .45-.89 | -- | -- | .7-.84 | -- | -- | -- | Van Eeck- |
| Slate | -- | -- | .8 | -- | -- | -- | -- | -- | -- | hout (1976, |
| Coal Shale | -- | -- | .25-.65 | -- | -- | .43 | -- | -- | -- | 1979) |
| Quartzitic Shale | -- | -- | .52 | -- | -- | -- | -- | -- | -- | |
| Quartizitic Sandstone | -- | -- | .51 | -- | -- | .75 | -- | -- | -- | |
| Dolerite | -- | -- | .65 | -- | -- | .97 | -- | -- | -- | |
| Sandstone | 4600-12700 | 2800 | .61 | .47-1.12 | .34 | .3-.72 | 550-900 | 260-530 | .47-.59 | |
| Silty Shale | 2800-8100 | * | -- | .68-1.04 | * | -- | 220-700 | -- | -- | Chugh, 1977 |
| Shale | 6400-9200 | * | -- | 1.1-1.4 | * | -- | 1350 | -- | -- | |
| Gray Shale | 3100-8400 | * | -- | .78-1.14 | * | -- | 220-1010 | 120-480 | .47-.54 | |
| Shale | 27000-32000 | 15000-17000 | 0.53-0.55 | -- | -- | -- | 1700-2200 | 1250 | 0.55-0.70 | Scott, 1969 |
| Shale(Dykersburg) | 8000-9500 | 1800-2500* | 0.22-0.26 | -- | -- | -- | -- | -- | -- | Aughenbaugh and Bruz- eweski, 1976 |
| Shale(Dykersburg) | 5320 | 1685* | 0.31 | 0.425 | 0.084* | 0.198 | -- | -- | -- | Chugh et al(1980) |

*100% Relative Humidity

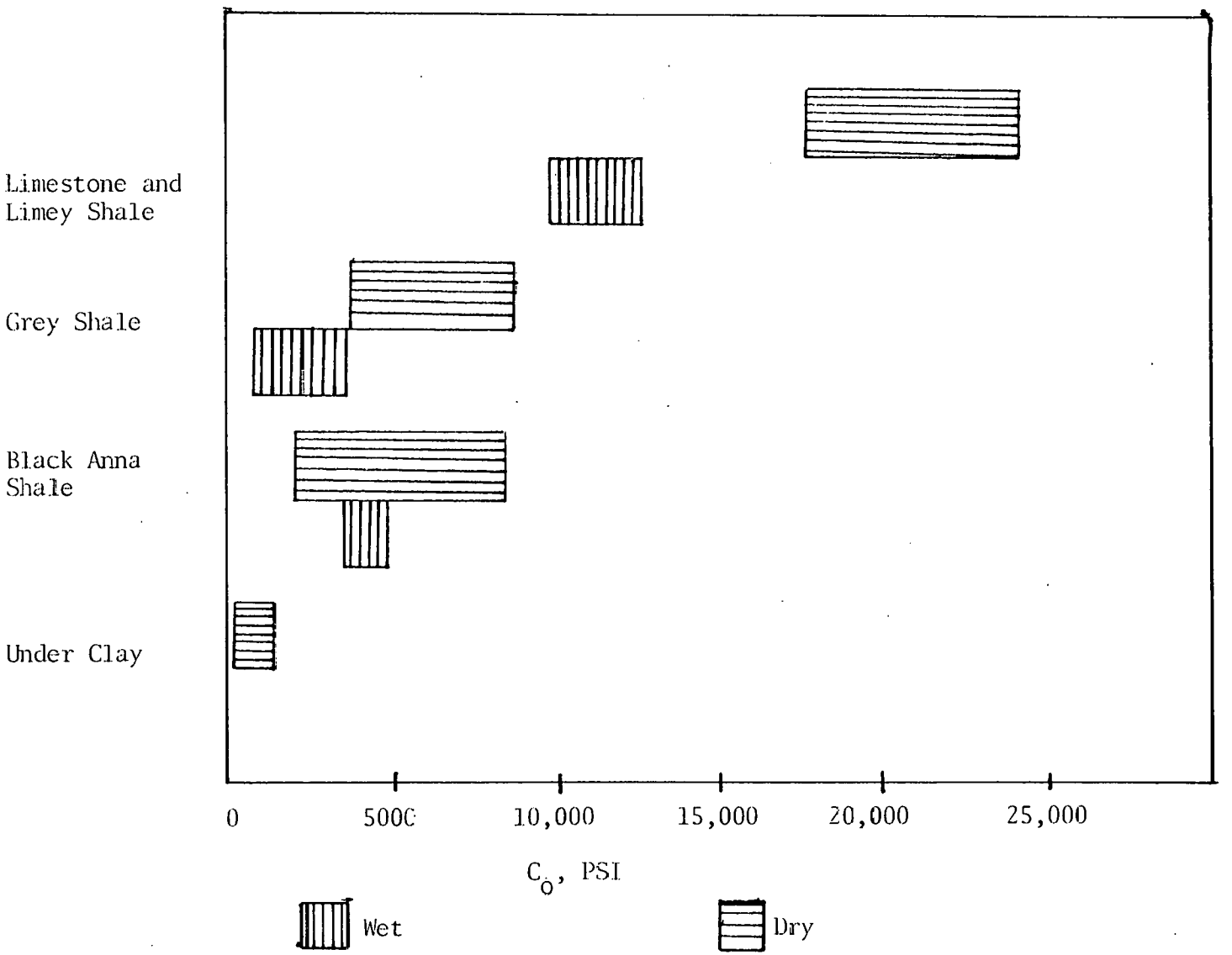


Figure 2. Effect of Wetting on Compressive Strength of Rocks Overlying Herrin Coal Seam (Chugh, 1977)

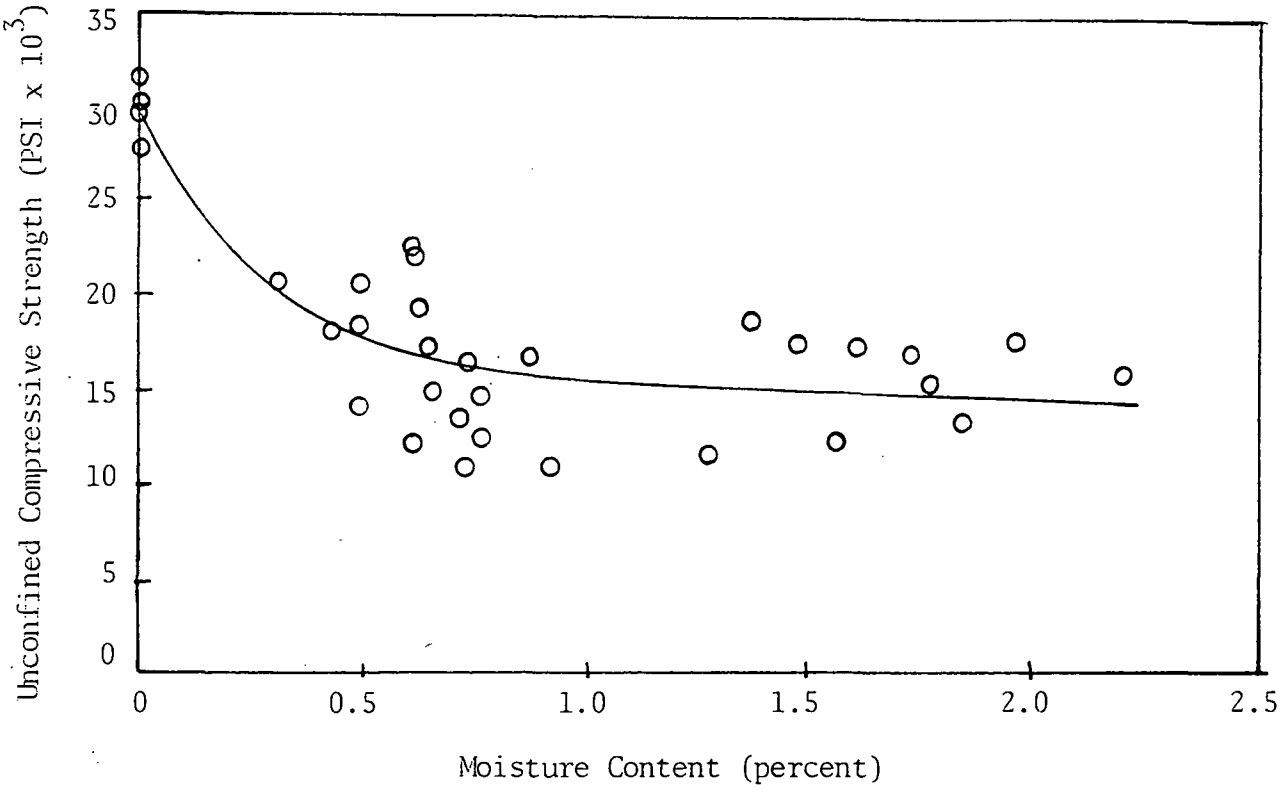


Figure 3. Effect of Moisture Content on Compressive Strength of Shale (Scott, 1969)

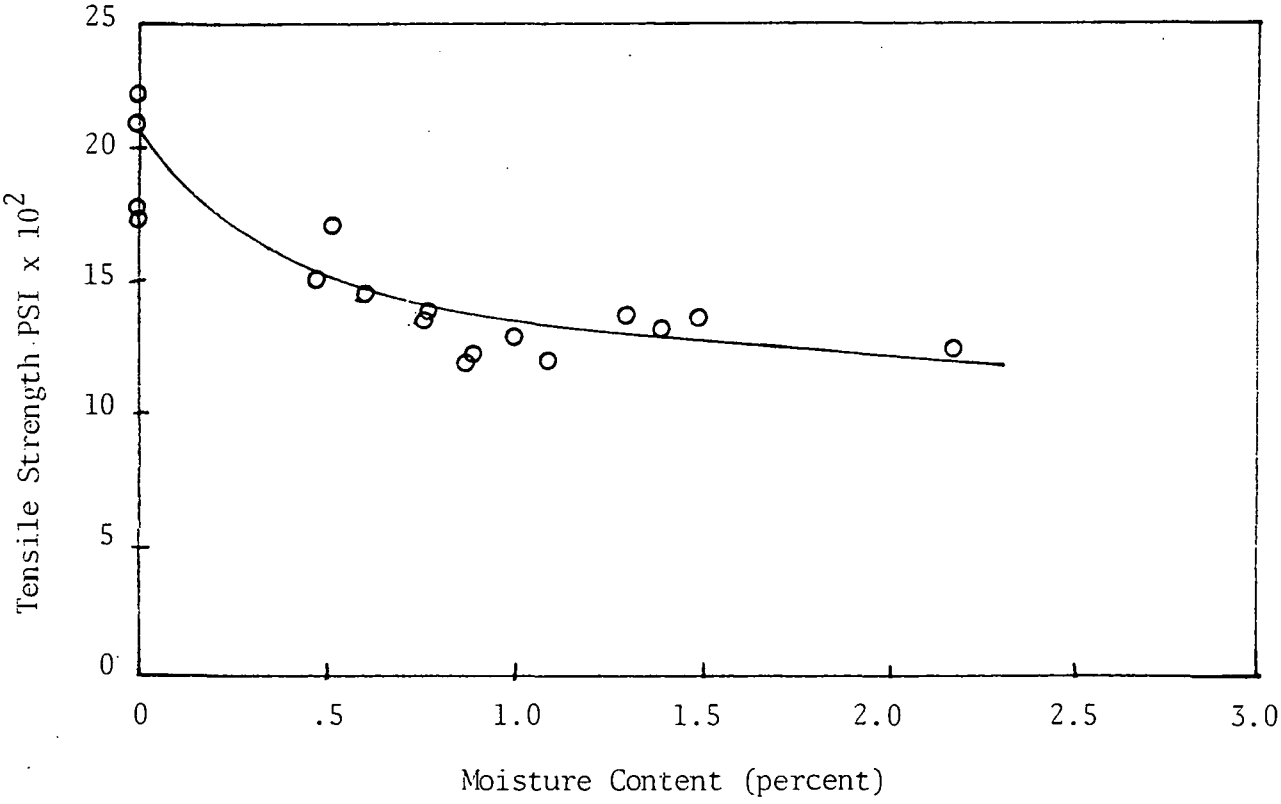


Figure 3. Effect of Moisture Content on Tensile Strength of Shale (Scott, 1969)

Table 2. Natural Moisture Content of Coal Measure Rocks in the Illinois Coal Basin

| Type of Rock | Natural Moisture Content (Percent) | | | | Number of Samples | Source |
|---|---|---|---|---|---|---|
| | Minimum | Maximum | Average | Standard Deviation | | |
| Limestone and Limey Shales Above Herrin Seam | 1.60 | 2.30 | 1.99 | 0.39 | 6 | Chugh, 1980 |
| Gray Shale above Herrin Seam | 1.80 | 7.48 | 4.59 | 1.95 | 14 | Chugh, 1980 |
| Black Anna Shale Above Herrin Seam | 3.28 | 7.43 | 5.64 | 1.40 | 12 | Chugh, 1980 |
| Underclay Below Herrin Seam | 7.10 | 13.73 | 8.93 | 1.82 | 13 | Chugh, 1980 |
| Gray Sandstone (Near Anvil Rock Sandstone Channel) | 3.47 | 5.92 | 4.81 | 1.24 | 3 | Chugh, 1980 |
| Gray Shale above Herrin Seam | 0.1 | 10.80 | 5.49 | 3.22 | 20 | Conroy, 1973 |
| Dykersburg Shale above Harrisburg Seam | 2.7 | 3.5 | 2.98 | 0.47 | 5 | Conroy, 1973 |
| Dykersburg Shale above Harrisburg Seam | 1.86 | 3.61 | 2.35 | 0.67 | 6 | Chugh, 1980 |
| Gray Siltstone (Below Harrisburg Seam) | 2.62 | 3.90 | 3.26 | 0.90 | 2 | Chugh, 1980 |

the humidity is below their current level. Some of the data reported by different investigators are summarized in Table 3. Moisture gain as a function of time takes the form of an exponential curve. Aughenbaugh and Bruzewski (1976) found that all shales have a unique relative humidity index (slope of the percent changes in weight vs. relative humidity curve) and this index can be used as an indicator of the susceptibility of shales to moisture. They also concluded that daily humidity changes influence moisture absorption or desorption in shales. The data are presented in Figure 4.

The gain or loss of moisture is a surface related phenomenon and is therefore dependent upon size and shape of specimens. Moisture absorption is much higher along the bedding planes than across the bedding planes. Hartman and Greenwald, Haynes, and Aughenbaugh and Bruzewski further observed that the effect of varying temperature on moisture absorption in mines was not significant since annual temperature variations in a mine were relatively small.

## Slaking and Swelling Behavior of Shales

### Swelling Properties

Shales increase in volume (swell) when they come in contact with moisture. Swelling tests include placing specimens in environmental chambers with controlled temperature and humidity or immersing specimens in water and measuring longitudinal and lateral strains. Swelling strains parallel and perpendicular to bedding planes under unconfined state have been studied by several investigators and the results are summarized in Table 4. A typical plot of strain perpendicular and parallel to bedding plane as a function of time after immersing in water is shown in Figure 5. The plot is exponential and ultimate swelling strain of 4-10 percent was observed in the direction perpendicular to the bedding plane. Only 1-2 percent swelling strain is observed parallel to the bedding plane. Greenwald and Hartman (1941) and Chugh et al. (1980) observed during their tests in environmental chambers that swelling strain increased exponentially to a relatively constant value after 7-10 days. This would indicate that daily humidity changes should have samll effect on roof shales but seasonal humidity changes would have significant effect on roof stability.

Swelling strains have been observed to be elastic or of plastic nature. The plastic or permanent strains generally occurred perpendicular to bedding plane indicating some type of irreversible physical (cracking) or chemical process.

Parker (1966) studied the variation of lateral swelling strain with distance from exposed surface. He observed that swelling strains mostly occurred within two inches nearest the exposed surface.

### Induced Swelling Pressures

If lateral swelling strains are not permitted to occur (confined swelling tests) as in a mine, lateral swelling stresses are induced. Aughenbaugh and Bruzewski, Haynes, Parker have studied in the laboratory as well as in the field swelling pressures generated due to moisture absorption in shales. The swelling pressures reported varied from 4200-14,000 psi; the higher values are for Illinois basin shales while the lower values are for Alabama shales. Such pressures can cause failure in most shales. The failure of shale samples from Illinois basin consisted of a rim 1/4" wide and a central undisturbed core. This was interpreted as expansive shearing of the rim and tensile failure in the center. Chugh, Okunola and Hall (1980) have calculated swelling pressures from observed swelling strains and effective modulus of elasticity on specimens after swelling tests. The swelling pressures were computed to be as high as 3900 psi.

Swelling is caused by absorption of water by clay minerals especially montmorillonite, illite, and mixed-layer clay minerals. Murray (1954) in his studies of the shales from the Applachian region as well as the Interior coal basin did not observe montmorillonite

Table 3. Moisture Gain or Loss In Shales
At Different Relative Humidities (Base R·H = 50%)

| Rock Type | Location | Moisture Gain or Loss (Percent) 10% R·H | 30% R·H | 70% R·H | 90% R·H | 100% R:H | Reference |
|---|---|---|---|---|---|---|---|
| Shale | S. W. Texas | -1.0 | -0.5 | +0.5 | +1.5 | +2.7 | Chenevert, 1970 |
| Shale | Gulf of Mexico | -3.6 | -2.4 | +4.1 | +27.4 | ---- | |
| Shale | Pacific Coast | -4.1 | -2.4 | +3.0 | +7.7 | ---- | |
| Shale | Louisiana | -2.0 | -1.0 | +1.2 | ---- | ---- | |
| Shale | E. Texas | -1.2 | -0.5 | +0.6 | +2.4 | ---- | |
| Shale | Central U. S. | -0.86 | -0.6 | +0.8 | +2.8 | ---- | |
| Beatrice Shale | ----- | -0.50 | ---- | ---- | ---- | +0.3 | Eeckhout and Peng, 1975 |
| Shale | Southern Illinois | -0.60 | -0.3 | +0.5 | +1.6 | +2.6 | Aughenbaugh and Bruzewski, 1976 |

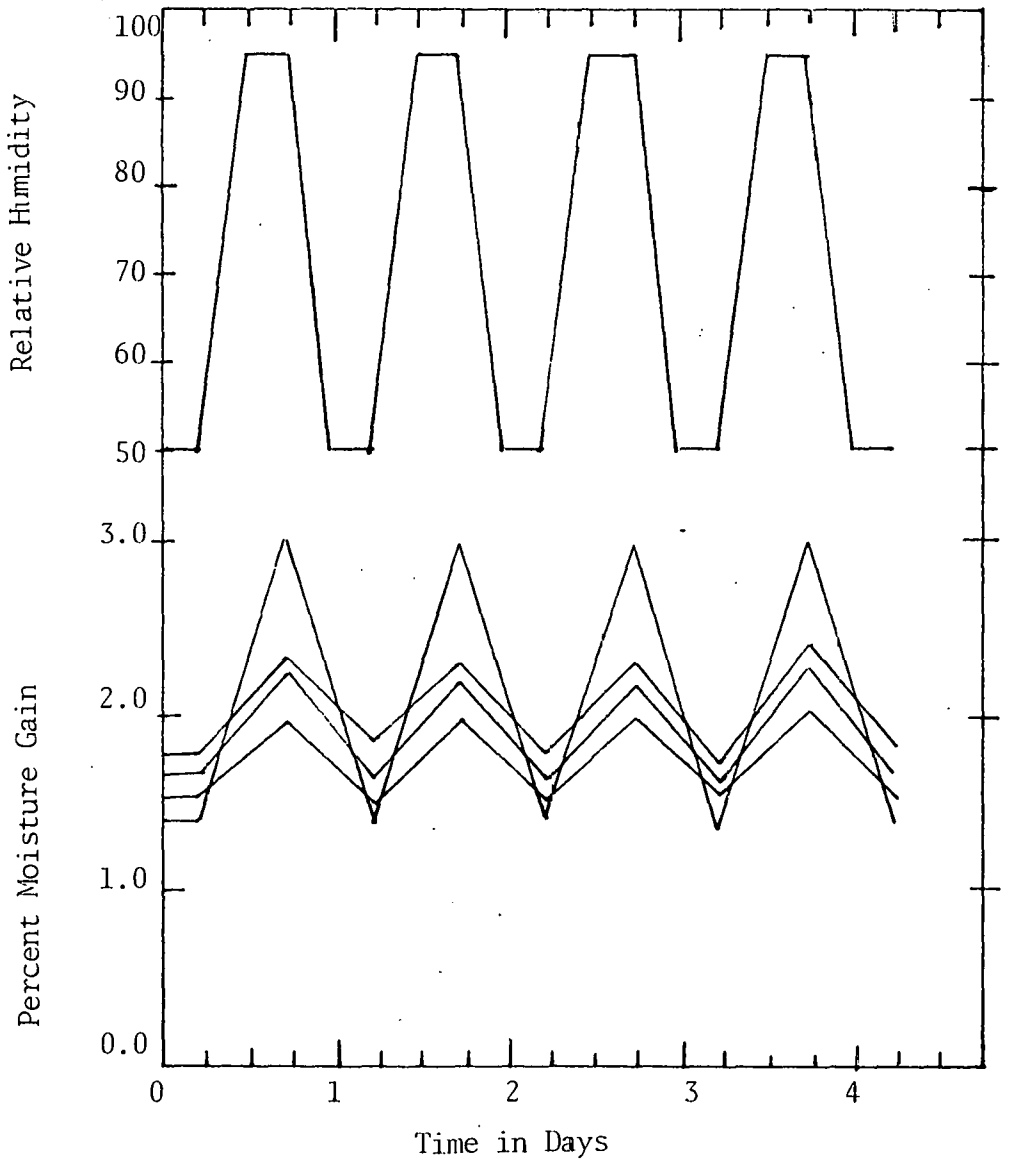


Figure X- Effect of Daily Humidity Changes on Moisture Absorption in Shales (Aughenbaugh and Bruzewski, 1976)

Table 4. Swelling Strains For Some Coal Measure Rocks

| Rock Type | Location | Percent Swelling Strain Perpendicular to bedding | | Percent Swelling Strain Parallel to bedding | | Remarks | Reference |
|---|---|---|---|---|---|---|---|
| | | Minimum | Maximum | Minimum | Maximum | | |
| Limestone and Limey Shales above Herrin Seam | Southern Illinois | 0.03 | 0.20 | 0.0 | 0.05 | Immersed in Water | Chugh, 1980 |
| Gray Shale above Herrin Seam | Southern Illinois | 0.60 | 6.4 | 0.2 | 2.40 | Do | Chugh, 1980 |
| Black Anna Shale | Southern Illinois | 0.70 | 1.90 | 0.0 | 0.3 | Do | Chugh, 1980 |
| Dykersburg Shale | Southern Illinois | 0.75 | 2.52 | 0.03 | 0.09 | Do | Chugh, 1980 |
| Dykersburg Shale | Southern Illinois | ---- | 0.48 | ---- | ---- | At 100% Humidity | Chugh, 1980 |
| Draw Slate | Pittsburgh | 0.70 | 1.30 | 0.35 | 0.37 | Immersed in Water | Hartman and Greenwald |
| Beatrice Shale | ------- | ---- | 0.90 | ---- | 0.35 | Immersed in Water | Van Eeckhout and Peng, 1975 |

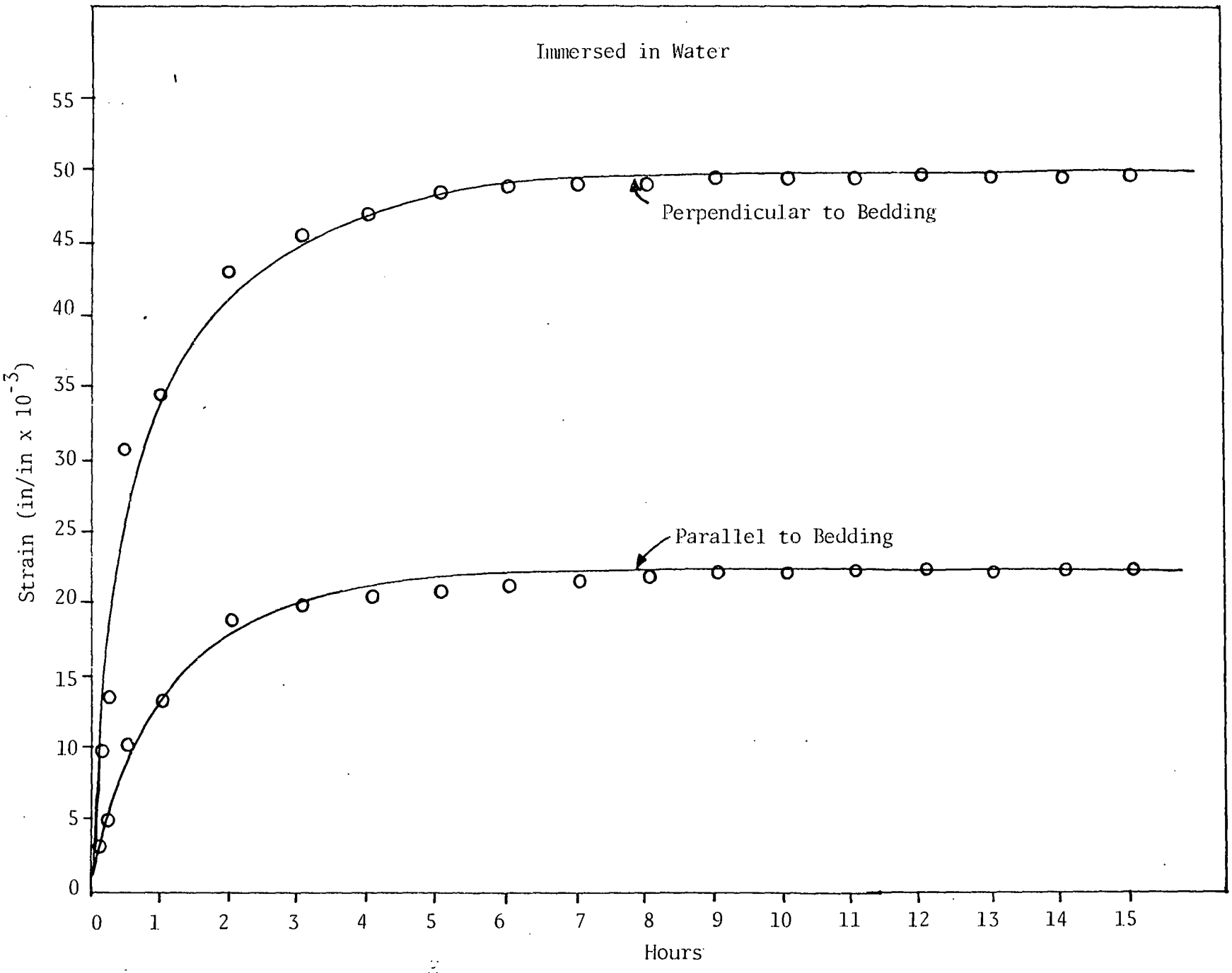


Figure 5. Unconfined Swelling Strains as a Function of Time For Grey Shale (Chugh, 1980)

clay mineral. Illite was most abundant followed by kaolinite and chlorite. Conroy (1973), however, did observe montmorillite and mixed-layer clays in the Illinois basin shales overlying No. 6 and No. 5 coal seams.

Holland (1956) indicated that swelling strains depended upon chemical composition of clays, exchangeable ions, particle size, voids, internal structure of the shale, and nature and magnitude of the confining stress. Davidson (1949) indicated that swelling of monmorrillonitic clay may be influenced by certain chemicals such as water soluble cationic amine acetate. Similarly kaolinitic soil swelling is influenced by fatty amine acetate (Davidson and Glab, 1949).

## Slaking Properties

Most shales disintegrate to some degree when they come in contact with moisture. This can result in severe weathering problems leading to rashing of immediate roof and localized roof falls in some cases. Holland (1956) discussed at some length the effect of mineral content of shales on weathering of mine roof.

Mielenz and King (1955) postulated the mechanism of slaking in shales as follows: 1) during drying, shrinkage cracks are formed which are filled with air, 2) swelling of shale during readsorption cycle causes air to be trapped causing tensile stresses in shale, 3) differential swelling pressures are set up during the readsorption cycle, and 4) combination of tensile stresses and differential swelling causes slaking of shales.

Some shales contain from small to large amounts of iron sulfides. In the presence of moisture and oxygen these convert to $FeSo_4$ and $H_2So_4$ (Burke and Downs, 1938). Ferrous sulfate can absorb water and swells considerably. Swelling pressures could cause localized failures. The sulfuric acid can chemically react with feldspars, carbonates and some of the clay minerals to form new compounds which can shrink and swell causing differential swelling pressures and result in weathering of shales.

White (1954) in his studies indicated that Illinois basin shales containing kaolinite and crystalline Illite and Chlorite are associated with good mine roof. Shales which contain montmorillonite and mixed-layer clays generally form bad roof. Sandy shales are generally good (Meyerton, 1956). Silty shales generally form poor roof. Roof shales are generally unsatisfactory if they contain more than a few tenths of a percent of iron sulfide. Some of the roof shales may contain appreciable quantities of soluble salts. These may chemically react with liquids and contribute to roof failure.

Laboratory tests, Franklin and Illinois slake tests (Aughenbaugh and Bruzewski, 1976) are available to evaluate slake durability of shales. However, none of them give satisfactory results. Some of the data developed by different investigators are given in Table 5.

# Field Studies of Moisture Effects on Strata Control

## Humidity and Temperature Variations in Mines

Aughenbaugh and Bruzewski (1973) observed that large temperature and humidity fluctuations on the surface are considerably dampened as the air travels down the intake shaft and intake airways. The velocity of air has a significant effect on the distance required for stabilization of air to near constant humidity and temperature. Seasonal humidity variations are still experienced in the intake airways even though amount of variation is considerably reduced. The seasonal humidity variations on the return airways are completely eliminated.

During summer, considerable water is absorbed in the mine from mine air. Most of this is deposited on the rock in the form of surface condensation; a portion of this is absorbed in the rock itself. This is indicated by continuous gain of moisture by mine air in winter after the rock surface has dried out. During winter, natural water content of rocks is considerably lowered and this increases the suscep-

tibility of rocks to gain moisture during summer. At the face, the air is fully saturated.

Several investigators, Aughenbaugh and Bruzewski, 1976; Haynes, 1975; Scott, 1969; Parker, 1970 have studied in the field the effects of moisture on strata control in coal mines and mine openings in shales. Some of the pertinent observations made were:

1) Compression type failures due to swelling pressures are observed. In some cases swelling pressure may cause the whole structure to fail.
2) Progressive failure of roof layers may cause slab type failures or plate type failures due to swelling perpendicular to bedding planes.
3) Shear movements occur along roof layers due to swelling parallel to bedding planes.
4) Squeezing ground conditions may occur in openings in shales which are bentonitic or illitic.
5) Moist air slaking can lead to severe weathering of immediate roof layers.
6) Time dependent failure of shales can lead to shear fractures along ribs in a mine opening or at intersections.
7) Rocks laminated with thin shale layers containing expansive clay may cuase shear fractures.
8) The convergence rate of roof shales seems to follow changes in the humidity cycle as shown in Figure 6.
9) Deterioration of roof conditions is generally associated with large roof convergence.
10) Slippage of roof bolts seems to follow the humidity cycle. This can be explained due to overstressing caused by swelling pressures.
11) Where moisture effects on strata control are significant, height of roof fall equals the length of roof bolts. This is due to moisture migration along the hole to the anchor point.
12) Moisture effects can have a significant effect on the efficiency of roof bolts. This could be due to degradation of anchor point, swelling or shrinking pressures at the bearing plates. Bolt load drop or increase of 2000-3000 lbs. have been recorded in the past (Oitto) and Goode, 1972, Oitto et al., 1970).

## Measures to Deal With Moisture Effects

The measures to deal with moisture effects can be subdivided under three sub-headings:

1) Control of humidity changes
2) Reducing moisture migration in shales
3) Rock reinforcement

## Control of Humidity Changes

Seasonal humidity variations in intake airways in the vicinity of a shaft are quite large and most ground instability conditions due to moisture effects are concentrated in these areas. Controlling the humidity changes implies that the surface air being circulated through the mine is conditioned to reflect temperature and humidity equilibrium conditions in the mine. This may be achieved by 1) tempering chambers, 2) air conditioning, and 3) continuous wetting.

Tempering Chambers: These basically consist of old worked out areas to condition the surface air before circulating it through the mine. The length and number of entries in a tempering chamber depend upon volume of air flow, surface air conditions and approximate equilibrium temperature and absolute humidity for the mine. Entries in tempering chambers are adequately supported with wire mesh, wooden props, and chocks. Tempering chambers represent additional resistance in the mine ventilation circuit.

Air Conditioning: Tempering of air may be done by using mechanical means. It is generally practiced in deep mines. Additional cost of air conditioning may not be justified.

Continuous Wetting: To reduce the effects of varying humidity levels, several mines have attempted continuous wetting of areas affected by varying humidity effects. It is not very popular.

## Reducing Moisture Migration

Moisture migration into shale occurs

Table 5. Franklin Slake Durability Index For Coal Measure Rocks in the Illinois Coal Basin

| Rock Type | Location | Franklin Slake Durability Index | | | | No. of Samples | Reference |
|---|---|---|---|---|---|---|---|
| | | Minimum | Maximum | Average | Std. Deviation | | |
| Limestone and Limey Shales overlying Herrin Seam | Southern Illinois | 97.7 | 98.8 | 98.3 | 0.78 | 3 | Chugh, 1980 |
| Grey Shale overlying Herrin Seam | Southern Illinois | 41.3 | 100.0 | 80.4 | 18.3 | 10 | Chugh, 1980 |
| Black Anna Shale | Southern Illinois | 90.7 | 99.0 | 94.9 | 3.5 | 6 | Chugh, 1980 |
| Underclay below Herrin Seam | Southern Illinois | 10.5 | 50.4 | 29.0 | 17.8 | 4 | Chugh, 1980 |
| Dykersburg Shale | Southern Illinois | 96.0 | 99.6 | 97.8 | 1.40 | 5 | Chugh, 1980 |
| Dykersburg Shale | Southern Illinois | 96.96 | 98.29 | 97.7 | 0.60 | 6 | Aughenbaugh and Bruzewski (1976) |

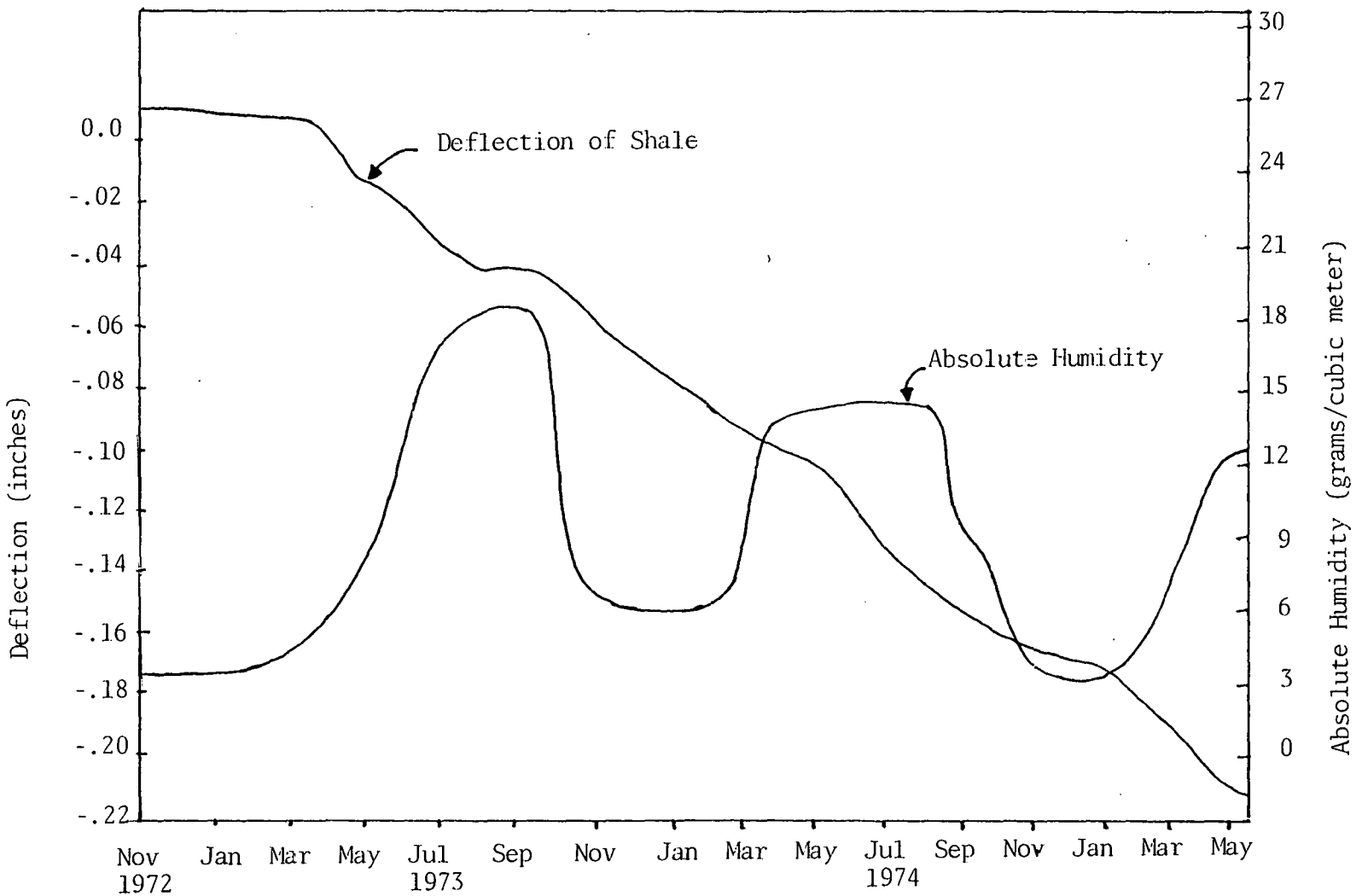


Figure 7. Effect of Seasonal Humidity Variation on Deflection of Shale Mine Roof (Aughenbaugh and Bruzewski, 1976)

directly by exposure, through roof bolt holes, fractures opened up by blasting, etc. Some of the measures recommended to reduce moisture migration are: 1) leaving coal in the roof 2) application of mine sealants 3) use of resin bolts 4) dry drilling, and 5) controlled blasting.

In the mid 70's, application of sealants gained a great deal of popularity. Mitchell et al. (1975) and Dwosh (1977) discussed briefly the different sealants in use at that time -- silicate based materials, portland cements, and magnesium-oxysulfate cements -- each with their own fillers and additives. Non-gypsum sealants cure very slowly. This caused two problems 1) in areas of high air velocity the material tends to artifically dehydrate causing cracked and weakened coating and 2) coal tends to loosen before sealants cure and harden sufficiently to add rib confinement. Chugh (1975) used steel reinforced gunite and dry shotcrete for stabilizing entries with shales in the immediate roof.

Resin bolts should prevent moisture migration along the bedding planes and to the anchor point thus preventing swelling of the immediate roof shales and degradation of the anchorage horizon.

Dry drilling should reduce moisture migration along bedding planes. Controlled blasting should considerably reduce roof convergence and subsequent moisture migration along bedding planes.

## Rock Reinforcement

None of the measures discussed above are completely satisfactory in eliminating moisture effects on strata control. Adequate rock reinforcement immediately after mining is the key to containing shales. It is recommended that mats or wire mesh should be utilized in conjunction with rock bolts. The rock surfaces may be sprayed with gunite or shotcrete after bolting. Wherever economically possible, resin bolts should be utilized. Additional support along the ribs such as inclined bolts or roof trusses should be considered.

## Concluding Remarks

Moisture has a very pronounced effe on ground control in rocks containing shales. The U.S. Bureau of Mines has conclusively proved this for ground failures in the U.S. coal mines (Stateham, Radcliffe, 1976). The swelling pressures due to moisture absorption can be very high (4000-14,000 psi) and are probably the major cause of ground failures. Such failures may occur in immediate roof (slab type failures), or along the ribs or at intersections as shear type failures. Although several measures could be taken to reduce the moisture effects, none of these are completely satisfactory.

Additional research is needed to 1) study the behavior of shales and underclays at different humidities, 2) obtain more data on convergence, sag, and load on supports in the field as a function of varying humidity, 3) determine effects of moisture on efficiency of roof bolts anchored in shales, and 4) modify slake durability tests to be representative of field experience.

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# THE LOSS OF NATURAL MOISTURE CONTENT AND ITS EFFECT ON THE MECHANICAL PROPERTIES OF SOME PENNSYLVANIAN SHALES FROM THE ILLINOIS BASIN

Robert A. Bauer

Illinois State Geological Survey, Urbana, IL 61801

## INTRODUCTION

The preservation of the natural moisture content of shale cores should be a primary concern to companies who are engaged in exploration and who want to know the physical properties of rocks associated with coal seams. To properly evaluate pre-mining roof and floor conditions of coal seams, testing should be conducted on cores that represent the natural conditions of the rock; therefore, the moisture content of these cores should resemble that of the rock mass being tested. As yet, researchers have not stressed the need for the preservation of natural conditions of rock core prior to testing. Top priority is given to this concern in the field of soil mechanics, however, where great care is taken to preserve the natural structure and moisture content of soil test samples so that strength test results are performed on samples that closely represent in situ conditions.

The effects of moisture content variation on the strength of rock cores have been demonstrated by researchers (1, 2, 3, 4, 5). The strength of cores which were artificially humidified to various levels and also air- or oven-dried changed considerably with the moisture content. No attempt was made to properly report or preserve the original moisture levels; however, no one but Broch (1) determined how rapidly moisture could be lost from the different rocks.

## MATERIALS AND METHODS

Core samples should be protected from moisture loss immediately after coring because they dry out very rapidly if left unprotected. A simple experiment demonstrated the extent of this loss. Pieces of core from the same shale unit were left unwrapped. Each day, the moisture content and the mechanical strength index of a different piece of core were determined. Moisture content was determined for pieces of shale from the centers of the cores. Figure 1 shows how much moisture was lost for two separate shale sample sets during periods of 4 and

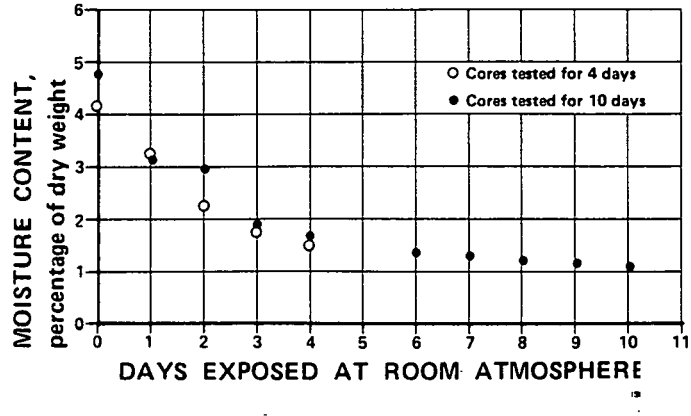


FIGURE 1. Rate of moisture loss from unprotected shale cores stored at room atmosphere (70° F)

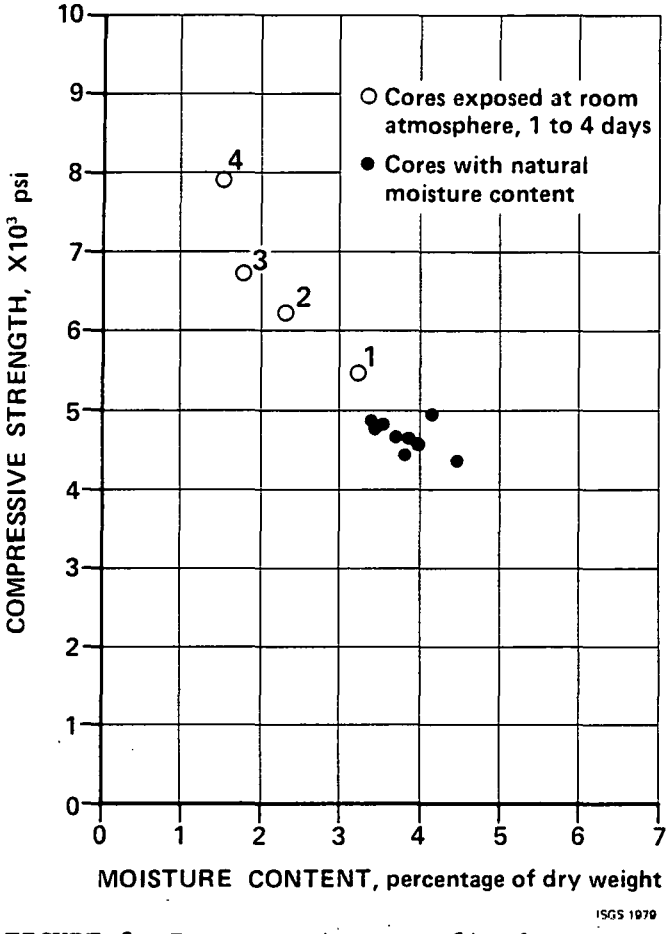


FIGURE 2. Increase in unconfined compressive strength due to moisture loss

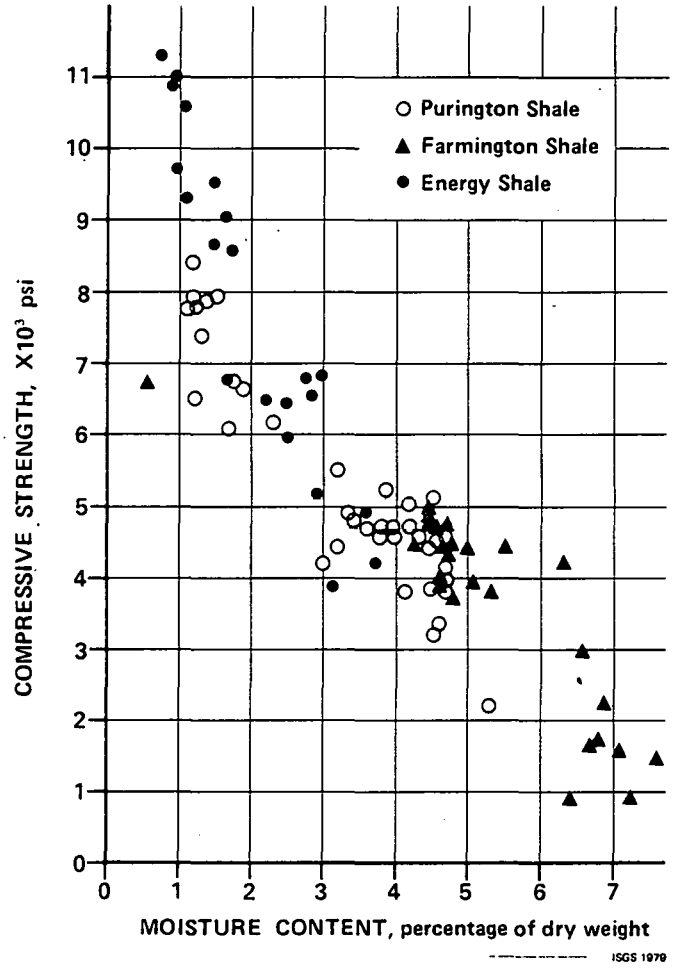


FIGURE 3. Correlation of moisture content with compressive strength of three gray shale members

10 days. Both sets show that 70 percent of the natural moisture was lost by the fourth day at room atmosphere. Figure 2 shows a corresponding increase in the strength of the shale cores. During the first day, the moisture content dropped from 4 to 3 percent while the unconfined compressive strength increased from 4,600 PSI to 5,500 PSI. After only 4 days of drying, the compressive strength of the shale had nearly doubled.

Many authors have found similar increases in strength for other types of rock: Colbach and Wiid (2) for quartzitic shale and sandstone; Broch (1) for quartzdiorite, gabbro, and gneiss; Price (6) for sandstone; Parker (3) for siltstone and sandstone; and Van Eeckhout (4) for shale. All showed that oven-dried rocks were 2 to 3 times stronger than rocks saturated 100 percent.

Our rock mechanics testing of protected cores and cores that had been allowed to dry out to various degrees led to characteristic plots of strength versus moisture content of several shale members. Figure 3 shows the compressive strength versus moisture relationship for three gray shale members. These members are similar in clay mineralogy and lithology, and all are found either immediately above a coal seam or just above a black shale that rests on the coal. A similar relationship of strength versus moisture content is also being developed for black shale lithologies. As of this time, only a few black shale samples have been tested, and Figure 4 shows that they exhibit a trend similar to that of gray shale lithologies. If similar relationships could be shown for other shales, then moisture content determination could be used as a valuable index property.

Besides moisture content determination, there are other strength tests that show similar variations of strength with changing moisture contents. The

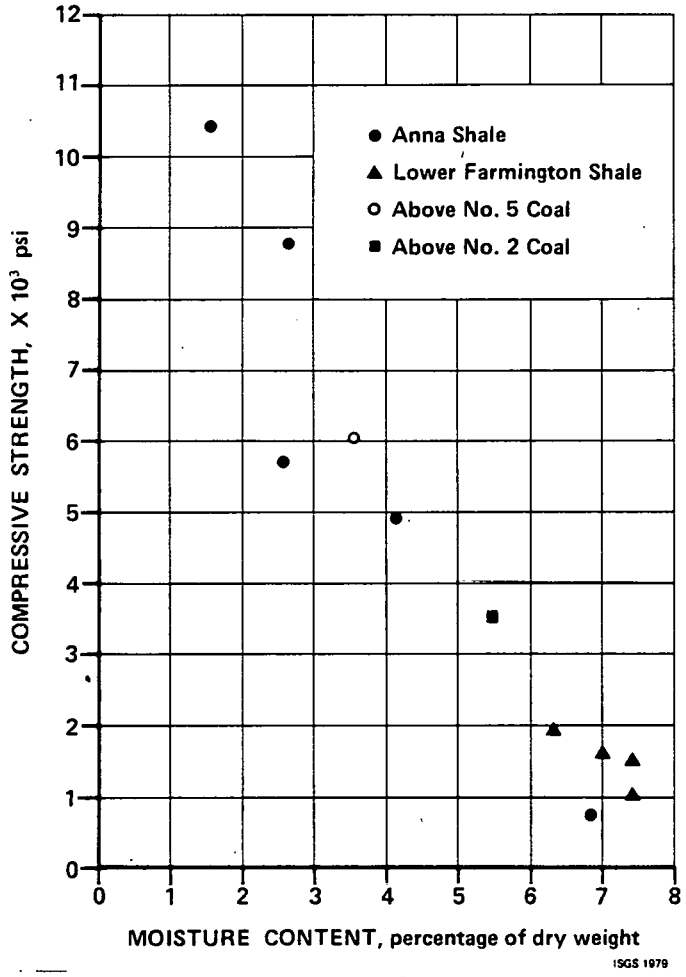


FIGURE 4. Correlation of moisture content with compressive strength of four black shale members

"Brazilian" indirect tensile strength test (Figure 5) and the point load test (Figure 6) both show similar strength relationships with changes in moisture content (Figures 7 and 8). An increase in the tangent modulus that occurred with loss of moisture content (Figure 9) was also found.

These rock mechanics test results give a strength index value for the intact rock samples, not the strength of the rock mass itself. At present these mechanical indices are valuable for comparison of the contrast in strength of the floor, coal, and roof rock units, and the variation in strength within a unit. Great contrast in strength properties from unit to unit or within a single unit may help identify potential zones of failure. Figure 10 illustrates the variation in compressive, indirect tensile, point load, and tangent modulus values of a gray shale unit immediately above the Danville (No. 7) Coal Member. Tests indicate that a sharp drop-off of all values occurs at 8 feet above the coal seam (Figure 10); yet there was

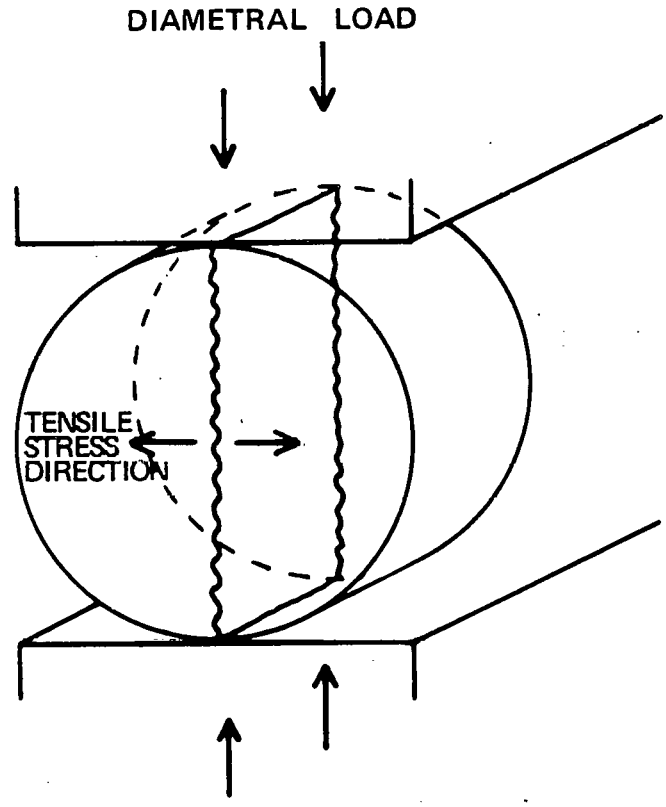


FIGURE 5. "Brazilian" indirect tensile test: load applied diametrically with bedding parallel to plane of paper

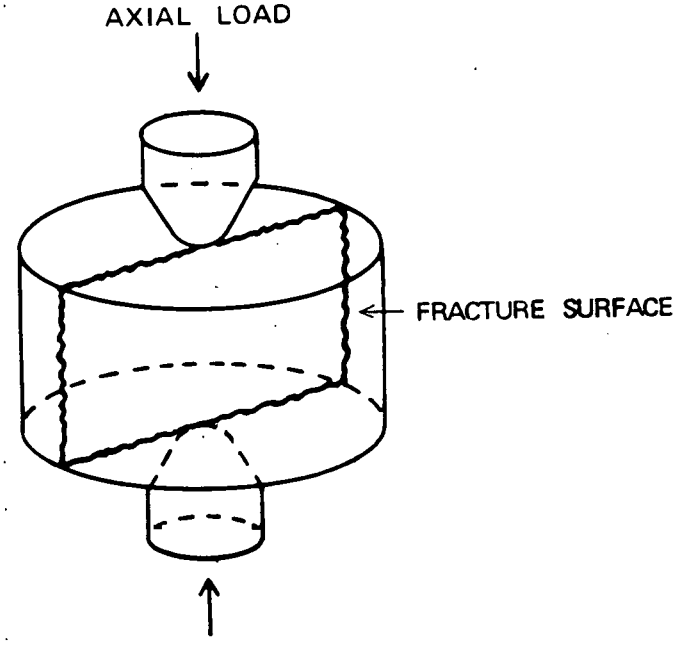


FIGURE 6. A disk of core undergoing a point load test: load applied perpendicular to bedding

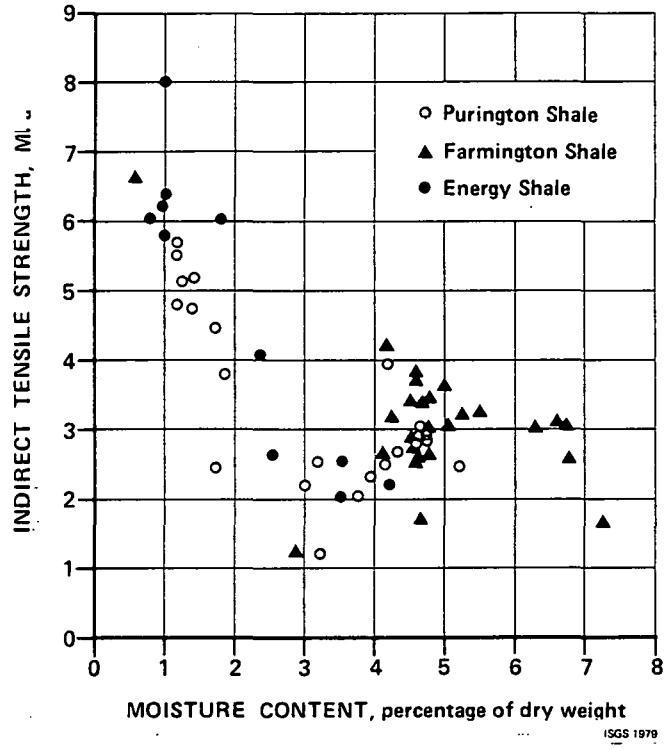


FIGURE 7. Correlation of moisture content with indirect tensile strength of three gray shale members

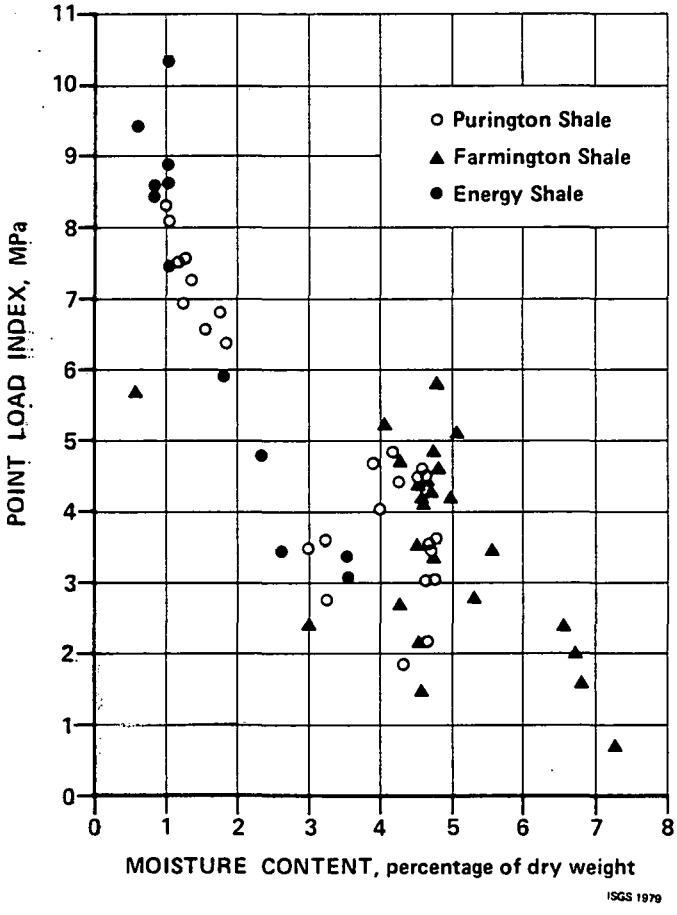


FIGURE 8. Correlation of moisture content with point load index of three gray shale members

no clear visual indication of a change in physical properties when the core was initially logged. The drop in the average compressive strength was from 4,400 PSI above 8 feet to 1,375 PSI below the 8-foot mark.

In Figure 11 the same mechanical results used for Figure 10 are plotted with the moisture content superimposed. This plotting shows that there is a substantial increase in moisture content in the shale below the 8-foot mark. The average moisture content 8 feet above the seam is 4.6 percent; below it is about 7 percent. The strength indices within this single roof unit in the immediate 8 feet of roof rock are, on an average, one third of those of the overlying material (Figures 10 and 11). Thus, a pre-mining evaluation using

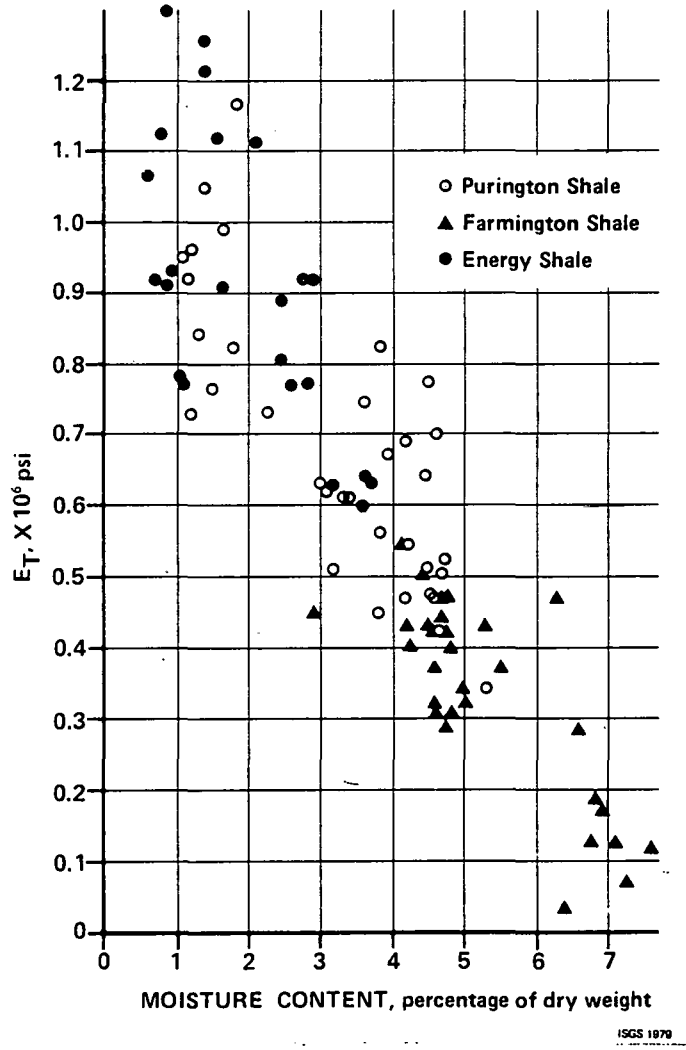


FIGURE 9. Correlation of percent moisture content with tangent modulus of three gray shale members

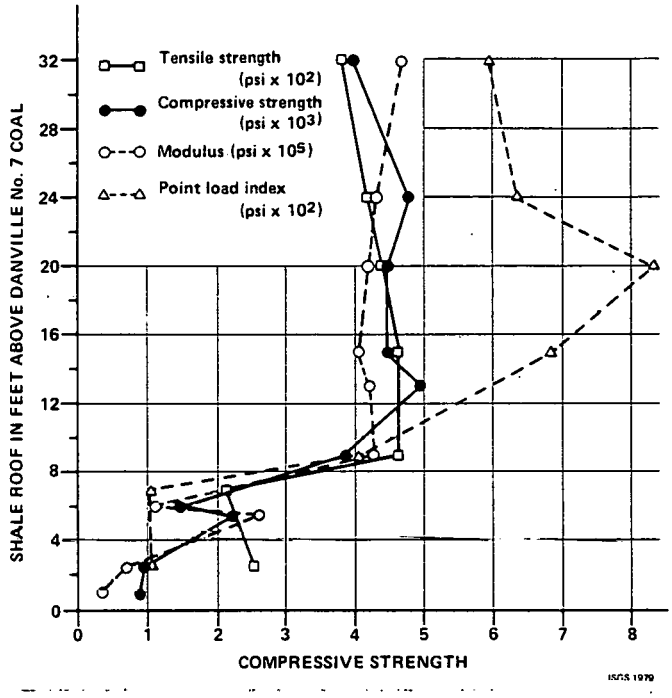


FIGURE 10. Compressive, tensile, modulus, and point load strength values taken at different heights above the Danville (No. 7) Coal Member

these tests would suggest a possible roof support problem, due to a weak immediate roof about 8 feet thick.

The lower 8 feet have a slightly greater content of carbonaceous material. The reasons for the higher moisture content and weakness of the lower, slightly more carbonaceous zone are uncertain and we have begun an investigation into th causes. The variation in clay mineral composition between the two sections is small and is an unlikely cause for the changes in moisture content or shale strength.

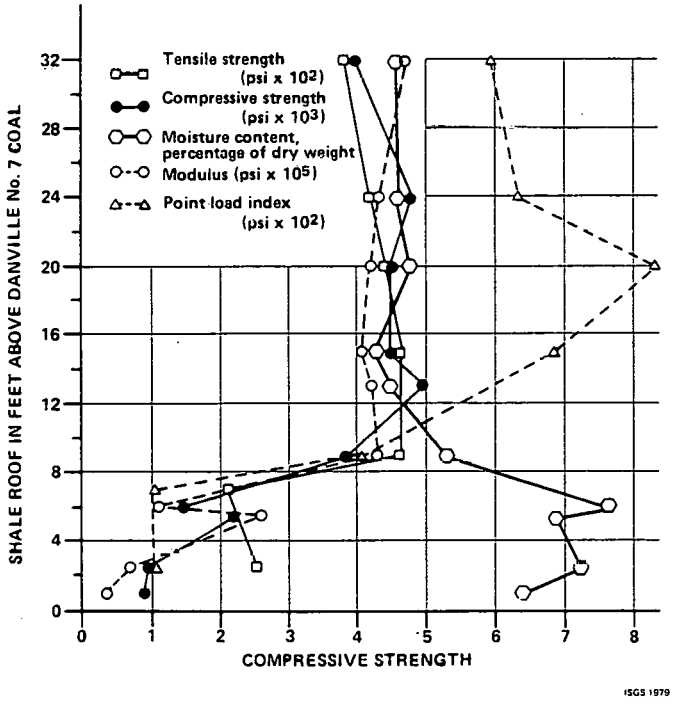


FIGURE 11. Strength values in Figure 10 with moisture content superimposed

Moisture-preserved cores used for these studies have been supplied by several companies interested in obtaining physical strength indices of cores drilled during coal evaluation. The cores used for the study of the rate of moisture loss were about 2 inches in diameter.

## SUMMARY

The goal of our testing program is to develop a data base of the strength characteristics of Pennsylvanian strata in the Illinois Basin to aid in exploration and research. At this time, the data base is still limited. The evaluation of potential stability problems in mine entries is currently conducted by comparison of strength differentials between different units and within single units.

Testing of unprotected shale cores has shown that 70 percent of their natural moisture content can be lost in only a few days; therefore, the strength indices of the shale can easily double. Rock mechanics tests should be conducted on cores with identical moisture contents as those present in the natural state of the rock, since initial mining takes place in that state. Rock mechanics testing of material strength should be accompanied by determination of the moisture content to facilitate comparisons among materials.

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# SESSION III

## ARTIFICIAL SUPPORTS

Session Co-Chairpersons:

Dr. Y. P. Chugh, Department of Mining Engineering
Southern Illinois University at Carbondale

Dr. C. Haas, Rock Mechanics and Explosives Research Center
University of Missouri at Rolla

# INTERACTION OF MECHANICAL ROCK BOLTS WITH SHALE ROOFS IN COAL MINES

by
N. B. Aughenbaugh, Chairman
Department Mining, Petroleum, Geological Engineering
University of Missouri-Rolla
and
Marvin E. Adam
Project Engineer
Quartz Mining Company
Clarington, Ohio

## INTRODUCTION

Roof falls have been responsible for approximately half of all underground coal mine fatalities and injuries in the United States. In an attempt to establish stable roof conditions, federal legislation now requires all coal mines to have supported roofs. Because of economics, ease of installation, unobstruction of the passageways and past usage familiarity, the expansion shell-type, mechanical rock bolt, Figure 1, is the most common roof support method used in Illinois underground coal mines.

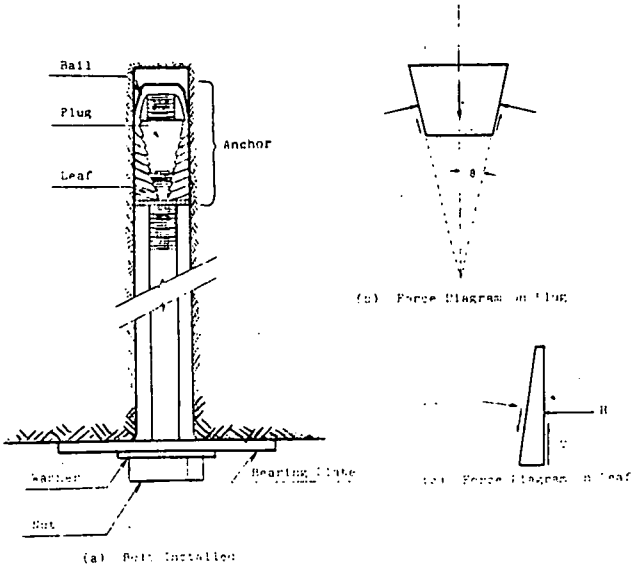


FIGURE 1 Expansion Shell Roof Bolt

The expansion shell bolt utilizes the principle of forcing an expandable serrated shell laterally into the rock of a drilled hole by a screw tightening mechanism. The gripping action of the shell is maintained by the bolt being put into tension.

The types of roof support attributed to mechanical roof bolts are 1) beam building, 2) supension, 3) keying, or 4) combinations of thereof.

The success for all three of these supports offered by expansion bolts depends upon the gripping action of the three inch long shell with the adjacent rock mass. Thus the zone of rock where the expansion shell is anchored has an important role in the function of these type of bolts.

Many underground coal mines in the Illinois Coal Basin have problems maintaining stable roof conditions even though the roof bolts are spaced and installed to approved design standards. Inquiries on bolting procedures have established the roof bolt crews, with very few exceptions, have installed the bolts correctly. The problem of roof instability, therefore, stems not from incorrect bolting methods but rather from conditions

in the roof and the interaction of the bolt and roof material.

This paper discusses the interaction that can take place between the expansive shell-type rock bolt and the roof rock when the roof consists of shale or shale-like materials. The authors found that certain shales, upon exposure to the mine atmosphere, will take on moisture which causes them to expand and generate swelling pressures. Laboratory investigations demonstrated these pressures are sufficient to cause failure of the bolted roof. After establishing that shales in mine roofs can react to moisture changes and that the humidity of the ventilated air within a mine will vary with the seasons of the year, an in situ investigation was designed to monitor roof bolts in an Illinois coal mine (1). Conclusions presented in this paper are based upon data collected in this study, information from correlating laboratory investigations, and observations in other coal mines.

## INFLUENCE OF SURFACE WEATHER ON MINE AIR

Field investigations by Bruzewski and Aughenbaugh (2) have shown weather conditions at the surface are reflected into a mine via the ventilation system. Data from automatic recording weather stations placed on the surface and at various locations within a mine established the moisture conditions of the surface air will change as it is drawn down the intake shaft and is circulated through the mine. The net result is a tempering of the air to the mine conditions due to an interchange of moisture between the air and the exposed mine rock. Depending upon the amount of humidity in the air, moisture will be either absorbed by the rock from the air or transferred into the air from the rock.

The amount of humidity in the surface air varies with the seasons of the year. As the air warms in the spring the humidity increases and remains relatively high throughout the summer. Conversely, the air undergoes a humidity drop in the fall. The winter atmosphere is cold and dry.

By monitoring the intake and exhaust air of several mines, the gross effect of moisture interchanges between the mine rock and air was established. Figure 2, illustrates the absolute humidities at the intake and exhaust air shafts over a two year period for an Illinois coal mine.

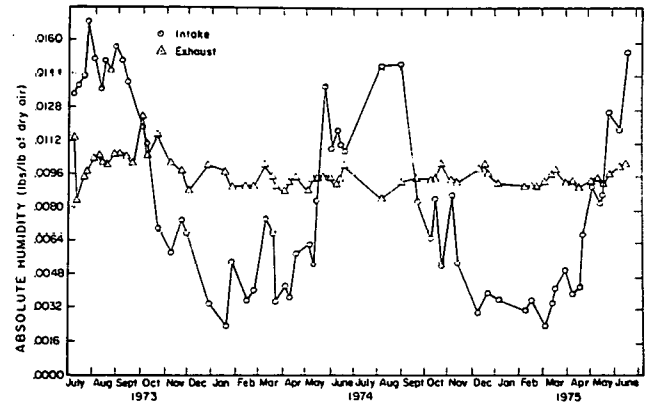


FIGURE 2 Ventilation Intake and Exhaust Humidity Conditions

The graph vividly exhibits the seasonal fluctuations of the humidity of the intake air. In contrast, the exhaust air remains relatively constant.

The area between the two curves is a measure of the moisture interchange between the mine and surface air. When the intake curve is above the exhaust curve, the mine is absorbing water. Conversely, the mine is giving up water when the intake curve lies below the exhaust curve. The overall effect is a cyclic action of moisture transfer into and out of the mine rock. The phenomenon is most intense near the intake air shaft and in the inby air passageways.

## EFFECT OF MOISTURE ON SHALES

Shales and shale-like materials are recognized as rocks that may undergo rapid weathering when exposed to the atmosphere or possibly disintegrate if immersed in water. The phenomenon of how the moisture causes

the alterations is not clearly understood. To attribute the process only to expanding clay minerals is not valid since many shales that rapidly deteriorate, when brought into contact with moisture, contain no expansive clay minerals. This section will briefly discuss some of the known facts of moisture effects on shale.

In an unconfined condition, a shale that absorbs moisture will expand. The volume increase causes a reduction in density, an increase in porosity and a loss of strength, Figure 3.

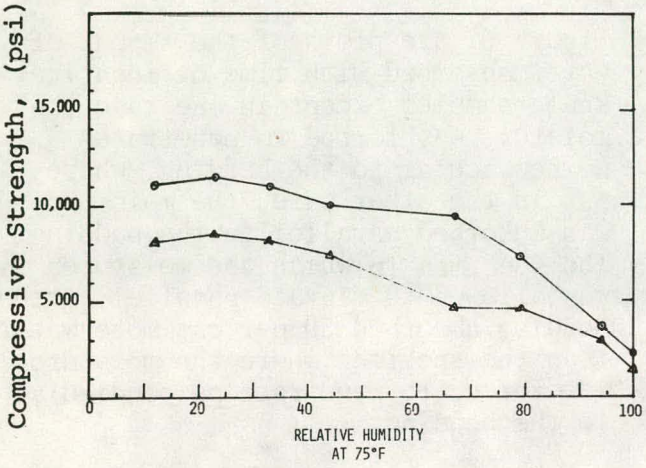


FIGURE 3 Changes in Unconfined Compressive Strength with Relative Humidity For Two Shale Samples From Illinois Coal Mine Roofs.

Shales that expand and disaggregate into a loose mass when immersed in a beaker of water is a good but extreme example of this action. Under confined conditions the absorption of moisture will generate swelling pressures capable of causing rupturing in adjoining rocks and failing the shale itself. In a laboratory experiment, samples of shales from mine roofs of Illinois coal mines were placed in a metal jacket to prevent lateral expansion and immersed in water. For each test the apparatus was placed in a loading machine and back loaded to prevent axial expansion. In every case, the experiment ended when the sample audibly broke. The load recorded by the testing machine represented the swelling pressures generated by the shale sample. Figure 4, is a picture of a failed sample.

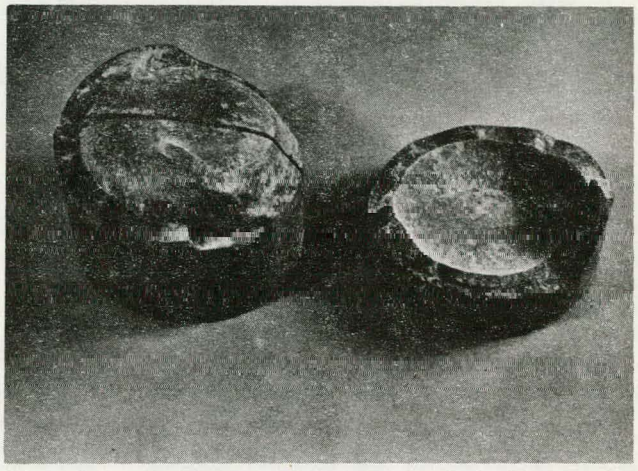

FIGURE 4 Failed Swelling Test Sample of Shale

All the samples tested generated swelling pressures in excess of 4,000 psi. The highest swelling pressures recorded exceeded 14,000 psi.

When a shale gives up water, the dehydration causes a reduction in volume. If a significant amount of volume change takes place, shrinkage cracks can develop. Even if cracking is not evident to the unaided eye, microcracks may occur which will weaken the rock and cause more accentuated deterioration when wetting occurs. Therefore, some shale samples, which are relatively durable to increased moisture conditions alone, will deteriorate under cyclic wetting and drying in laboratory tests. Each cycle will have an accumulative detrimental effect.

The absorption and loss of moisture by a shale is a near-surface phenomenon and is a time-controlled process. When the humidity conditions in the air adjacent to an exposed shale surface change, an interaction begins between the moisture in the air and shale to bring the rock mass into equilibrium with the new environment. The moisture interchange by absorption

(or dehydration) is most intensive immediately after the conditions have changed. The rate of absorption decreases rapidly with time and will essentially equilize in 10 to 20 days. After 5 to 6 days over 80 percent of the total absorption will have taken place. Figure 5, is a plot of the change of weight of shale samples with time in a laboratory test utilizing varying relative humidities under constant temperature conditions.

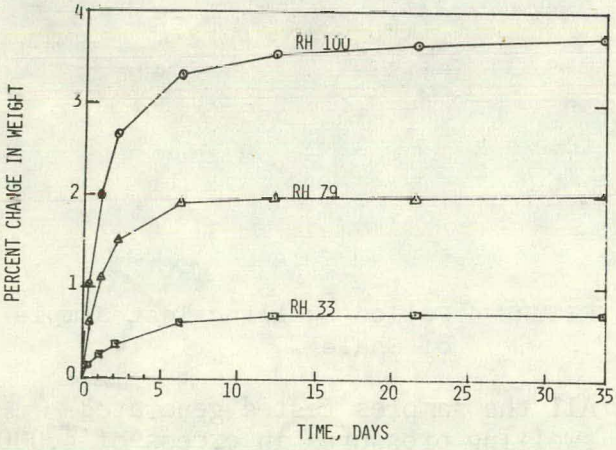


FIGURE 5 Rate of Absorption of Moisture into Shale

Even though the amount of moisture varies with humidity level, all the curves have the same characteristic in that they are very steep the first 24 hours and level off with increasing time.

Various laboratory studies have revealed the depth to which moisture is absorbed by a shale is very shallow if no discontinuities or defects occur. In most cases, it is less than a quarter of an inch. In Figure 4, the distinct narrow band circling the sample near the edge is the depth of penetration by the moisture.

Water however, can migrate rapidly and to great depths along discontinuities such as bedding planes, fractures and holes. The net result is more moisture is absorbed by the rock to greater depths causing a greater volume of the rock to be affected by volume expansion, swelling pressures and strength loss.

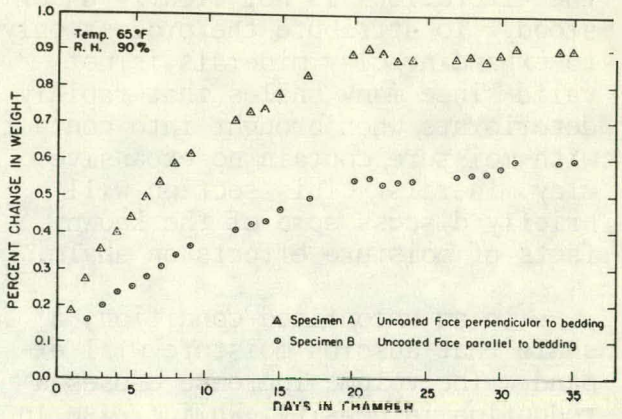


FIGURE 6 Effect of Bedding on Moisture Absorption

Figure 6, are plots of the amount of water absorbed with time of identical shale samples except in one case the moisture was forced to penetrate perpendicular to the bedding, whereas, in the other case, the moisture was absorbed parallel to the bedding. The specimen in which the moisture was allowed to migrate parallel to the bedding absorbed 50 percent more water than the specimen where the moisture was forced to penetrate perpendicular to the bedding.

In another set of tests, a hole was drilled in the sample with the bedding perpendicular to the direction of moisture absorption. This sample took on approximately the same amount of water as the sample with bedding parallel to the direction of moisture migration. These laboratory tests indicated the depth and amount of moisture absorbed into an exposed shale rock face is greatly increased when fractures and holes are present in the rock.

## IN SITU ROOF BOLT STUDY

After establishing that some shales are highly sensitive to moisture changes and that roof bolt holes could be potential avenues of moisture migration into the mine roof, an in situ study was set up to monitor the performance of expansive shell-type roof bolts in an Illinois Coal mine.

Originally, four areas were established to evaluate different roof and ventilation conditions within a mine. Area A, the largest of the four monitoring sites, was along an intake air entry. Data was collected at this site for over a year and a half. Area B was established at an intersection where the intake air was split for transmission to different parts of the mine. Soon after this monitoring site was established, it had to be abandoned because the roof became extremely unstable. Area C was located at the intake of a panel. It was maintained for seven months at which time the roof became unstable. Area D, established within a panel, was monitored for 16 months.

The four monitoring sites consisted of numerous roof bolt deflection stations. Each individual deflection station consisted of the installed expansion shell roof bolt and a steel rod driven into the floor through the underclay to bear upon a competent rock. To prevent movements of any weak floor materials from affecting the measurements, the steel rods were placed within protective pipes which were also founded in competent rock. A schematic of a roof bolt deflection station is illustrated in Figure 7.

A portable extensometer, accurate to 1/1000 of an inch, was used to measure the distance between the top of the floor pin and the head of the roof bolt. Readings were taken weekly and plotted to reflect changes in the roof bolt height above the floor pin.

In addition to the pin measurements, project personnel visiting the site observed and recorded bolt and roof conditions in order to document the overall roof stability for correlation to the measured data.

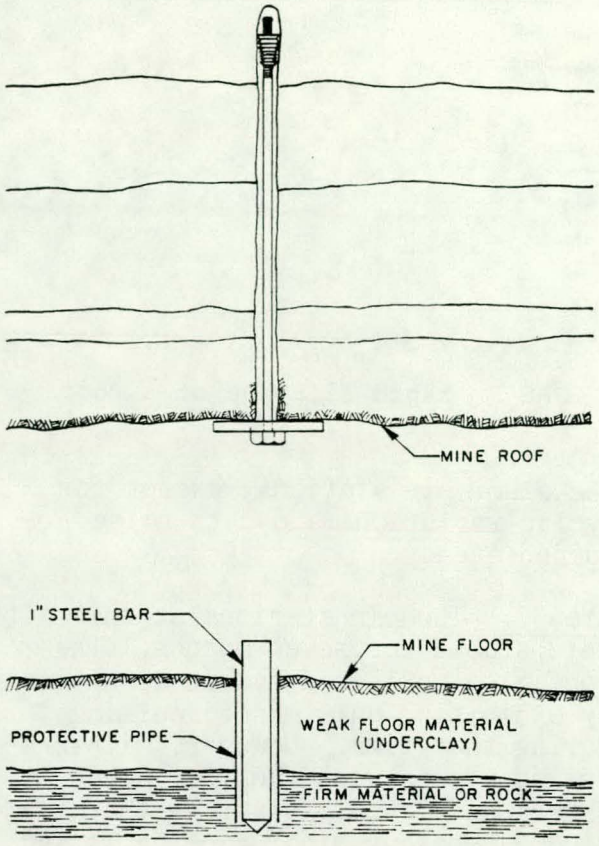


FIGURE 7 Roof Bolt Deflection Station

General Findings At Each Monitor Site

Areas A, C, and D exhibited different roof stability changes although some pin stations at each site yielded similar data. Since Area B had to be abandoned within a few weeks, not enough data was collected to be meaningful with respect to roof bolt movements. The roof of that area was very unstable and had to be re-supported with bracing.

Area A. The roof remained stable throughout the monitoring period. Two significant roof bolt movements were recorded. In January, one pin moved downward two-thirds of an inch in a period of one week and in early May another pin also slipped downward a tenth of an inch. In each case, the two stations held steady before and after the movement took place. Figure 8 illustrates one of the slippages.

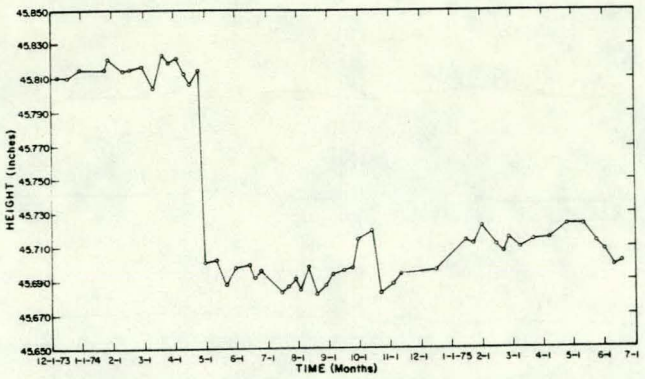


FIGURE 8 Rapid Slippage of a Roof Bolt

The other pin stations, except for cyclic variations and data noise, remained steady.

Area C. The pin stations at this site were stable for seven months. The roof also reflected stable conditions by exhibiting only minor spalling during that time. However, within a period of one week the roof began to experience intense distress that included numerous minor roof falls and dripping water. One pin station registered a half inch drop of the roof bolt. By the following week major roof falls had occurred and conditions became too hazardous for entering the area.

Area D. The roof and pin stations in Area D remained stable for six months. In May and June the roof began experiencing rashing and small roof falls. At this time, two of the pin stations evidenced some downward movement. Roof conditions continued to worsen in July and August. Slabs four to six inches thick fell as masses over 20 to 30 square feet. Rashing was very intense. By early September a number of roof bolt bearing plates lost contact with the roof due to rashing. Two of the pin monitoring stations became dangling roof bolts. The hazardous roof conditions stabilized in September and remained good until early spring. At that time, the roof began to progressively deteriorate again. The area was abandoned in late May. All the roof bolts used as monitoring stations either evidenced slippage or lost contact with the roof due to rashing of the shale around the base plate.

Analysis Of The Data

Examination of the pin station data for all four monitoring areas revealed three different catagories of bolt movements. In one case, the bolt undergoes a sudden slip and restablizes. Under other conditions, the bolt will exhibit slow slippage over many weeks. Lastly, some bolts experienced cyclic lengthening and shortening over a year. All of three of these phenomena are attributed to moisture sensitive shales in the roof generating swelling pressures due to the absorption of water from the air. The slippage and cyclic changes correlate to the seasons. The slippage and bolt lengthening occur in the high humidity seasons of spring and summer, whereas, the shortening or non-movement take place in fall and winter. The only exception was a pin station in Area A registered over half inch of slippage in January.

The rate and magnitude at which a bolt will slip will depend upon the general strength of the shale, how well the expansion shell is embedded in rock and the magnitude of the swelling pressures generated in the moisture sensitive shales. For the cases where rapid slipping takes place, the expansion shell probably is poorly embedded into the adjoining rock. As the roof bolt takes additional tension, a point of failure is reached between the shell and the rock allowing the shell to slip downward. Slow movement over many weeks indicates the material in which the shell is embedded in is weak and will yield when overstressed. Figure 9, is an example of a slow, long term slippage.

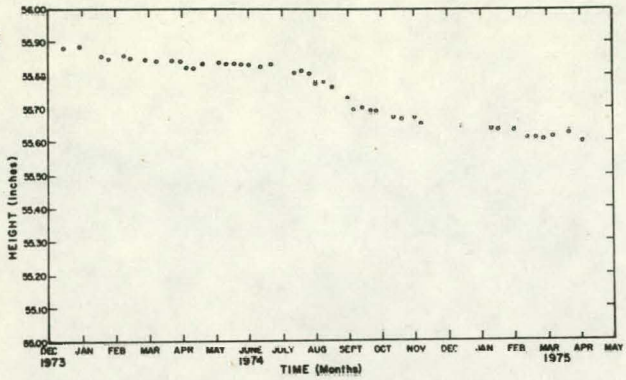


FIGURE 9 Slow Slippage of a Roof Bolt

The cyclic shortening and lengthening recorded at some pin stations, Figure 10, is interpreted as extension and contraction of the bolt itself in response to the development and relaxation of swelling pressures.

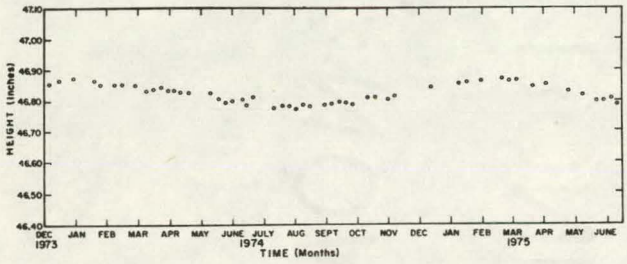


FIGURE 10 Cyclic Movements of a Roof Bolt

In this case, the swelling pressures are not sufficient to fail the frictional bond between the expansion shell and the adjoining roof rock.

The investigation also revealed roof bolts can be rendered non-functional by the exposed roof rock rashing around the bolts. In the first stage the bolt plate may continue to contain some roof material above as illustrated in Figure 11. However, this column of material will slough off leaving the bolt and plate dangling and isolated below the stoping roof.

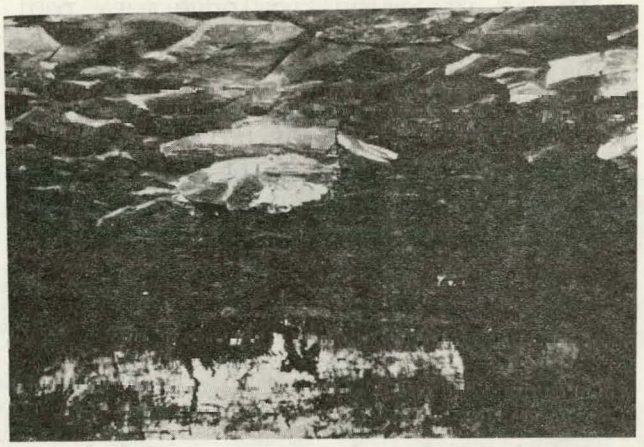

FIGURE 11. Rashing Around a Roof Bolt

The location, thickness and relative sensitivity of the shale strata to moisture will determine the type of instability and deterioration the roof undergoes. If the exposed material is moisture sensitive, rashing and small roof falls will be common. Sensitive shales above the roof will not be affected by moisture migrating from the mine air because the depth of penetration is very shallow. These non-exposed shales will be affected only if fractures or other discontinuities provide conduits for the water to them. Roof bolt holes do provide an access route for humidity in the mine air to penetrate to sensitive shales above the roof.

Beside the pin monitoring study, other lines of evidence indicate moisture is reaching sensitive shales above the exposed roof via roof bolt holes. Some mines not bolted prior to federal law had stable roofs. Upon initiating bolting, the roofs became unstable. At one mine when examining a panel with very bad roof conditions, the senior author viewed an entry and several crosscuts that inadvertly had been left unbolted. These unbolted areas were the only stable roof in the entire panel. Some mines have problems with dropped roof bolts. Examination of these bolts have revealed the threads are stripped or the bolts have necked and broke. Necking indicates tension strain exceeding the strength of steel. Fin-

ally, it is common knowledge many roof falls have a height approximately equal to the length of the roof bolts. One mine reported that as it changed the length of the roof bolts in an area plagued by roof falls, the height of the roof falls also changed and correlated to the new roof bolt lengths.

## CONCLUSIONS

Field and laboratory studies have established humidity changes in the mine air can cause coal mine roof instability in the Illinois Coal Basin if the mines contain shales or shale-like materials that arc sensitive moisture variations. In mines with moisture sensitive shales, the use of the expansive, shell-type roof bolt may compound roof stability problems rather than correcting them. Therefore it is important to evaluate and design roof control not only for stress and adverse geologic features but also for the environmental condition of humidity in the mine air. For those mines or portions of a mine where sensitive shales occur, roof control methods other than the expansive shell roof bolt should be used.

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# USE OF INCLINED ROOF BOLTS IN COAL MINES

Madan M. Singh
Engineers International, Inc.
Downers Grove, Illinois 60515

and

William J. Karwoski
Spokane Research Center
U.S. Bureau of Mines
Spokane, Washington 99207

## INTRODUCTION

Roof bolts are the most common method of roof support used in United States coal mines today. Yet their mechanism of action is improperly understood and most hypotheses advanced in this connection are empirical. These do not provide sufficient information to reliably evaluate new roof-bolting practices or schemes that may be introduced.

The Federal Coal Mine Health and Safety Act of 1969 requires that each mine operator continually improve the roof control system of each mine. It is the responsibility of the District Manager of each Coal Mine Health and Safety District, as organized by the Mining Safety and Health Administration (MSHA) of the U.S. Department of Labor, to approve all roof control plans submitted by mine operators before they can be adopted. Hence, both the mine operator and MSHA need some criteria for guidance in making these decisions.

In view of these information gaps, this investigation was undertaken to evaluate the use of inclined bolts in coal mines, especially those where shearing along the rib occurs. Several examples of these are known to occur in the Illinois coal basin.

The vertical shear stresses in a laminated beam roof are highest at the edge of the coal pillar or rib, and least at midspan. Hence, there is often a tendency for some roof strata to shear vertically along the pillar. It may be theorized that by installing inclined bolts that are anchored over the rib, these shear stresses may be minimized, without significantly reducing the horizontal shear resistance along the various strata or adversely affecting their load carrying capacity.

This method of installing inclined bolts entails placement of a long conventional mechanical bolt in such a fashion so that the collar is a few feet away from the pillar, and the anchor is located over the pillar. Some advantages that appear to result from this configuration are:

- support is not affected by cracks in the roof at the middle of the opening span

- the shear strength of the roof near the pillar edge is enhanced
- better anchorage may be obtained with time as the loads on the pillar increase.

This type of bolt installation has been used in the past, but it appears that no structural studies of the system have been made and the procedure went into disfavor because of practical difficulties associated with installing inclined bolts. Hence a systematic study of the system is merited.

## PREVIOUS STUDIES

The concept of installing inclined roof bolts in coal mines is not new, and the system was used in United States coal mines several years ago (Schmuck, 1957; Moroni, 1975; Harrell, 1975). However, because of practical considerations such as the longer time in installing such bolts and lack of understanding as to how they act, this practice has been discontinued.

Thomas (1962) reviewed rock bolting practices in the United States and compared vertical bolts with angular bolts. He found that "as the cost of drilling angular holes is somewhat higher than that for vertical holes, the latter became the more accepted practice. Angular bolting is now used in places where it is difficult to hold the roof with vertical bolts. The practice, although more costly, is particularly recommended at intersections and other points where additional support is needed."

Gräbsch (1953) presents a strong case for inclined bolts and Figure 1 depicts a case where it is evident that correctly angled bolts in the mine roof tend to minimize strata displacement by invoking both the shear and tension resistance of the roof bolts.

Recently, several publications have referred to the use of inclined roof bolts, indicating that these may be more effective than perpendicular bolts (Hugon and Costes, 1959; Schuermann, 1960; Tincelin and Sinou, 1964; Singh and Chugh, 1968). Gambrell and Haynes (1969) have also reported on laboratory experimental studies, but this is essentially a comparison with roof trusses. More recently, Mallicoat (1978) has discussed the action of roof trusses in mines.

Heuzé and Goodman (1973) conducted some model studies with angled cables for a truss and noted that these supported the roof better than vertical bolts. They suggested that angled bolts should behave in a similar manner. Further, they investigated the contribution of a bolt against shearing along a joint or failure plane, and found this to be significant. It was hypothesized that this was not only due to the shear resistance of the bolt, but also the crushing of the rock adjoining the bolt prior to failure. Van Ham and Tsur-Lavie (1970) conducted a laboratory study of perpendicular and inclined bolts, using photoelastic techniques, and concluded that the reinforcement factor for inclined bolts is

- larger than with perpendicular bolts, for otherwise identical roof conditions
- enhanced with the presence of a soft layer between the roof and the ribs, but decreased for perpendicular bolts
- highest in the clamping zone under idealized conditions, but is nearly equal to that at the midspan of the opening for perpendicular bolts
- not computed accurately from midspan sag measurements, but this can be accomplished for perpendicular bolts.

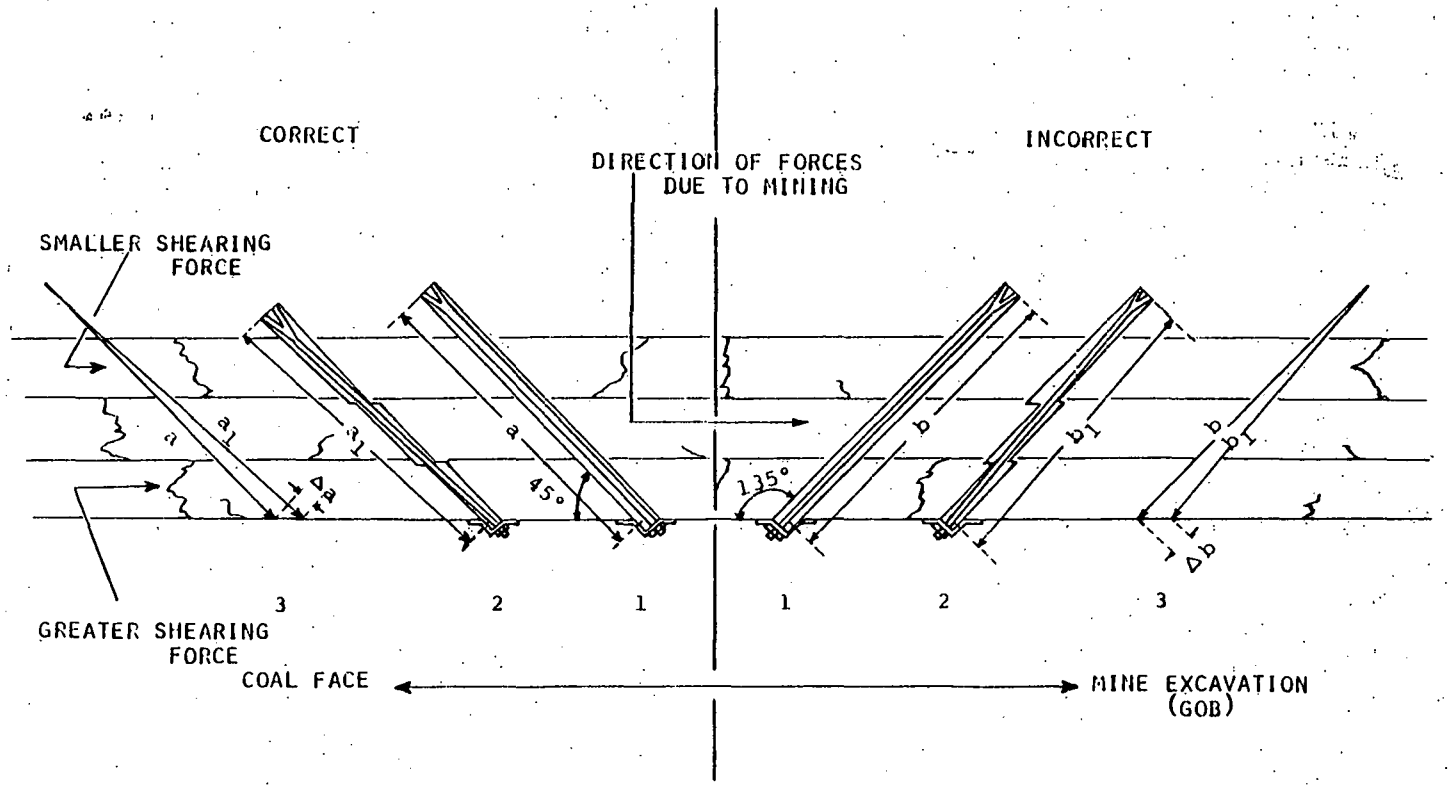


Figure 1. Influence of Layer Displacement on Angle Bolt Action (after Gräbsch)

Further, they noted that inclined bolts

- produce a smaller region of tensile stress above the opening, than for perpendicular bolts
- are effective even if anchor creep occurs.

Haas, Clark and Nitzsche (1974) (also see Haas, 1976) conducted some laboratory shear tests on blocks of rock with bolts and concluded that with low normal pressures (i.e., 25 psi), bolts inclined at 45° to the failure plane provided considerably greater resistance to shearing than those perpendicular to it. At a higher normal pressure (i.e., 250 psi), the effect was similar, but the contribution to shear resistance of the bolt was less in both cases. These effects were observed in both conventional and resin grouted bolts and in both limestone and shale. In light of these findings, it should be pointed out that the normal stress between layers of the immediate roof is equivalent to the overburden pressure before the existence of the underground opening, but once the roadway is driven the various strata start to sag and part, so that the interface pressure decreases considerably and may even become zero. Hence, the results obtained with low normal pressures are more pertinent.

## FIELD INVESTIGATIONS

Field investigations of inclined bolts were conducted in Peabody Coal Company's Camp No. 2 mine, located in Union County, Kentucky.

In one intersection, which served as the control site, 6-ft long, 5/8 in. diameter bolts were installed vertically with ERK shells, at 5-ft centers. Thus there were 4 bolts across each roadway. In an adjoining intersection, the test site, only the center bolts were vertical. The bolts near the rib on either side of the roadway were inclined at 45° to the vertical. These were 9-ft long so as to anchor over the rib at approximately the same horizon as the 6-ft vertical bolts. The location of the bolts and instrumentation are depicted in Figure 2.

The bolts were all torqued to around 150 ft-lb. In order to determine the relationship between the torque applied and the tension in the bolts, 30 Ailtech roof bolt cells were installed on selected bolts. Torque measurements were taken with a conventional, manually-operated torque wrench. Convergence stations were placed along 3 lines, one along the center and two on the sides a few feet from each rib, in both the entries and crosscuts. Eight (8) observation boreholes were drilled into the immediate roof (between 8-ft and 9-ft high, 1-1/4 in. diameter), which were carefully examined with a Lenox Model LICO 6103 borescope. Pull tests were conducted on 38 bolts to determine their anchor capacity. At two locations photoelastic cell clusters were installed, to obtain the stress changes with depth. These did not give meaningful results.

After about 100 days of mining in the instrumented intersections, mine management decided to place some props in these. This eventuality had already been planned for, and 100,000-lb load cells were placed under them. A total of 8 props, 4 in each intersection as shown in Figure 2, were placed.

The bolting at both the control and test intersections was done with a FMC/Galis Model 340 roof bolting machine. The drillhead of this bolter could rotate through an arc of 180°, from left horizontal through vertical to right horizontal. Thus it could drill and install bolts at any angle along this semi-circle. The bolter was designed for use in higher coal and was modified for use at Camp No.2 by Peabody. Besides, it should have been fitted with stabilizer jacks near the front. So its use was awkward, and these deficiencies tended to slow the

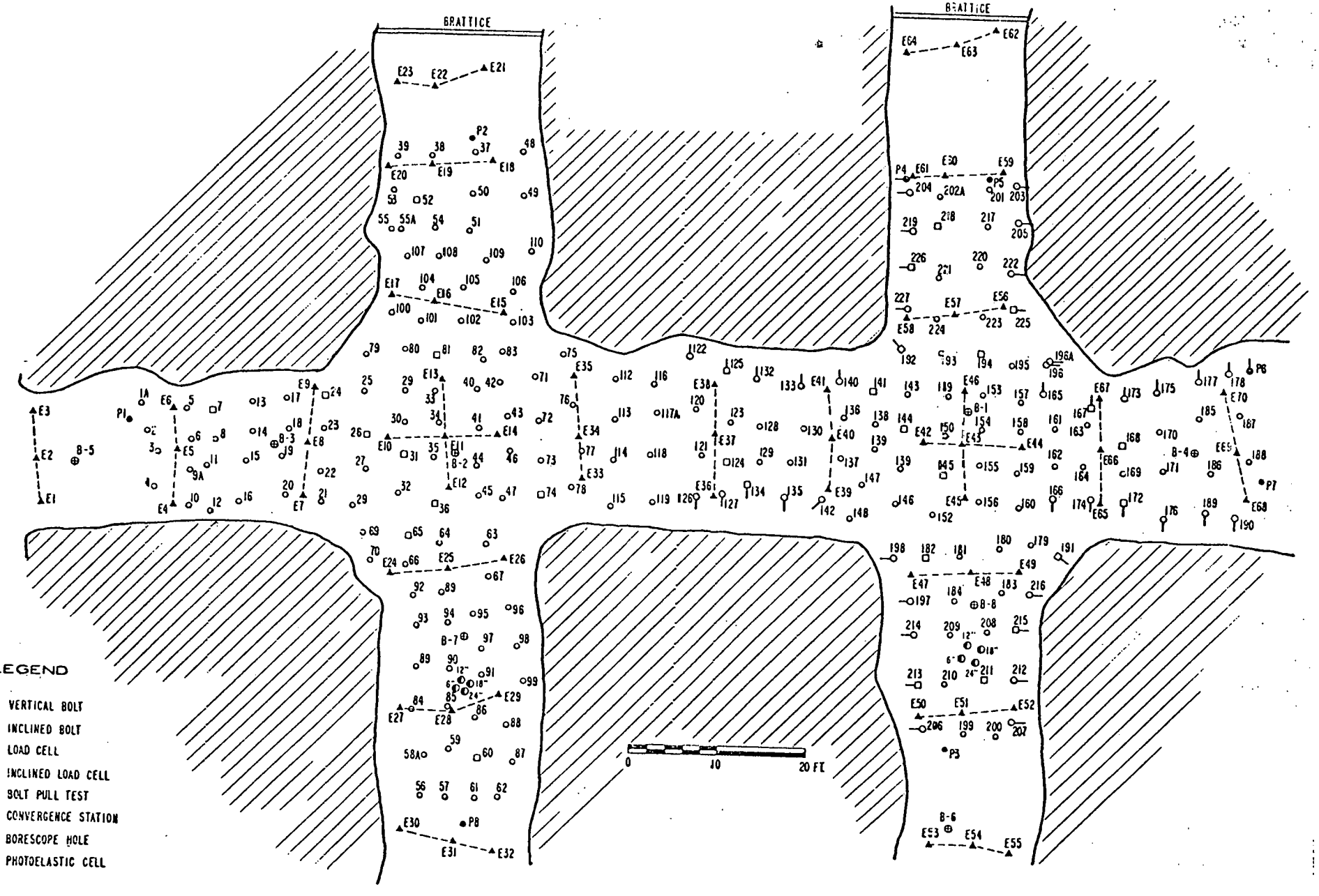


Figure 2. Plan of Control and Test Intersections Showing Location of Instrumentation.

installation of the bolts. The operators soon developed a dislike for the machine. The same machine was used for the control and test sites, however, to minimize the effects of machine characteristics on bolt installation.

Data from the sites were taken for a period of over 4 months. A considerable amount of data were collected and analyzed, including statistical comparisons between the test and control sites. The overall convergence contours are shown in Figure 3.

## Results

Based on the results of the experimental work done in the mine, no clear advantages for the use of inclined bolts were established. No observable differences in roof behavior were noted, and the statistical analyses of the data did not provide any substantive evidence of superiority. However, it was also noted that the inclined bolts did not appear to perform any worse than vertical bolts. The field experiment did not prove any hypotheses concerning the benefits of using inclined roof bolts, but neither did it disprove this. Inclined bolts took longer to install and the equipment used needs to be improved. Now that commercial roof bolters are available that have the capability of installing inclined bolts, many of the difficulties encountered during the project no longer exist. Hence a repetition of the experiment would proceed much more smoothly.

Many of the instruments installed did not provide reliable data. It may be safely stated that torque-tension relationships under field conditions are dubious at best. Since most of the data on bolt conditions were obtained from bolt torque measurements, the variations in the data were such that differences could not be detected with confidence. Direct tension readings on bolts (taken with Ailtech cells) were too few to provide significant results.

In April 1978, one of the authors visited Camp No. 2 mine to see what long range effects had occurred in the areas where the experiment was conducted 2-1/2 years earlier. That section had been worked out and sealed off. Hence it was only possible to go up to the seal for observation of roof conditions. However, discussions were held with mine management on the overall behavior of the experimental area - both in the control and test sites. It was learned that there had been no significant deterioration at either site up to the time of sealing. Both the control and test sites behaved similarly and had remained open throughout the working life of the panel. It could, therefore, be concluded that the inclined bolt site behaved at least as well as the vertical bolt site.

Since the data obtained during this contract could not establish the advantage of using inclined bolts as compared with vertical bolts, no further inclined bolts have been installed in the Peabody mines. However, subsequent to the experiment in Camp No. 2 mine, Peabody has tried trusses. First these were introduced in the Panama mine, but later spread to several mines in northwestern Kentucky - operated by Peabody, Island Creek and other coal companies.

## ANALYTICAL STUDIES

Since the field studies did not yield conclusive results, some simple analyses were performed to provide insight into the effectiveness of inclined bolts installed in ribs. The objective of these investigations was not to prove the concept, but merely to stimulate more comprehensive studies.

### Influence of Angle Bolts on Arch Stability

The redistribution of stress due to the excavation of entries causes

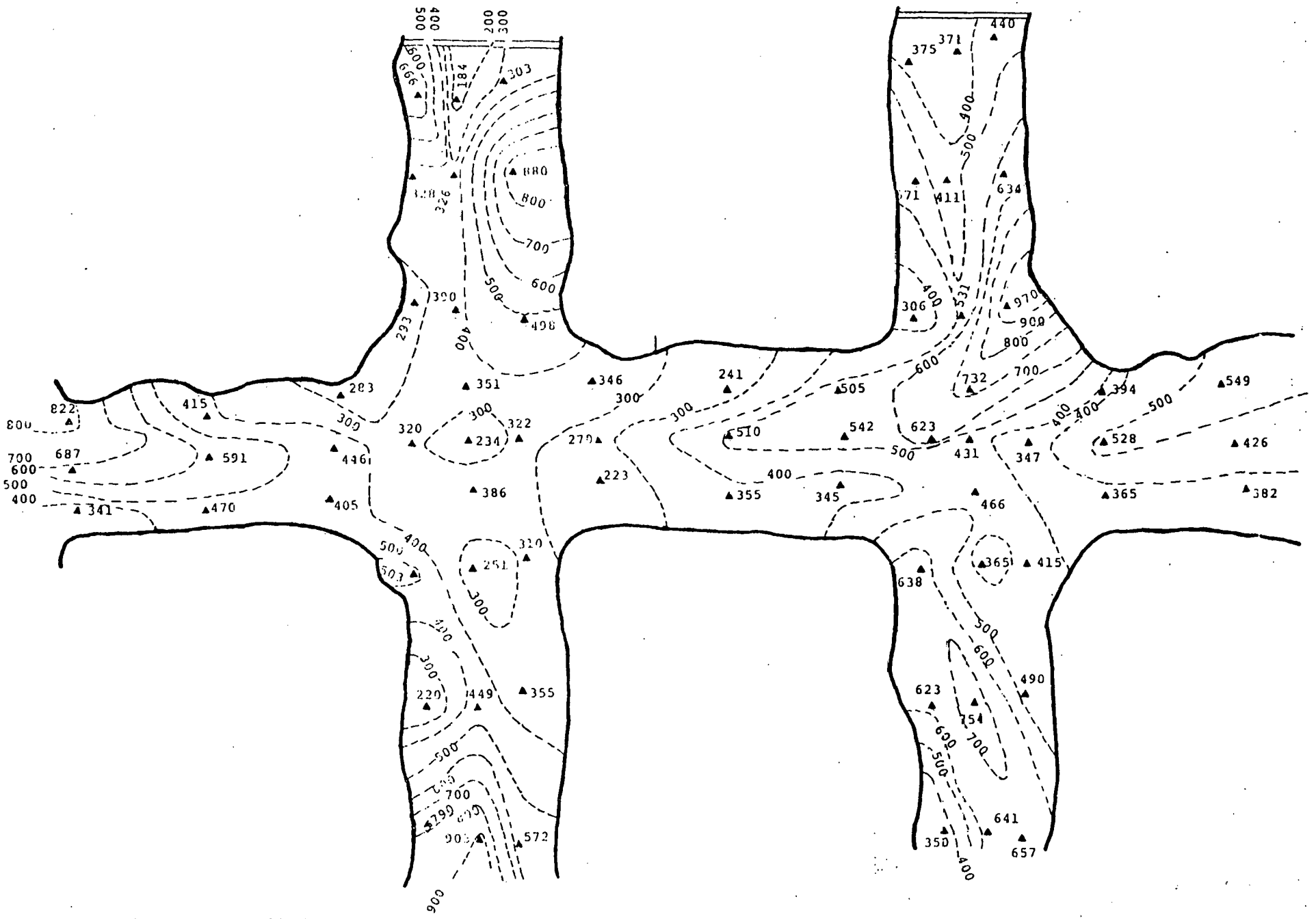


Figure 3. Plan of Control and Test Sites Shcwing Roof Sag Contours.

stress concentrations in the pillars between entries. The maximum stress concentration occurs at the corner formed by the pillar with the roof. This stress redistribution is schematically depicted in Figure 4. It is shown by stress trajectories that the vertical stress, $\sigma_v'$ is concentrated at the pillar-roof corner. At the free surface of the excavation, since the normal stress is zero, the state of stress at the corner consists only of the principal stress, $\sigma_1$ (see Figure 4). This stress state is shown in Figure 5a in a Mohr stress circle.

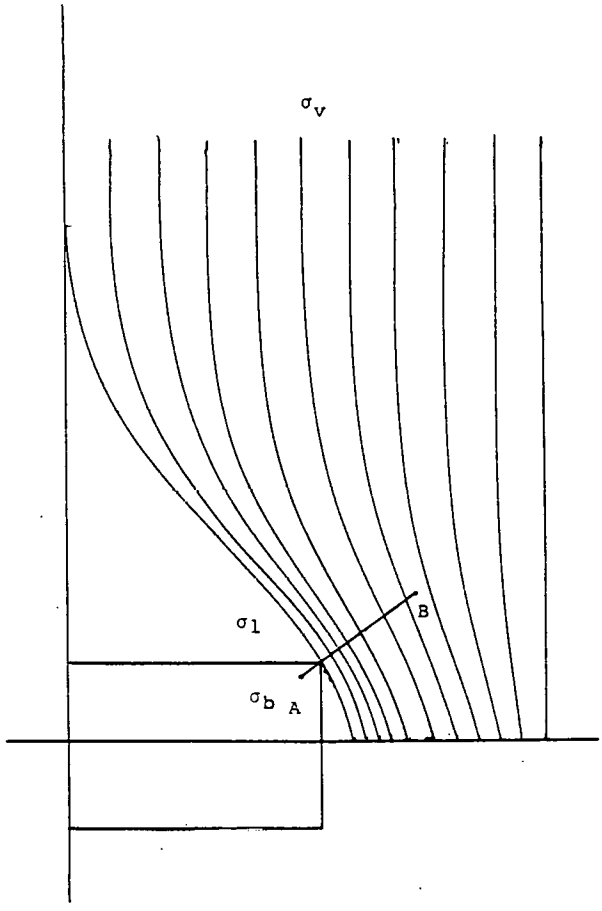


Figure 4. Stress Trajectories Around a Coal Mine Entry.

Assuming that failure of the rock follows the Navier-Coulomb criteria, shear failure occurs at the corner when the stress circle, due to $\sigma_1$, becomes tangent to the Navier-Coulomb failure envelope given by the equation

$$S = c + \sigma_n \tan \phi$$

where,

S = shear stress at failure

c = cohesion

n = normal stress

$\phi$ = angle of internal friction

Resistance to shear failure, however, may be improved if confining pressures are applied normal to the $\sigma_1$ trajectories. This is depicted also in Figure 5b, by the presence of a confining pressure, $\sigma_b$. As a result of applying $\sigma_b$, a larger stress circle is developed, and $\sigma_1$ is increased to $\sigma_1'$ before the Navier-Coulomb envelope is exceeded. This indicates the mechanics for increasing the resistance against shear (arching) failure. The confining stress, $\sigma_b$, can be provided by a tensioned anchor bolt.

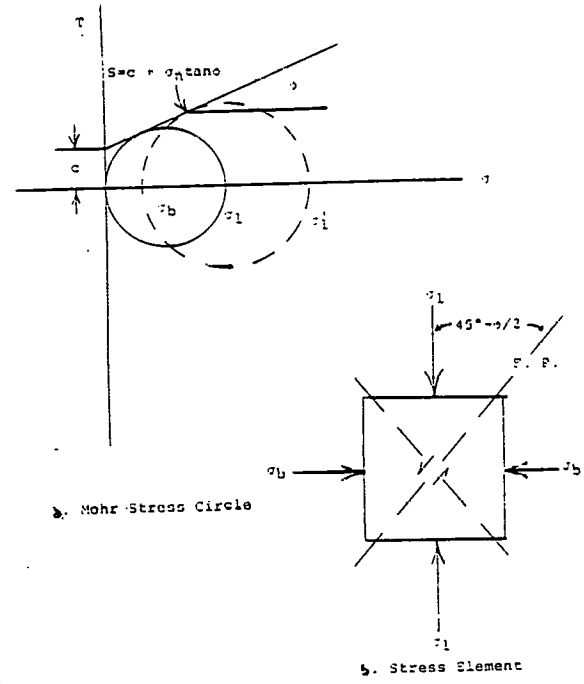


Figure 5. Influence of Continuing Bolt Stress on Increasing Arch Stability.

To gain maximum benefit of the anchor bolt, it must be oriented normal to the $\sigma_1$ trajectories at the point of greatest stress concentration. Schematically, this is shown in Figure 4 where a bolt is placed between points A and B. By tensioning the bolt to a load $P_b$, confining stresses, $\sigma_b$, are developed. The resulting state of stress between the bolt head and the anchor point is approximated by the stress element shown in Figure 5b.

To estimate the effectiveness of inclined bolt placement for improving the shear resistance of the rock, expressions can be developed between the initial stress, $\sigma_1$, and the increased stress, $\sigma_1'$, that results from the bolt. This involves relating the bolt stress, $\sigma_b$, to the bolt load, $P_b$, and expressing $\sigma_b$ in terms of $\sigma_1$.

Consider, first, that inclined anchor bolts are placed into the roof and over the pillar along an entry. Spacing between the bolts is the distance, s. By pretensioning the bolts, opposing forces, $P_b$, are introduced at points A and B in the direction of the bolt. However, between points A and B, these loads are resisted by a complex stress distribution, which is shown schematically in Figure 6.

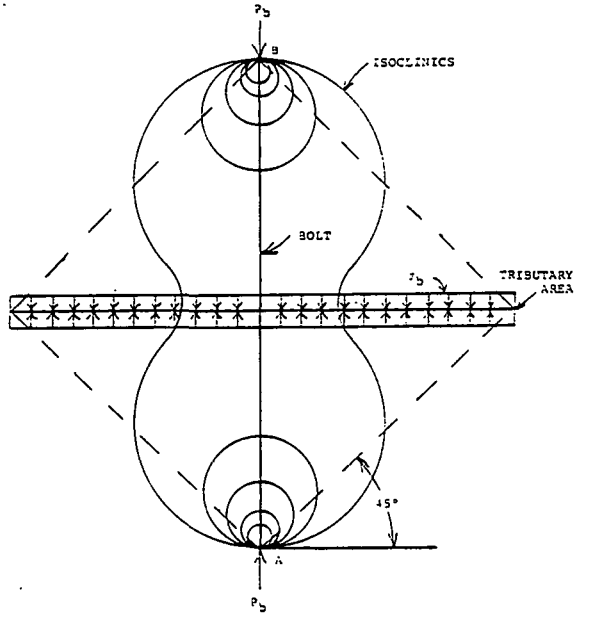


Figure 6. Assumed Stress Distribution Between Bolt Anchor and Collar.

This stress distribution can be approximated by assuming that stresses between points A and B are uniformly distributed along any given horizon in the apexes formed by 90° angles drawn from these points. The $\sigma_b$ stresses, highest at the bolt head and anchor points, decrease to a minimum midway between these points. Hence, at the midpoint between the bolt head and anchor point,

$$\sigma_b = \frac{P_b}{A_e} = P_b/s\ell$$

where

$\ell$ = bolt length

This approximation holds best for closely-spaced bolts, with a spacing, s, of the order of the bolt length. The term $s\ell$ will be referred to as the effective tributary area, $A_e$.

Second, the relationship between $\sigma_b$ and $\sigma_1$ is taken from Dixon and Mahtab (1976) as follows:

$$\sigma_b = \frac{1}{1 + \sin \phi}[\sigma_1(1 - \sin \phi) - 2c \cos \phi]$$

Without anchor bolts, $\sigma_b = 0$ and

$$\sigma_1 = \frac{2\ c \cos \phi}{1 - \sin \phi}$$

With anchor bolts, $\sigma_b = P_b/A_e$, and

$$\sigma_1' = \frac{P_b}{A_e} \frac{1 + \sin \phi}{1 - \sin \phi} + \frac{2c \cos \phi}{1 - \sin \phi}$$

Dividing the equation for $\sigma_1$ without bolts by that for $\sigma_1'$ with bolts, we have

$$\frac{\sigma_1'}{\sigma_1} = \frac{P_b}{2A_e c} \frac{1 + \sin \phi}{\cos \phi} + 1$$

The term, $\frac{\sigma_1'}{\sigma_1}$, may be termed the load improvement factor, and is a measure of the effectiveness of an inclined

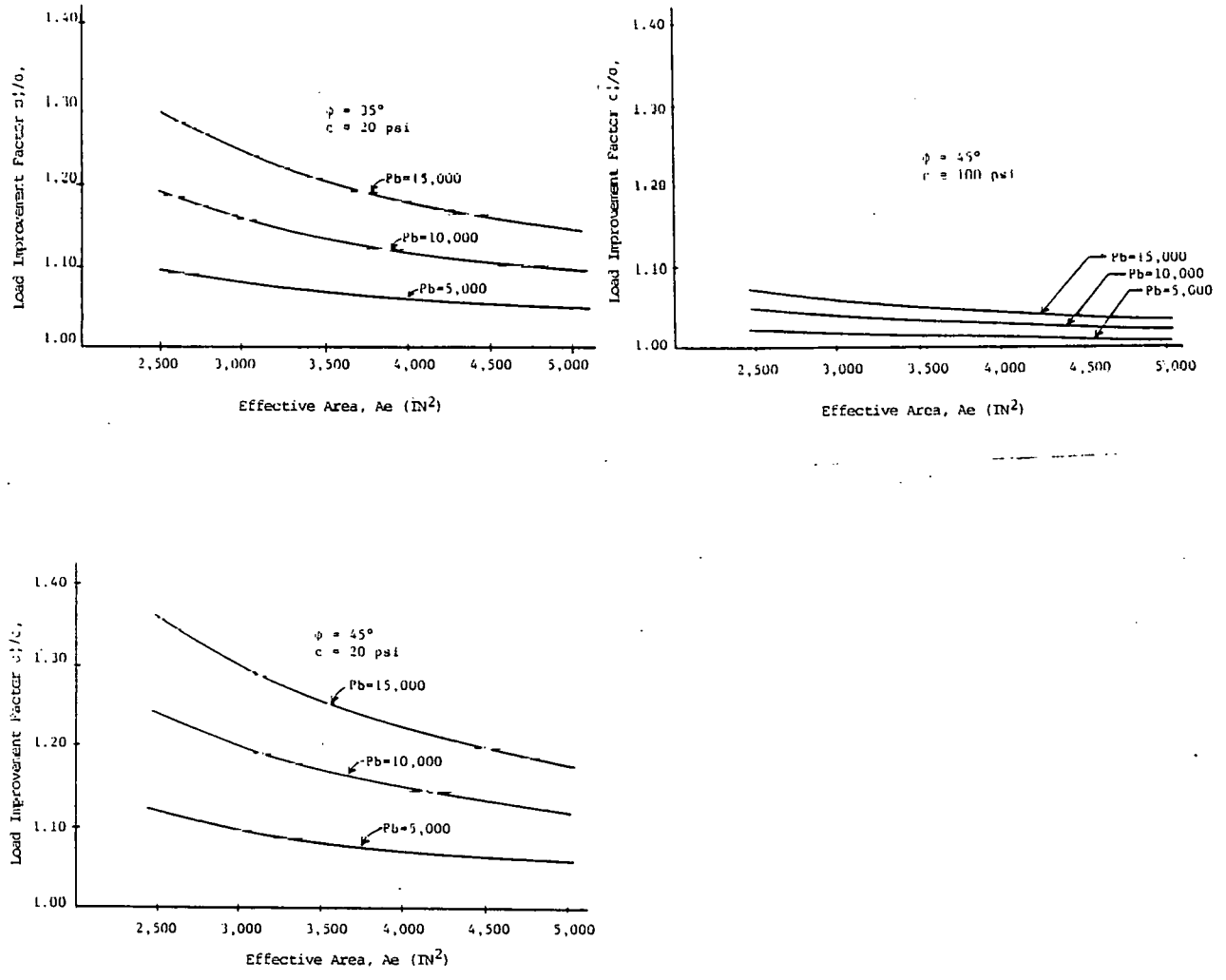


Figure 7. Influence of Angled Rib-Side Bolting Parameters on Bolt Effectiveness.

pretensioned anchor bolt. It is plotted against $A_e$ in Figure 7 to indicate the sensitivity of the parameters $P_b$, $\phi$ and c.

From these plots, the following conclusions are made:

- For high friction, low cohesion rocks, the load improvement factor is greatest and inclined bolting should have the most effect.
- For high cohesion rocks, the load improvement factor is smallest and angled rib-side bolting would have almost no effect.

The foregoing analysis, carried out by making several simplifying assumptions, has indicated that in some situations, anchored rock bolting could significantly contribute to roof control in room and pillar mines.

So far, the presence of a horizontal stress has been ignored. Since the optimum inclination of an inclined bolt would be most beneficial if directed normal to the minimum principal stress trajectory, it is instructive to evaluate qualitatively the change in orientation of stress trajectories near the pillar roof corner due to a compressive horizontal stress field.

Referring back to Figure 4, it is seen that for the case of no horizontal stress field, that the minimum principal stress trajectory is inclined counter clockwise from the vertical at a relatively small angle, say $\theta$. The presence of a horizontal stress field would add a compressive stress component oriented horizontally. By superimposing these two fields, stress trajectories of the combined states of stress can be determined. Since we are not dealing with exact stress distributions, a qualitative analysis based on graphical Mohr-circle construction is sufficient to indicate the orientation of combined stress trajectories. For illustrative purposes, assume that the compressive horizontal stress is smaller in magnitude than the vertical stress field.

In the process of superimposing the two stress fields, the following procedures are followed:

1. The horizontal stress field, $\sigma_h$, is rotated to the original stress field, $\sigma_{v_1}$, to evaluate components of normal and shear stresses in this direction.
2. The stress component $\sigma_{v_1}$, is added to the stress compontnt $\sigma_1$, (see Mohr-stress circle construction, Figure 5), and a new Mohr analysis is performed to find the resultant direction of the stress trajectory $\sigma'_{v_1}$ and its orientation $\theta'$.

The conclusion that results from this analysis is that the introduction of a horizontal stress field causes a counter clockwise rotation of the minor principal stress trajectory, as depicted in Figure 8. The new minor principal stress is shown as $\sigma'_{v_1}$ at

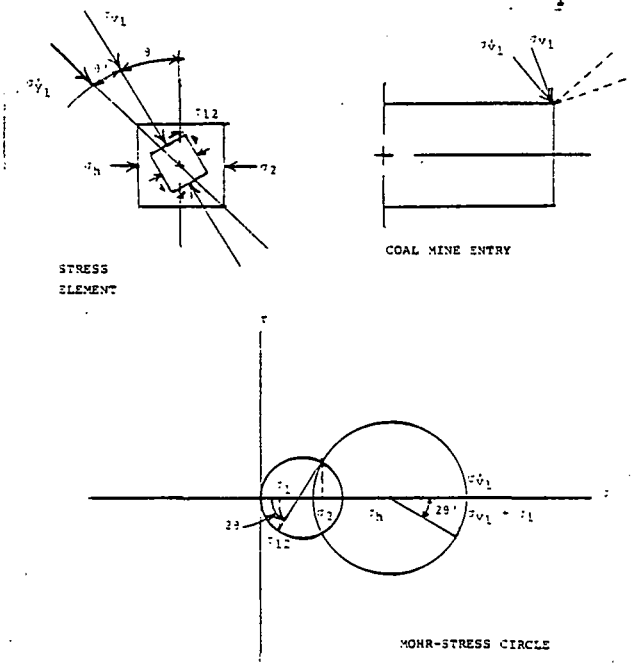


Figure 8. Effect of Lateral Stress Field on Minor Principal Stress Trajectories.

an angle ($\theta' + \theta$) to the vertical. For determining the optimum angle for inclined bolting, the effects of the horizontal stress field should be considered. For correspondingly increasing horizontal stress fields, the roof bolt should be more steeply inclined.

## Influence of Angle Bolts on Beam Suspension

The suspension effect of bolting is that of transferring a portion of the weight of the immediate roof beam to deeper roof strata capable of carrying this load. Thus the internal loading of the immediate roof beam, the moments and the shears, are reduced, thereby decreasing the failure tendencies of this beam. Hence, a measure of the effectiveness of the beam suspension function is the extent to which moments and shears in the immediate roof beam are decreased. Pretensioned anchored bolts are most effective for providing this function. After bolt installation, they are pretensioned to some load, say $P_b'$ dependent on the bolt strength, size and anchorage capacity. When rib-side bolts are angled over the rib, the calculated bolt load does not change, however, the vertical component of this load will be reduced as a function of the angle. Because the angled bolt provides a lower vertical load component than a conventional vertical bolt, the moments and shears in the roof beam are changed. If the change is small or negligible, it could be concluded that angled bolts do not compromise the beam suspension efficiency. If the change is moderate or great, the use of angled rib-side bolts tend to compromise this function.

A comprehensive investigation of this problem was beyond the scope of this study. However, simplified analyses were performed to indicate first order approximate results.

The immediate roof beam is considered as a simple beam with fixed ends (see Figure 9). The span of the roof beam is the distance, $\ell$. The roof beam is suspended by three bolts, one placed at the beam center, the others placed at a distance, e, from the rib-side. Each of the bolts are pretensioned to a load, $P_b$. The rib-side bolts are angled over the rib at an angle, $\alpha$, measured from the vertical. The roof beam is uniformly loaded by its own weight per unit length, w.

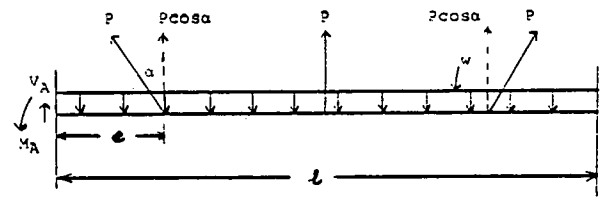


Figure 9. Beam Suspension Model with Roof Bolts.

The analysis of this condition consists of finding the moments and shears throughout the length of the beam. The purpose of this is to find how the bolting parameters, $P_b'$, e, and $\alpha$, influence the moment and shear distribution in the beam. This is accomplished by comparing the maximum moments and shears produced when the rib-side bolts are vertical to the moment and shears when the rib-side bolts are angled. These factors may be identified as the moment-carrying efficiency, M'/M, and the shear-carrying efficiency, V'/V, where

- M is the maximum moment due to vertical rib-side bolts,
- M' is the maximum moment due to angled rib-side bolts,
- V is the maximum shear due to vertical rib-side bolts,

V' is the maximum shear due to angled rib-side bolts.

The analysis is performed, first, by finding the fixed-end moments and shears by superposition of the various load conditions on the beam, and second, by calculating the resultant moment and shear distribution throughout the beam.

A selected range of beam and roof bolt parameters were evaluated. The beam loading, w, was assumed to be 1,000 lb/ft. Three pretension loads, $P_b$, were taken to be 5,000, 10,000 and 15,000 lb. Rib-side bolts were angled at 0° to 60° from the vertical, and were located 1, 2, and 3 ft. from the rib. The width of the entry was taken as 12 ft.

The results are presented in Figure 10. The moment and shear suspension efficiencies are plotted versus the rib-side bolt angles. The influence of bolt tension on the beam suspension efficiencies is shown in Figure 10a and the influence of the rib distance from rib-side bolts is shown in Figure 10b.

These plots indicate that as the bolt angle is increased from the vertical, the beam suspension efficiencies are decreased. For small angles (<15°) these efficiencies are not affected significantly, but when the angle becomes large (say 60°), the beam efficiencies are reduced markedly. It is also evident from these graphs that the greater the bolt tension, the higher the efficiency of the bolts, and the less it is affected by bolt angle.

It should be noted, however, that if the beam weight, w, is increased somewhat, to say 1,500 lb/ft, failure occurs in shear immediately, even with vertical bolts. In practice, of ourse, the beams are generally hicker, but do not load the bolt system as such, because of friction, cohesion of strata and rock strength, tendency to form an arch, and other factors. This emphasizes the simplicty of the current approach. Further detailed analysis of this problem is desirable.

Conclusions that may be drawn from the foregoing studies are:

1. If the primary function of roof bolting is beam suspension;
   a) use of low-angled bolting (<15°) would not significantly influence this function,
   b) use of high-angled bolting (>30°) would significantly influence this function.
2. If the primary function of roof bolting is to prevent arching roof failures, roof suspension functions would not be overly compromised unless the bolting angle exceeded 45°.

The results brought out by these simplified analyses are sufficiently encouraging to warrant further investigations that would more completely evaluate the influence of angled rib-side bolts.

## CLOSURE

Inclined bolts are used for roof control in various mines abroad (e.g., in France) and in hard rock mines in this country. They were commonly used in the United States coal mines at one time. It appears that the only reason they went out of favor here was because of the longer time it took to install them thereby increasing costs. No technical deficiencies contributing to their disuse have been cited, to the best of the authors' knowledge. Hence, the utility of inclined bolts, especially under certain conditions, must not be

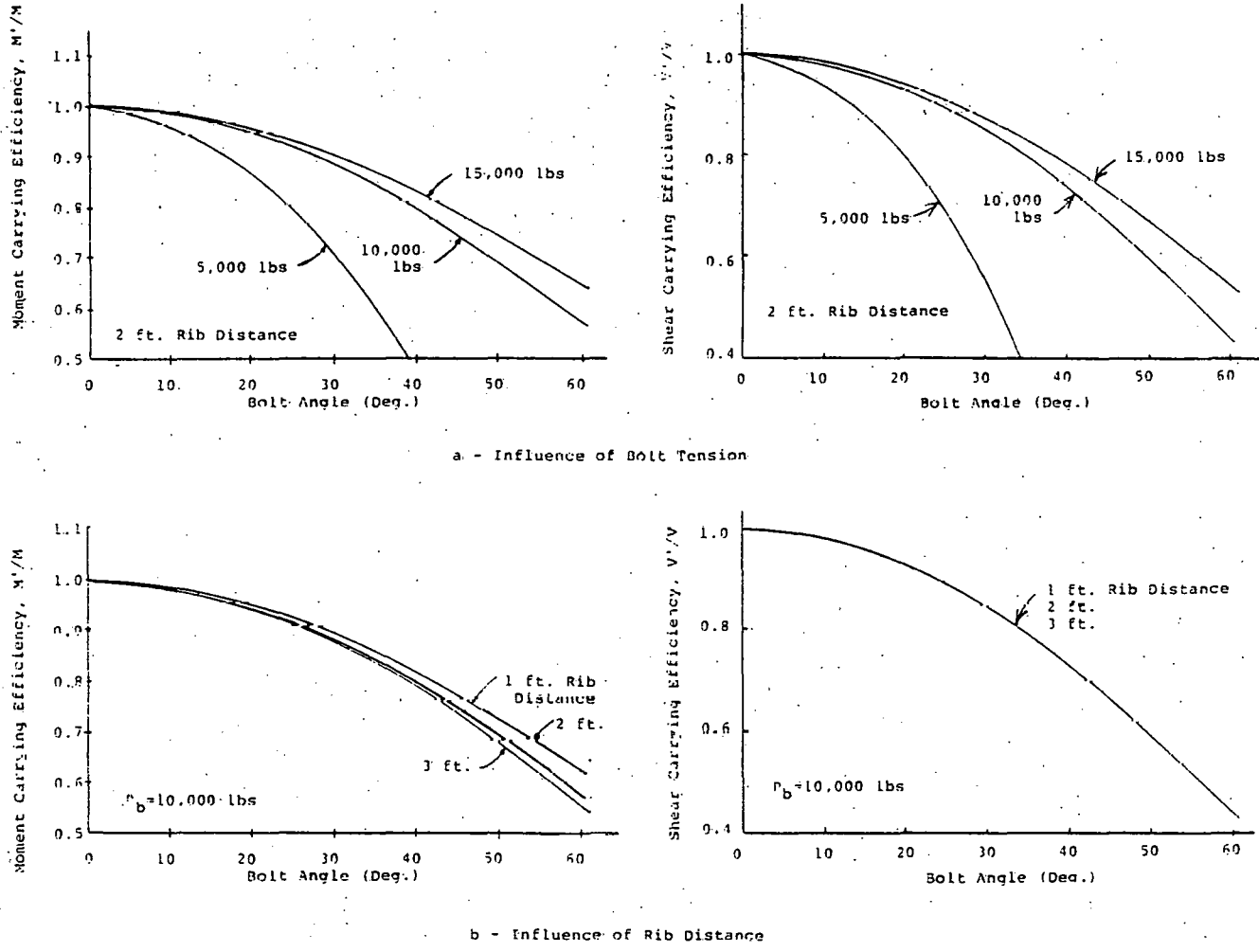


Figure 10. Influence of Angled Rib-Side Bolts on Beam Suspension Efficiency.

discounted too lightly. Further investigations, both theoretical and in the laboratory and field appear to be warranted. These may well help the industry solve another one of its roof control problems.

## ACKNOWLEDGEMENTS

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# RECENT PROGRESS IN ROOF TRUSS TECHNOLOGY

C. P. Mangelsdorf

University of Pittsburgh*
Pittsburgh, Pennsylvania

## HISTORY

The device which White introduced in 1967 and patented in 1969 (1) was intended to reinforce a main entry roof in the same way that a queen's post truss reinforces a beam. His original design, using two mechanically anchored bolts angled at 45 degrees up over the ribs and connected by a horizontal chord consisting of a combination of flat straps, threaded rods and a turnbuckle, was soon superceeded (2) by a system based entirely on bolt stock, Figure 1. In this later and now standard system the inclined bolts extend below the roof line and are bent over the collars of their holes to form part of the horizontal chord. In addition to the turnbuckle which is required for initial tensioning of the truss, a "wedge box" gives some adjustability to the standard lengths of the rod components in order to compensate for variations in hole lengths and locations. The bearing blocks shown (dimpled steel plates are now used) separate the horizontal chord from the roof to facilitate tightening.

The truss, or truss bolt as it is sometimes called, was hailed as a major breakthrough in supplemental support and was widely tried throughout the coal

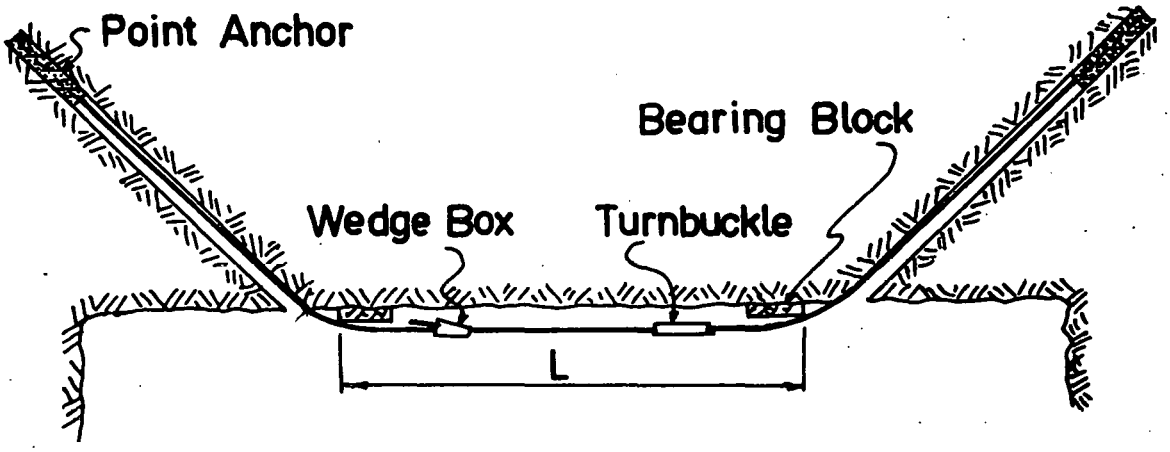


Figure 1

*The author gratefully acknowledges the support of the Bureau of Mines, Grant No. G0166088, in the work herein credited to him.

mining industry with mixed success (3, 4,5,6). The primary difficulty seems to have been the inadequate anchorage supplied by mechanical anchors. When fully grouted resin bolts became available, trusses were generally abandoned because of their higher costs.

In the # 9 seam of western Kentucky, however, full column bolts were not the answer. Failures continued to occur above the upper ends of the bolts. In 1976 McDowell, Greenfield and Hawkins (7) developed the resin point anchor system, a partial resin anchorage 18 to 24 inches long. With the aid of the point anchor, the truss concept has enjoyed singular success in supporting roofs which only timbering had previously held. At the present time, White's firm is producing 13,000 sets monthly of which over half are used in the Illinois coal basin.

In the historical context one must remember that trusses have been and continue to be regarded by MSHA as "supplemental" support rather than primary. For all practical purposes bolts are required everywhere. If bolts are insufficient, trusses may be added. It may well be, however, that the trusses are the primary support.

## INSTALLATION

Part of the cost of a truss is the time required for installation. The most notable success in reducing this time has been achieved by Mallicoat (8) using a Fletcher model DDJ dual boom roof bolter in cycle behind a continuous miner. The machine advances into the cut installing vertical bolts according to the approved roof plan. As it retreats, the drill heads are tilted and used to install the inclined chords of the trusses. Final assembly and tightening to about 10,000 pounds is accomplished by hand. Mallicoat reports that a typical shift can install 100 bolts and 20 truss sets at five foot spacings.

## BEHAVIOR

The use of the queen's post truss concept was first proposed in Germany by Middendorf and Jacobi (9,10) in 1952. The heads of angle bolts over the ribs were connected by a non-tensioned cable. Analysis suggested that the upward and inward thrusts supplied once the roof deflected might restrain further roof motion. In field and laboratory tests, however, the large lateral deformations at the anchors, and the enormous vertical convergence experienced in the deep European mines, negated whatever advantage trusses might have.

A more recent experimental approach to the beam reinforcement concept has been conducted by Haycocks and his associates (11) in attempting to optimize truss support capacity. For the construction of two-dimensional photoelastic models, the overlying strata were simulated by strips of varying thickness but of the same material, a urethane rubber of low elastic modulus. The strips were carefully laid down over a temporary support positioned in the opening. When all layers were in place without initial interlayer shear stress, the temporary support was removed and the roof was permitted to deflect under gravity. A model of a truss was then attached and tightened with a calibrated spring, while deflection and fringe orders at midspan were observed.

The conclusion of the study was that somewhere between 45 and 60 degrees from the horizontal was the optimum angle for the inclined chord installation. Furthermore, more effective support could be achieved by moving the holes closer toward one another, but only at the expense of longer holes, if anchorage over the rib were to be maintained. The uthors recognized that roof laminations are neither uniform nor completely elastic, and that once installed the trusses (and the roof) do not remain at a constant state of stress. The results were intended to provide a guide to the relevant parameters, not conclusive design criteria.

Two alternatives to the beam concept have been proposed by Cox and White (12) in 1977. One of these considers the roof material below the pressure arch as pure rubble suspended by the truss. (In terms of composition, the roof over the Kentucky #9 seam would appear to fit this description.) The other concept is that the existence of the truss leads to the formation of a low, reinforced rock arch capable of supporting the material between it and the natural pressure arch. In-as-much-as Cox will be describing these ideas elsewhere in these proceedings, nothing more need be said here.

The author's own work on trusses has concentrated thus far on the behavior of the truss itself, and the role that friction plays in that behavior (13). To appreciate this role consider what happens during the tightening process as the turnbuckle increases the tension in the horizontal chord, Figure 1. No increase in tension is experienced by the inclined chords until their lower extremities begin to move away from the anchored ends. To help visualize these motions Figure 2 shows the detail of the truss in the region of the bearing block.

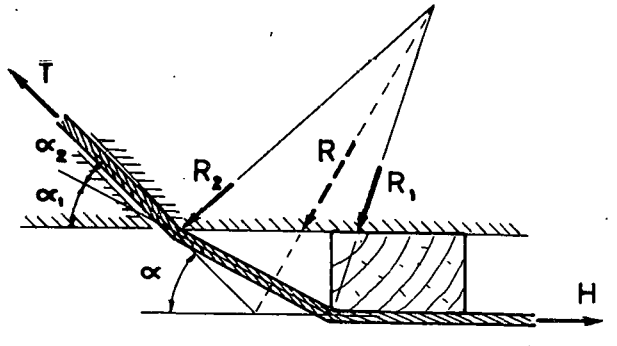


Figure 2

While Figure 2 is not a wholly true or complete picture of what happens at the junction of the chords of a truss, it is sufficiently accurate to allow one to see the nature of the motions required. In order for the tension in the inclined chord, T, to increase during the tightening process the point on the truss where the reaction $R_2$ is shown must have a component of motion along the axis of the inclined chord and away from the anchored end. This is clearly possible if the collar of the hole moves due to some overall roof motion or if some small amount of rod slips out of the hole.

If the increase in inclined chord tension, T, is to be related to the increase in the horizontal chord tension, H, some kind of motion must also be taking place where the rod bends over the bearing block. This can be accomplished if the rod slips over the block, if the block slips over the roof, if the block begins to overturn or if the block crushes.

Without going into detail here (see Reference (13)), it can be shown that only slipping is capable of producing the amount of motion necessary to produce the increase in T required. Because there must be friction, it can then be stated that during the tightening process the ratio T/H will have a minimum value less than one, corresponding to impending slip. Once the slipping occurs, T increases with a corresponding decrease in H. This concept of a lower bound of T/H during tightening is depicted in Figure 3 where the slope of the bound is less than 45 degrees.

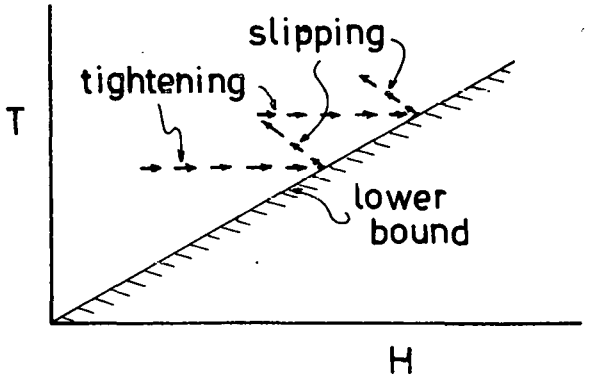


Figure 3

Once the truss is fully tightened, if nothing changes, the ratio T/H will remain constant, but experience indicates that there are many opportunities for change. One change to consider is the working of the roof such that more load comes down on the truss. This will be reflected immediately by an increase in T but without much change in H until such time as slipping occurs in the direction opposite to that associated with initial tensioning. Such slipping implies the existence of an upper bound to T/H, at a slope of greater than 45 degrees.

Figure 4 shows both the upper and lower bounds indicated by U and L. Tightening is depicted by the path 0-1 Roof working, as described above, follows the path 1-2-3. A typical change frequently observed in trusses is a general relaxation due to a variety of small inelastic deformations occurring at points of stress concentrations. This is shown by the path 1-4-5, which was observed in several trusses over a two week period after installation.

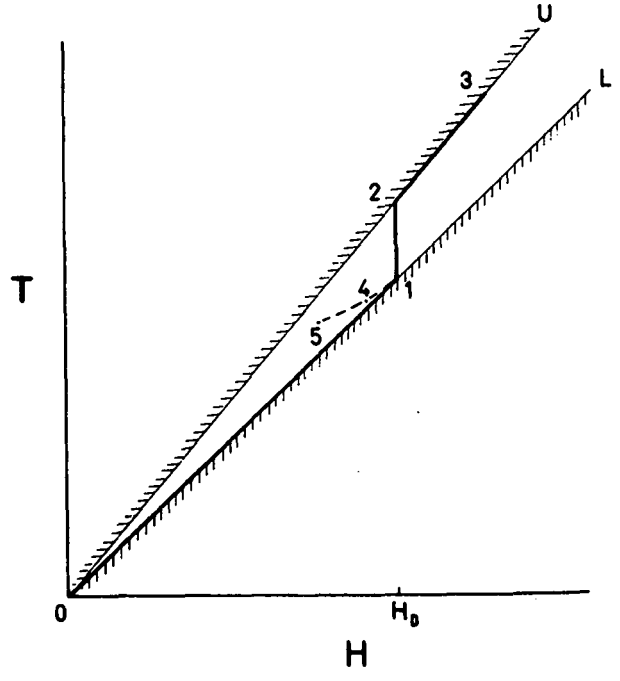


Figure 4

The important point here is that the relative chord tension can, for whatever reason, be anywhere within the bounds. Once they reach one of these bounds, however, and if the causes of change continue, the truss must begin to follow the bounds. The implication of this fact is, that if the chord tensions can be monitored they will give an account of what the roof is doing.

Another advantage in knowing the chord tensions is that the forces applied to the roof by the truss can be found. In Figure 2 the equal and opposite reactions of the forces on the truss applied by the roof are shown. The summation of forces in the vertical and horizontal directions permit one to calculate the magnitude and direction of the resultant R. The line of action of R may be shifted slightly from the intersection of T and H, due to the possible existance of small bending moments (not shown in Figure 2) but it's magnitude and direction will not be influenced by these moments. As the ratio T/H varies, however, the direction of R changes. During tightening, if T/H = 0.9, for example, R makes an angle of about 60 degrees with the horizontal. For T/H = 1.3, this angle becomes about 85 degrees.

Finally, by knowing all three chord tensions, one can predict the effective stiffness or resistance which the truss offers to the roof. For example, if the truss is at point 1 in Figure 4 and the roof begins to come down, the resistance to motion (in units of force per unit of displacement) provided by the truss is that supplied by the inclined chord alone. On the other hand if the truss is at points 2 or 3 the truss stiffness is that of a length of rod that includes part or all of the horizontal chord as well as the inclined chord, a less stiff condition. It should be noted that these differences are not related directly to the force required to stretch a given length of rod. Due to the curvature of the rod and the crushing of material at the bends, the stiffness of the truss is non-linear and considerably less than for the rod alone.

The bounds depicted in Figure 4 are probably not straight (because friction is not constant) nor do we know the magnitudes of typical bounds. Observations on a few trusses underground suggest that the ratio T/H varies from about 0.9 to 1.3 but many more trusses need to be observed before any reasonable predictions of performance can be made.

## MONITORING

The most direct method of measuring chord tensions is by means of electrical strain gages attached to the chords. The difficulty with the method is that it is expensive and time consuming and only a few trusses can be observed. To circumvent this difficulty the author (14) has developed a portable, battery operated instrument which measures the natural frequency of vibration of the horizontal chord. From a calibrated formula it is then possible to calculate the chord tension.

Figure 5 shows the results from one of several trusses monitored underground

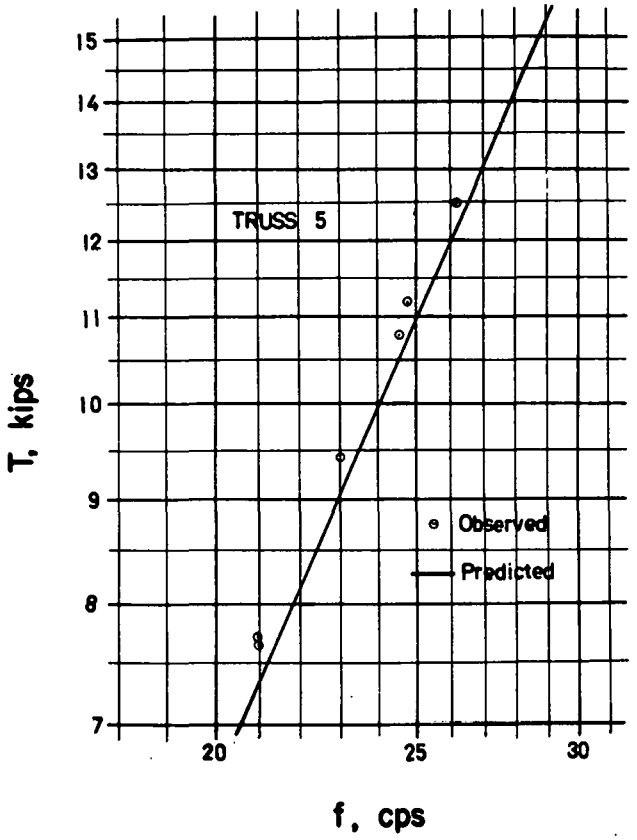


Figure 5

(the letter T in Figure 5 stands for tension in the horizontal chord, not th inclined chord). The observed data cam from strain gages and frequency measurements. The solid line is predicted by the formula.

The error in predicted tension can be in either direction but is typically not more than about 500 pounds. Because the slope of the field data follows the formula so closely and because the resolution of the frequency measuring device is within a tenth of a cycle, changes in chord tension of as little as 100 pounds can be detected. Although the technique has only been perfected for the horizontal chords its value in checking installation tensions and subsequent performance is obvious.

## FUTURE

Work is presently underway to make similar measurements on the inclined chords. The fact that these chords are anchored in the roof greatly complicates the problem of exciting and measuring their frequencies but laboratory measurements with amplified signals suggest that the technique might still be applicable.

In any event it will be necessary to observe a large number of trusses before the upper and lower bounds are defined well enough to be used for design purposes. But before truly rational design can be executed more knowledge of the interaction between roof and truss must be obtained.

In an attempt to speed up the installation process the Bureau is about to fund the development of a machine which should not only reduce drilling time, but decrease the number of manual operations involved in assembling and tightening trusses. A number of independent attempts are also being made along this line. If and when this goal is achieved trusses can be installed in the cycle. At that point it will be feasible to experiment with using trusses as the primary support system instead of as a secondary one. With an enhanced

competitive edge, trusses may begin to play an even larger role in roof support in the Illinois coal basin and beyond.

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# DESIGN AND APPLICATION OF THE MINE ROOF TRUSS SYSTEM in the Illinois Coal Basin

Dr. Robert M. Cox, P.E.

Mining Services, Inc.
Tuscaloosa, Alabama

R. Melton Cox, Asst. to the President

Birmingham Bolt Company
Birmingham, Alabama

## INTRODUCTION

Although notable progress has been made in the science of rock mechanics, mine roof control remains one of the more baffling problems confronting the mining engineer and roof falls continue to be the number one occupational hazard of underground miners. The authors have devoted considerable time and effort during the past decade working on roof-control problems in the Illinois Basin and have developed several innovative concepts and techniques that have aided in the general understanding and control of coal mine roof problems. The most notable contribution being the introduction and refinement of the mine roof truss system.

Conceptually, the roof truss system resembles a bridge truss assembly with the rock acting as the compression members and the steel rods acting as the tension members (see Figure 1). The roof truss in its simplest form consists of anchoring two steel rods at the back of holes collared a few feet from the rib and drilled at angles of 45 degrees over each mine pillar. The ends of the rods are connected together by a tie rod and tensioned by a turnbuckle. The tensioned rods produce an uplifting force in the roof rock at the blocking points with the corresponding downward reactions being transmitted to the region over the pillars at the anchor points. The ground forces generated by a tensioned roof truss can be designed to provide support by either direct suspension of the rock load within the failure zone or indirectly by the reinforcement of the immediate roof rock that transforms it into a self-supporting roof rock structure.

The main effort during the past decade has been directed toward proving the concept and developing the hardware necessary for the economic and efficient use of the roof truss system as a primary means of coal mine roof control. Initial installations were made in 1966 in the Concord Mine of U. S. Steel Corporation in Alabama using 3/4 inch diameter wire rope members to which short threaded rods had been swaged to allow for the installation of rock bolt anchors and tensioning nuts (see Figure 2). The assembly was tensioned by means of an offset bracket using an impact wrench. These four initial installations were placed on four foot centers in 20 foot wide entries and proved the concept of the system, but were expensive to fabricate and awkward and time-consuming to install.

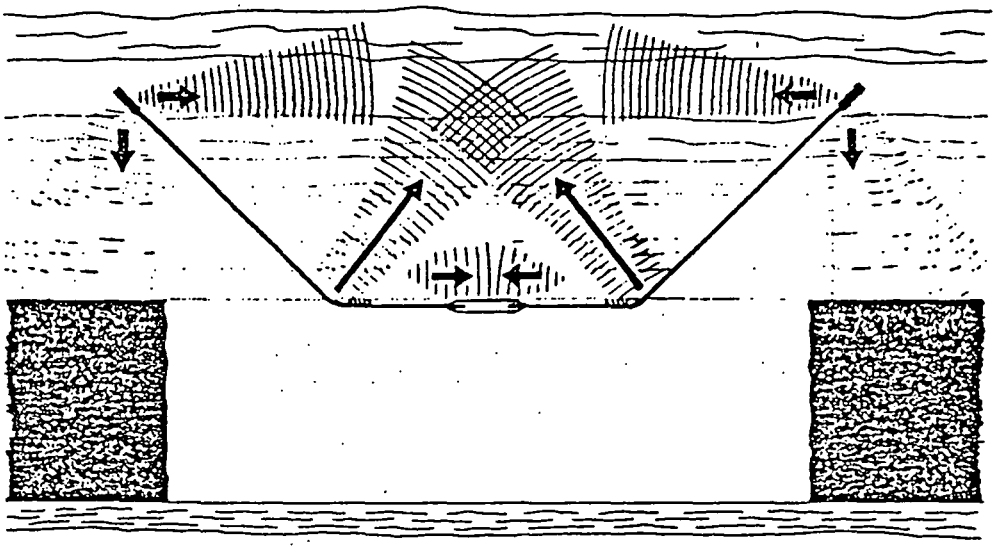

Fig. 1. The Roof Truss Conception. When the turnbuckle pulls the steel rods into tension, compression is produced in all of the shaded areas. The upward and inward thrust provides positive roof support.

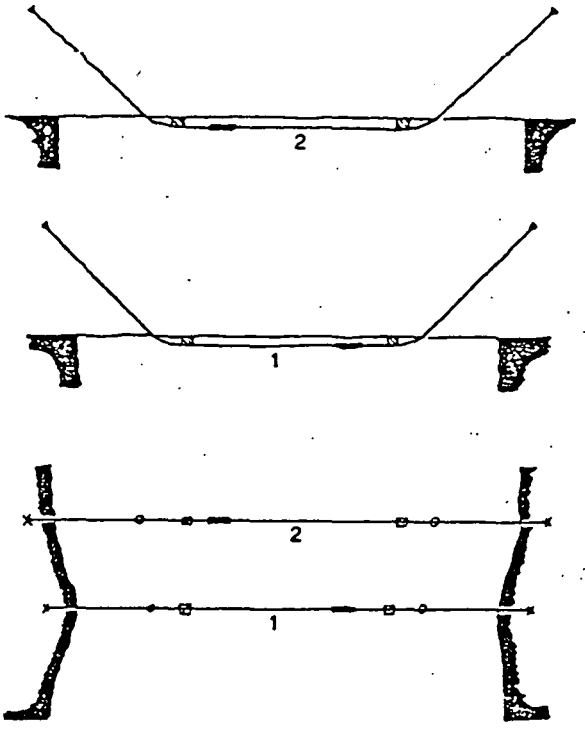


Fig. 2. Initial Roof Truss Installation Using Wire Rope Members. A - plan of installation; B - cross-section of first truss; C - cross-section of second truss; scale of drawing - one inch equals five feet (1 cm = 60 cm).

During the next three years numerous roof truss configurations were developed that consisted of a variety of combinations of steel straps, steel rods and roof bolt assemblies placed in a variety of configurations (see Figure 3). The concept of tensioning the truss assembly with a turnbuckle was incorporated during this development period. A major problem at this time was the design and manufacture of a strong economical turnbuckle. Many of these experimental truss systems were installed in Illinois Basin coal mines in West Kentucky to control severe roof conditions. Incidentally, the first installation of a truss in a noncoal mine was made in the fluorspar mine of the Minerva Oil Company near Cave-In-Rock, Illinois, in 1967. These roof truss assemblies consisted of roof bolt anchorage members to which were connected 3" x ¼" straps by means of a pin connection at the location of the roof bolt plates. These truss systems were placed on four foot centers in 12 foot wide entries to hold weak roof rock which previously had been unsupportable and thus allowed the mining of the lower ore bench in the area.

During 1968 the truss assembly began to evolve into the standard 3/4 inch and one inch diameter rod systems. During this time the system was successfully used to support 20 to 24 foot wide entries in a number of coal mines in Kentucky and Alabama. In addition, a compound truss was used to hold mine roofs in 30 foot wide entries (see Figure 4). Conceivably compound trusses could also be used to support wider openings. In 1968 the truss system was also first applied to the problem of supplying positive support for rock slope entries into the Chetopa Coal Mine in Alabama (see Figure 5). Such installations have since become commonplace for main mine entries and slopes.

The truss system was next applied to the difficult problem of supporting fourway intersections in room and pillar coal mines using a variety of installation patterns (see Figure 6). The choice of roof truss patterns for intersection support is dependent on the various combinations of mining sequence, health and safety regulations, material and supply logistics and local roof rock mechanics. These installations prove that the truss concept, if properly applied, provides a positive support method for fourway intersections in room and pillar mines.

By 1970 the concept of the roof truss system was definitely proven by the success of numerous installations under a wide variety of mining conditions. The mechanical hardware had evolved into the standard 3/4 inch and one inch diameter rod systems as shown in Figure 7. It should be mentioned in connection with this system of roof support that several patents have been issued in the United States and several foreign countries.

The next major advancement was the development of resin point anchorage for the standard roof truss assembly (see Figure 8). These developments were initially tested and refined in Illinois Basin mines of Peabody Coal and Island Creek beginning in 1975. In general at each test site a series of resin point anchor tests were conducted using 12 inch, 18 inch and 24 inch resin point anchors. The results of these tests support the current recommendation that 24 inch resin anchor points are sufficient to ensure a stable anchor (greater than 14 tons) for the roof truss system.

The resin point anchor development work resulted in a re-design of resin point anchors, and the development of a new drilled and taped rebar anchor that is standard for roof truss anchorage. The new resin point anchor is also being adapted as an anchor for roof bolting plans

## Current Development Work

The remaining problem that has restricted that use of the roof truss as a primary roof support system has been

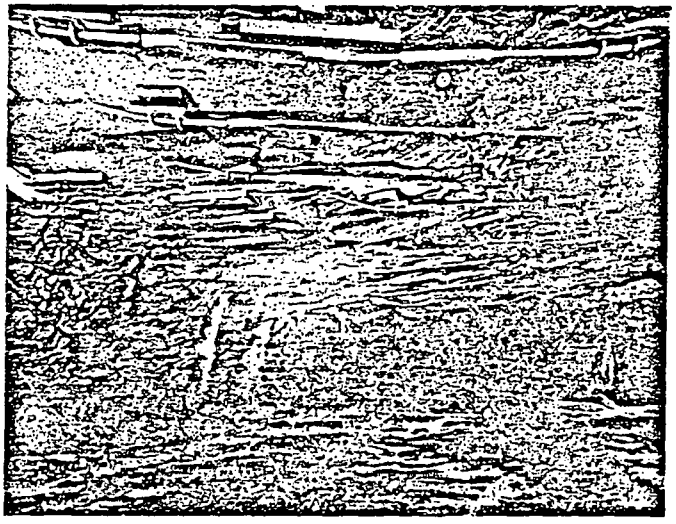

Fig. 3. Photograph of Roof Truss Installation in Coal Mining Entry.

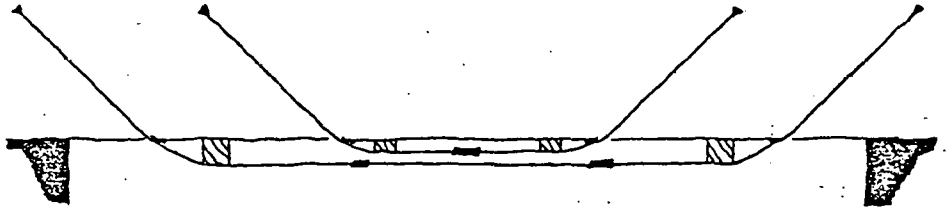

Fig. 4. Compound Roof Truss Used in Wide Entries. Scale of drawing - 1 inch equals five feet

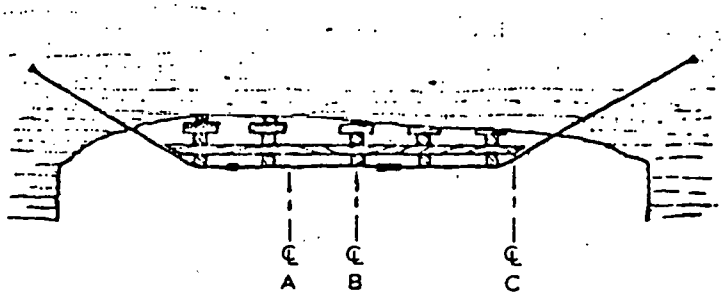


Fig. 5. Roof Truss Support Plan for Rock Slopes. A - centerline of track; B - centerline of slope; C - centerline of belt.

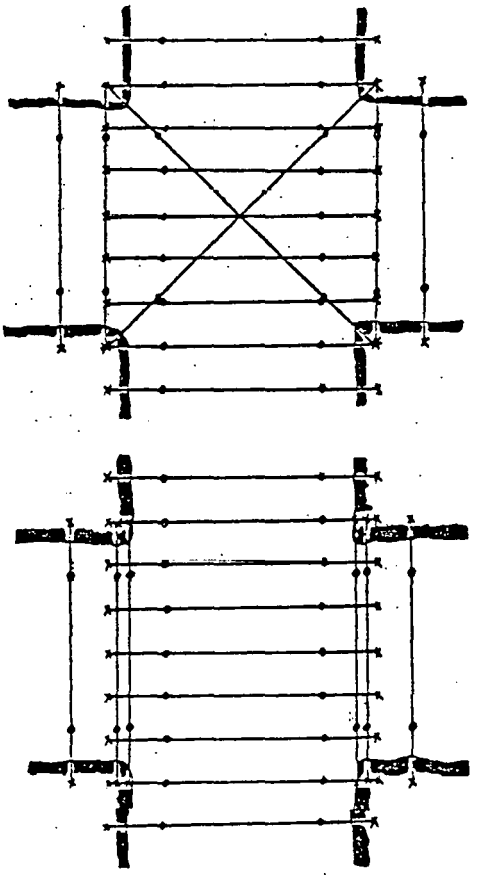

Fig. 6.Roof Truss Support Plans for Four-way Intersections. A - standard plan; B - cross plan; scale 1 inch equals 5 feet

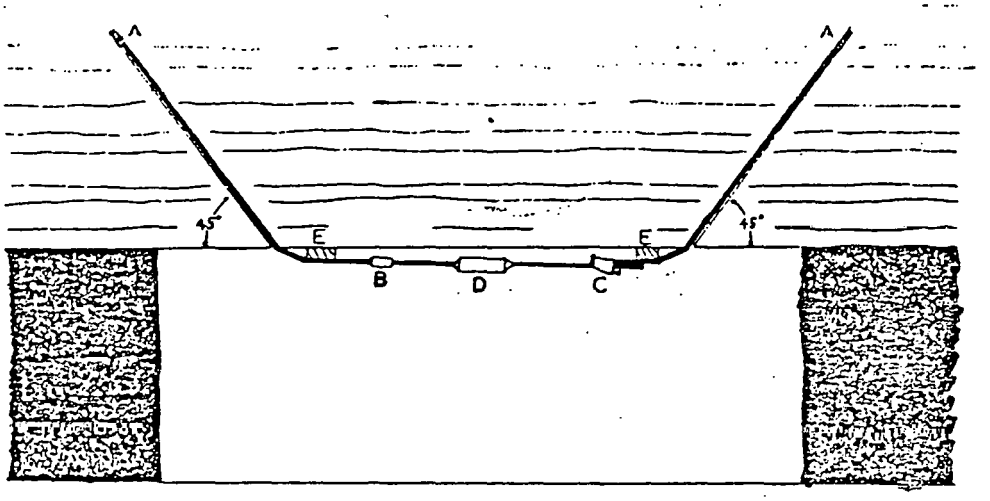


Fig. 7. Typical Roof Truss Installation. A - expansion shell anchored over the pillar; B - coupling for length extension; C - wedge box for length adjustment; D - turnbuckle for tensioning; E - support plate and blocking.

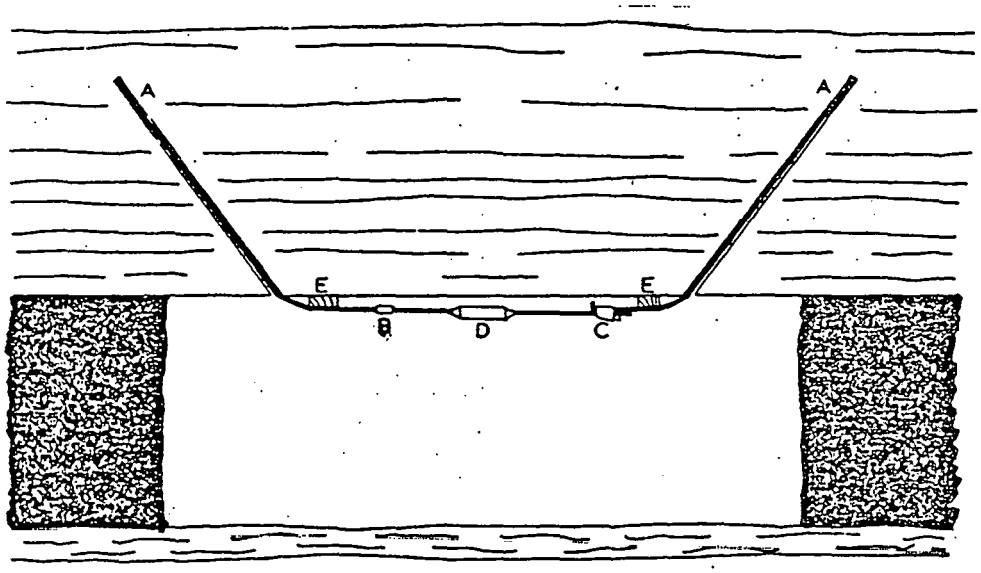


Fig. 8. Resin-anchored Roof Truss System. A - two foot (30 cm) rebar-resin anchor; B - coupling for length extension; C - wedge box for length adjustment; D - turnbuckle; E - support plate and blocking.

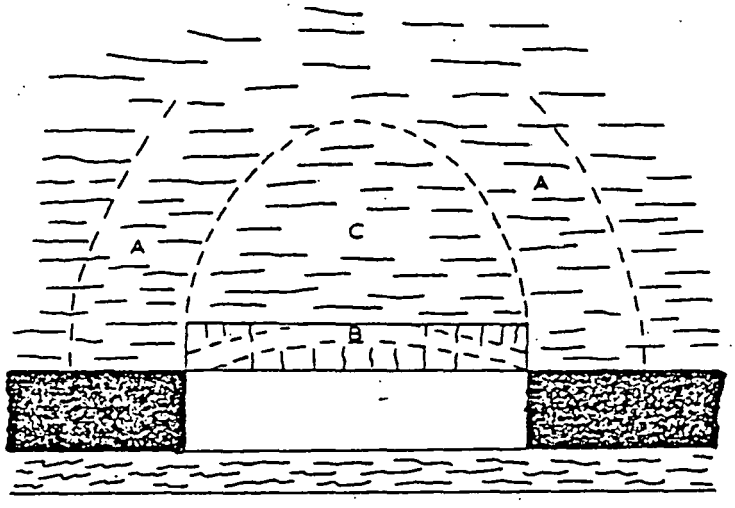


Fig. 9. Illustrative Concept of Ground Arch and Rock Arch. A - ground arch; B - rock arch; C - rock load.

the development of roof drills capable of drilling the inclined holes necessary for the correct installation of the truss system. Several prototype drill machines have been developed in recent years to drill the angle holes and research and development work is currently being sponsored by several private companies as well as the U. S. Bureau of Mines. The goal of this research work is to speed the development of the drilling equipment necessary for efficient applications of the roof truss system as primary support in the mechanical mining cycle.

Current tests are being conducted in Illinois basin coal mines of Peabody Coal, Island Creek and Old Ben to determine methods, equipment and procedures for installing roof trusses in the face cycle.

## Mechanics of Roof Truss Support System

Observations and study of numerous mine roof failures in the basin indicate that the typical coal mine roof fails in an arch configuration, the size and shape being dependent on: the opening geometry, the enclosing geologic structure and the in-situ ground forces, as altered by the mining process (see Figure 9). The stable ground arch remaining after a roof failure is generally accepted as the mechanism by which the major portion of the overburden load is transferred to the ribs of an underground opening. The weight of the rock within the limits of the ground arch comprised the rock load that must be supported to ensure opening stability. Two schools of thought have been advanced to explain the mechanism of roof failures and thus aid in the development of a positive roof support system. The "suspension" school assumes that in the worst case the support system must support the dead weight of the rock within the potential failure zone; whereas the "reinforcement" school assumes that the immediate roof rock can be transformed into a self supporting rock arch structure by the addition of an adequate roof rock reinforcement system.

The "suspension" theory adquately explains conditions encountered in background with weak roof rock and/or heavy ground pressure, whereas the "reinforcement" theory best explains the typical mine roof conditions. The mine roof truss system has proven to be a solution to both types of roof control problems. The truss system has been successfully used to support mine roofs in areas heretofore considered unmineable because of severe ground problems as well as being applied in numerous mines to provide positive support for long life entries being supported in the reinforcement sense.

## Suspension Analysis

The design of a roof support system by the suspension theory dictated that the support system be of sufficient strength to support the total weight of the potential roof failure. In the worst case the potential failure zone will extend vertically above the underground opening, thus the weight to be supported can be determined by the relationship:

$$W = whLb$$

where W is the rock load to be supported

w is the unit weight of the rock,

L is the width of the opening,

h is the height of the rock fall,

b is the spacing between supports.

The suspension support capacity of the roof truss is:

$$S = 2T \sin \alpha$$

where S is the support capacity,

T is the installed tension,

$\alpha$ is the angle of inclination of the anchor-points (typically $45^{o}$).

Since the support capacity (S) must balance the rock load (W) for effective support, a design relationship for determining the desired tension (T) in a roof truss can be developed by combining the two previous equations:

$$T = whLb/2 \sin \alpha$$

The variables in the design equation should be determined by observation of previous roof falls and/or experience in similar conditions. The roof truss system is presently manufactured using 3/4 inch and one inch diameter rods of high strength steel (50 ksi). Figure 10 depicts the typical suspension support capacity of these truss systems spaced at two and four foot centers along the length of an entry. The suspension criteria have been used to design roof truss systems for both severe ground conditions (weak rock and high falls), and draw rock problems in coal mine entries (one to four feet thick).

## Rock Arch Analysis

It has long been recognized by miners and engineers that the strength of the support system (i.e. timbers, bolts or trusses) required to stabilize a mine roof is typically much less than that required to support the roof by suspension alone. The success of such installations can be explained as the creation of a competent reinforced rock arch within the immediate roof rock overlying the underground opening (see Figure 9). To effectively analyze or design a competent self-supporting reinforced rock arch, the mechanics of the rock arch must be examined. The loads and reactions on a rock arch are shown in Figure 11. Since the rock is assumed to possess no tensile strength the horizontal reactions (H) at the abutment and crown of the arch can be determined by taking moments about point A (see Figure 13):

$$H = whL^2b/8z$$

where H is the horizontal reaction,

w is the height of the rock load,

L is the width of the opening,

b is the support spacing

Z is the rise of the rock arch axis, typically 3/4 t,

t is the thickness of the arch.

The vertical reaction or shearing load (V) acting at the abutments is equal to one-half the rock load:

$$V = whLb/2$$

The stability of the rock arch is dependent on four principle factors:

(1) Arch unity

(2) compressive strength of the rock,

(3) shear strength at abutments, and

(4) deformation characteristics of the rock.

The rock arch may occasionally develop naturally, but in most cases artificial supports must be installed to hold the rock together to form a competent arch. The principle supporting function of roof bolts (mechanical or resin) is thought to be the holding together of the roof rock to form a competent roof rock arch. The roof truss performs the same function as roof bolts (holding the rock together) as well as providing additional reactive support loads that significantly reduce both the compression and shear loads that must be resisted at the abutments of the rock arch to ensure roof stability.

The additional forces superimposed by the roof truss upon the rock arch are shown in Figure 11. The horizontal reaction ($H_t$) in a roof truss reinforced rock arch can be determined by the equation:

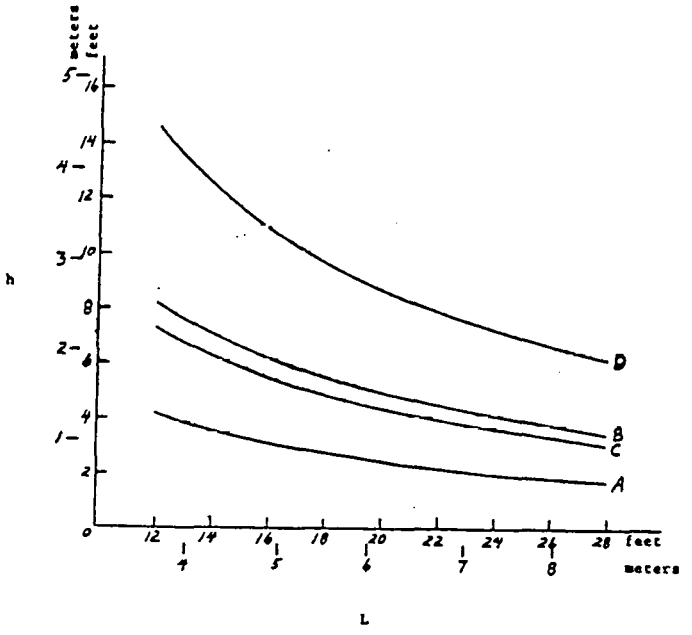


Fig. 10. Support Capacity of Roof Truss in Pure Suspension. A - 3/4 inch (18 cm) truss at 4 foot (1.2 m) spacing; B - 3/4 inch (18 cm) truss at 2 (.6 m) spacing; C - 1 inch (25 cm) truss at 4 foot (1.2 m) spacing; D - 1 inch (25 cm) truss at 2 foot (.6 m) spacing.

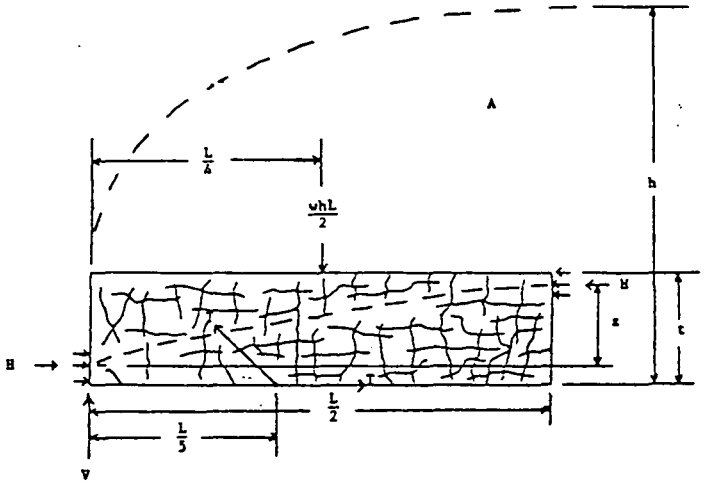


Fig. 11. Mechanics of Rock Arch. A - rock load to be supported; H - horizontal reaction; w - weight density of rock; h- height of rock load; L - width of opening; b - support spacing; z - rise of rock arch; t - thickness of rock arch; V - vertical reaction; T - tension in support.

$$H_t = \frac{whL^2}{8Z} - \frac{T}{bZ}\left[\frac{L}{5}\sin\alpha + \left(\frac{t-Z}{2}\right)(1-\cos\alpha)\right]$$

$$- \frac{T}{b}(1-\cos\alpha)$$

The vertical reaction or shear load ($V_t$) at the abutments of a roof truss reinforced rock arch is equal to:

$$V = \frac{whL}{2} - \frac{T}{b}\sin\alpha$$

The reactions generated by the installation of inclined roof bolts in shown in Figure 13. This system is sometimes used to simulate a tensioned roof truss but lacks the advantages as depicted in Figure 11. The horizontal reactions ($H_i$) of an inclined roof bolt reinforced rock arch can be determined by:

$$H_i = \frac{whL^2}{8Z} - \frac{T}{bZ}\left[\frac{L}{5}\sin\alpha - \frac{t+Z}{2}\cos\alpha\right]$$

The vertical reaction or shear load ($V_i$) of an inclined bolt is equal to:

$$V_i = \frac{whL}{2} - \frac{T}{b}\sin\alpha$$

The comparison of the reactions generated by typical support configurations have been determined and tabulated for typical mine roof conditions (see Table I). The results indicate the superior performance of the mine roof truss system in reducing mine roof loads that lead to roof failure. While the inclined bolts reduce the horizontal and vertical reactions by two and 18 percent, respectively, the 3/4 inch diameter truss reduces these same reactions by a margin of 40 and 37 percent, respectively, and the one inch truss reduces the same reactions by a margin of 60 and 55 percent, respectively. Conceivably larger and stronger roof trusses could be made to further reduce these roof loads and abutment reactions.

## Installation Procedures

The successful use of the roof truss system requires that the system be properly installed and maintained. The few failures that have occurred have been traced to either faulty installations (not placing the anchors over the rib line) or inadequate rod sized required to support heavy suspended loads. The successful application of the system requires that the mechanical strength of the system be properly designed for the intended roof support needs and that the system be properly installed according to prescribed procedures (see Table II).

The engineering concepts of the roof truss system have been proven and demonstrated by the success of several hundred thousand installations in coal, iron ore, trona, fluorspar and copper mines and numerous rock slopes and tunnels. With the continued development of equipment for the mechanical placement of the roof truss system in the mining cycle, we may well be close to the day that roof failures cease to be a major hazard to underground miners.

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TABLE I

Rock Arch Reactions for Typical Mine Roof Conditions

| Reaction | Normal Bolting Pattern | Inclined Bolts | 3/4 Inch Dia. Roof Truss | One Inch Dia. Roof Truss |
|---|---|---|---|---|
| Horizontal (H) | 16,000 | 15,705 | 9,577 | 6,365 |
| Vertical (V) | 9,600 | 7,832 | 6,065 | 4,297 |

The values used in these calculations are:

roof span (L) = 20 ft.
height of rock load (h) = 6 ft.
weight density of rock (w) = 160 pcf
rock arch thickness (t) = 4 ft.
rise of arch axis (Z) = 3 ft.
angle of inclination ($\alpha$) = $45^{o}$

roof bolt tension (T) = 10,000 lbs.
roof truss tension (t) = 20,000 lbs. (3/4 inch diameter)
for one inch diameter (t) = 30,000 lbs.
supporting spacing (b) = 4 ft.

TABLE II

Roof Truss Installation Procedures
(Job break-down)

I. Laying off Roof Truss Pattern on Roof

1. Measure width of entry and mark off straight line perpendicular to entry axis.

2. Mark an X on the roof one-fifth the entry width from each pillar.

II.(a) Lining up Stoper Drill for Drilling

1. Drop an object from each X on the roof and joint points with straight line along bottom of entry.

2. Collar holes in roof at X marks.

3. Position extension of stoper drill along line in bottom to get $45^{o}$ angle.

(b) Position Mobile Rotary Roof Drill to Collar Holes at Marked Point

III. Depth of Holes

1. (One-fifth of entry width plus anchor length) times 1.4.

IV. Mine Roof Truss Installation

1. Install support nuts on threaded studs.

2. Install expansion unit on threaded stud.

3. Insert assembly into drill hole to full depth and tighten with wrench to secure good anchorage.

4. Repeat above steps for other side.

5. Bend rods up against roof.

6. Connect the saw tooth rod with turnbuckle to threaded bolt about one inch (2.5 m).

7. Place wedge box end first onto 1/2 head bolt and slide down over saw tooth rod until it rests against the 1/2 head on bolt.

   Note: The small end of the wedge box should always point toward the center of the entry.

8. Place small end of wedge into large end of wedge box, push up on rods to take up slack from assembly and tap wedge into place.

9. Place support plate on rods within 12 inches (30 cm) of each hole. Place blocks on top of support plates.

10. Tension assembly by tightening the turnbuckle.

# SPLIT SET[R] FRICTION ROCK STABILIZERS [1]

Dr. James J. Scott

Adjunct Professor of Mining
Missouri School of Mines
University of Missouri - Rolla
and
President of Scott Mine Technical Services - Rolla, Missouri

[R] Registered Trade Mark-Ingersoll-Rand Corporation

1 The First Presentation of this Paper was at the Canadian Institute of Mining and Metallurgy Annual Meeting-April 23, 1979, in Montreal

## INTRODUCTION

Friction Rock Stabilizers, commonly called Split Set[R] Stabilizers, have been under development by the author and Ingersoll-Rand Company since early 1973. Five years of intensive laboratory and field testing led to its introduction to the metal mines in the United States in the fall of 1976. Actual sales started in February of 1977 and, to date, over five million units have been marketed to metal mines. The present rate of useage is near 3 1/2 million units per year which is approximately 50% of all roof fixtures used in the United States metal mining industry.

Mr. Richard Hoppe, Senior Editor of Engineering & Mining Journal in an article of the February, 1979, issue entitled "Winning the Battle Against Bad Ground" said, "In Hardrock mining operations in the western U.S., particularly underground uranium mines where exceedingly difficult ground is common, no roof support system has been accepted more quickly than the Split Set friction bolt invented by Dr. James J. Scott and developed by Ingersoll-Rand Co."

This paper describes how Friction Rock Stabilizers function and presents technical data to show the uniqueness of Friction Stabilizer systems.

## SPLIT SET[R] STABILIZERS

The Split Set system is a revolutionary development in the field of ground control. The system produces an interior reinforcement to a rock mass which:

1. Is simple in its concept.
2. Adjusts to the degree of restraint needed to match rock properties.
3. Provides a yieldable support and, later, a more rigid support

as it is needed.
4. Pre-loads the rock in compression to increase stability.
5. Is a full-contact, fully tensioned device to resist rock loads.
6. Minimizes the stress concentration effects inherent in any interior support device.
7. Improves safety for the miner.
8. Is cheaper than a grouted system.
9. Offers the rock mechanics engineer an opportunity to design in a fashion not before possible.

Split Sets are 1 1/2" outside diameter as manufactured and normally are put into 1 3/8" diameter borehole in rock. A Split Set Stabilizer for a 1 1/4" diameter hole has been developed to serve the Canadian Mining Industry where such holes are the standard. Depending on the smoothness, diameter throughout its length, straightness of the hole and frictional coefficient, an anchorage of 3/4 to 1 1/2 tons is obtained per foot of borehole contact upon installation. Figure 1.

A theoretical analysis has shown the manner in which the Split Set contacts the borehole.[1] Figure 2 is a free body diagram of a unit in the borehole. The side opposite the slot is plastically deformed and contacts the sidewall over some angle "p". From the edge of the slot to a point at the edge of the angle of contact the Split Set acts similar to a curved beam and is loaded within its elastic range and does not contact the hole surface. A line load is generated at the edges of the slot. This can be demonstrated practically by placing a paper inside a cylindrical hole and holding it to the light and the uncontacted area is readily seen. In the field, the actual contact of the Split Set to the wall is increased substantially due to roughness of the hole, rifling, scraping of the hole during insertion, etc. In essence, the hole serves as a die and the Split Set is force-fit to the borehole configuration.

The steel used in the fixtures is a high formability steel with approximately a 70,000 psi yield. Wall thickness is near 0.090 inch. The stabilizer is tapered on the upper end for insertion into the bore hole and on the other end, a ring is welded to the unit as a retainer for the roof plate.

## FRICTION ROCK STABILIZER SUPPORT MECHANISM

The Friction Rock Stabilizer, or Split Set$^{R}$, is a very simple device with no moving parts. Upon installation it provides a yieldable support and, with time, becomes more rigidly anchored in the rock to provide the restraint necessary for long term stability.[2] It is a full contact system which pre-loads the rock in compression to increase stability. Since the metal is in contact with the rock throughout the circumferance of the hole over the length of the device there are no large stress concentrations to creep or bleed off with time.

The yieldable aspects of the Split Set Stabilizers without loss of restraint against the rock cannot be overemphasized. It is the author's opinion that this feature is what has made the device so successful in supporting ground under the worst geologic conditions. So, the preload of the rock by the device during and after installation is very important. The preload acts in a plane perpendicular to the axis of the device and also along the axis as it is driven in. This combined effect produces a zone of triaxial compression around

# SPLIT SET STABILIZER CONFIGURATION

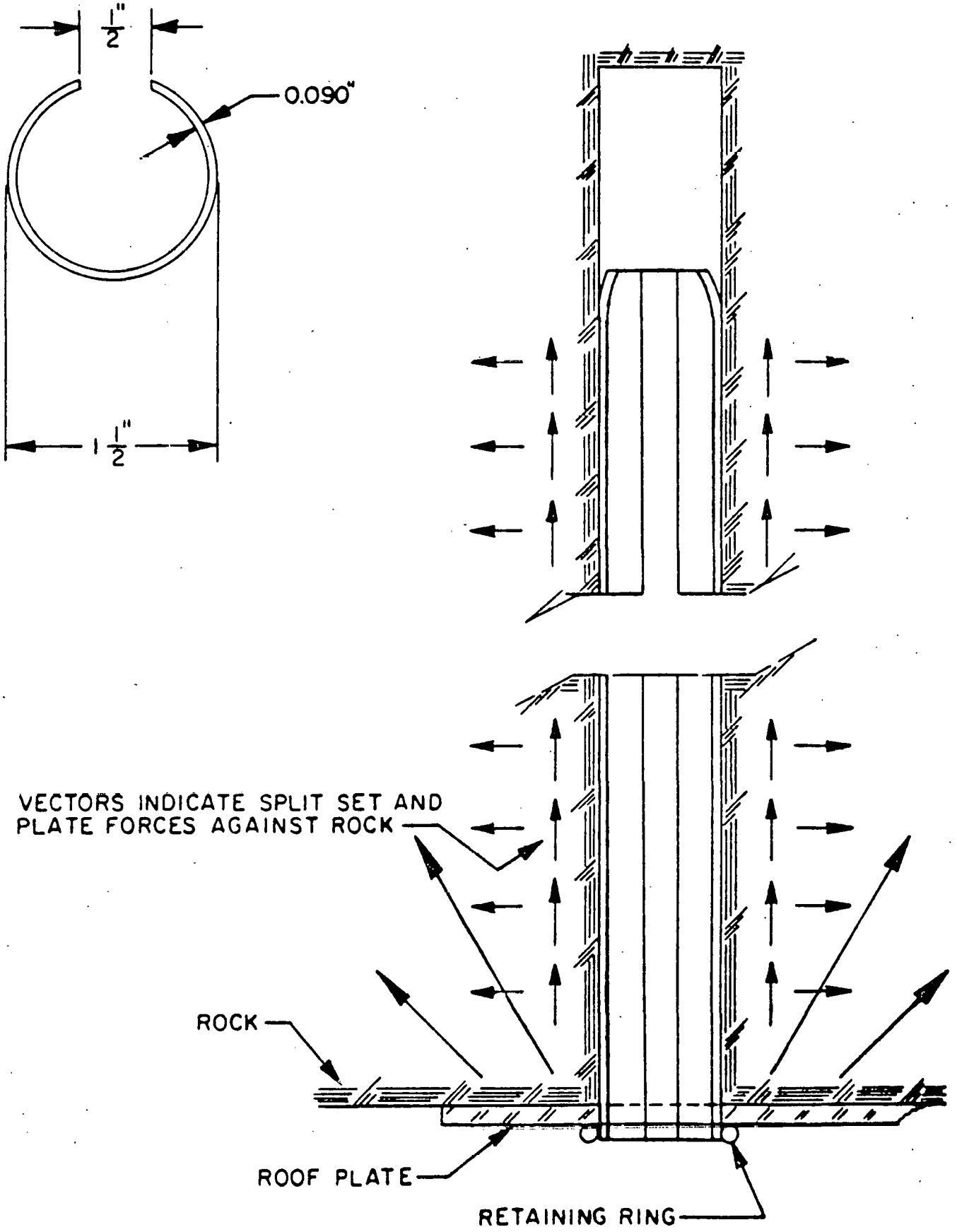



FIGURE 1

# FREE BODY DIAGRAM

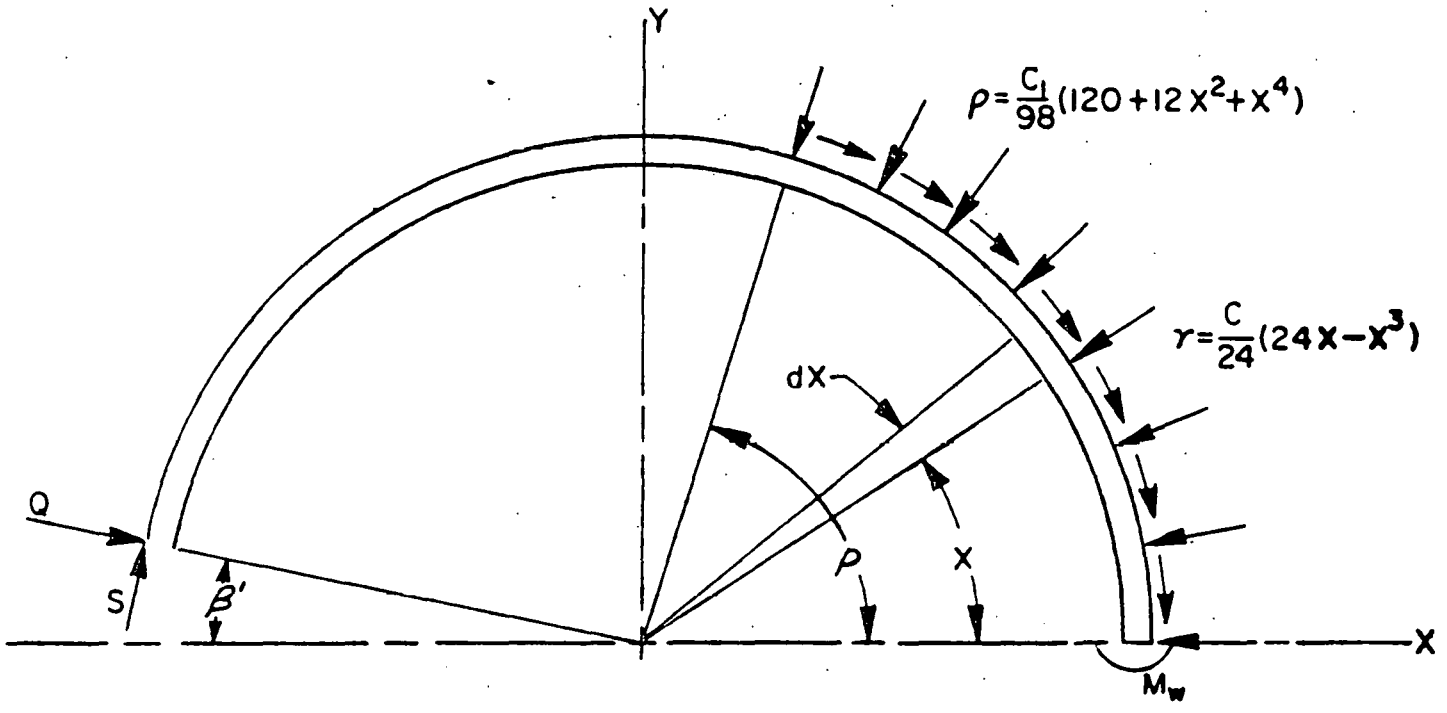



FIGURE 2 FREE BODY DIAGRAM OF THE SPLIT-SET CROSS SECTION

the device[3] and, when coupled with the pressure bulb which is present in the rock from the plate, we end up with a unique pear shaped compression zone around each device. When Split Set Stabilizers are placed in pattern so that the base of the pear interlocks with the adjacent one, stability is obtained even in very weak rock. With time anchorage increases. This is due to such things as the diameter of the borehole decreasing, changes in the frictional coefficient at the metal-rock interface, and offsets in the borehole which deform the stabilizer due to bedding plane slip or to shearing of the rock. See Figure 3.

ANCHORAGE TESTS

Field tests indicate that anchorage is greatest near the end of the borehole and farthest from the collar. This is due to hole enlargement due to lack of drill steel restraint which produces wobble and hole enlargement and thus decreases the confinement at the collar. Figure 4 shows the results obtained in holes drilled 6 ft. deep.[4] The space between the two curves indicates the range of values to expect for each Split Set length. The higher values obtained for the 4 ft., 5 ft. and 6 ft. length are due to the increased anchorage near the bottom end of the hole.

COMPARISON OF INTERNAL ROCK REINFORCEMENT SYSTEMS [5]

Three devices are compared with Split Set Stabilizers in regard to the manner in which they load the rock mass. The three units are: The standard mechanical bolt, a resin bolt using a headed rebar, and a resin bolt using a threaded bar with a nut for anchoring the roof plate. These devices are compared with a Split Set Stabilizer in Figure 5. An explanation of the mechanism of each follows:

a. Standard Mechanical Torque Tensioned Roof Bolt

This is an active device in that it loads the rock at the plate and at the anchor upon installation. Unfortunately, these devices are susceptible to blast damage and to anchor creep. In a properly installed unit, a pressure bulb compresses the rock to near 6000 lbs. under the plate and there is a like 6000 lb. reaction at the anchor. The pressure bulb developed by the anchor end is really not fully understood. Fracture of the rock at the anchor often occurs. Lang estimated rock stresses at mechanical anchors as high as 125,000 psi. [6]

b. A Resin Roof Bolt Using a Headed Rebar

This device is essentially a passive unit in that little plate load is obtained during placement. The maximum plate load possible is that which can be pushed against the plate with the installing machine. In the case of a mechanical roof bolter, this is probably near 1500 lbs. and in the case of an air leg, it is about 500 lbs. There is no pre-stress radially outward from the resin.

c. Resin Roof Bolt Post-Tensioned With a Threaded Nut

This device provides a pressure bulb under the roof plate in a manner somewhat similar to a good standard mechanical roof bolt. The tension in the rebar will only extend up the bar for perhaps six inches as this type of anchor system is quite rigid and can be classified as a high anchorage system. The whole system above this point is essentially passive and plays no support role until the rock move

d. Split Set[R] Stabilizer

This device is a fully active fixture. Plateloads, measured with compression pads, of 3 tons are readily achieved and radial loading exists along the length in all areas where the slot is

# DETAIL OF SPLIT SET STABILIZER IN ROCK

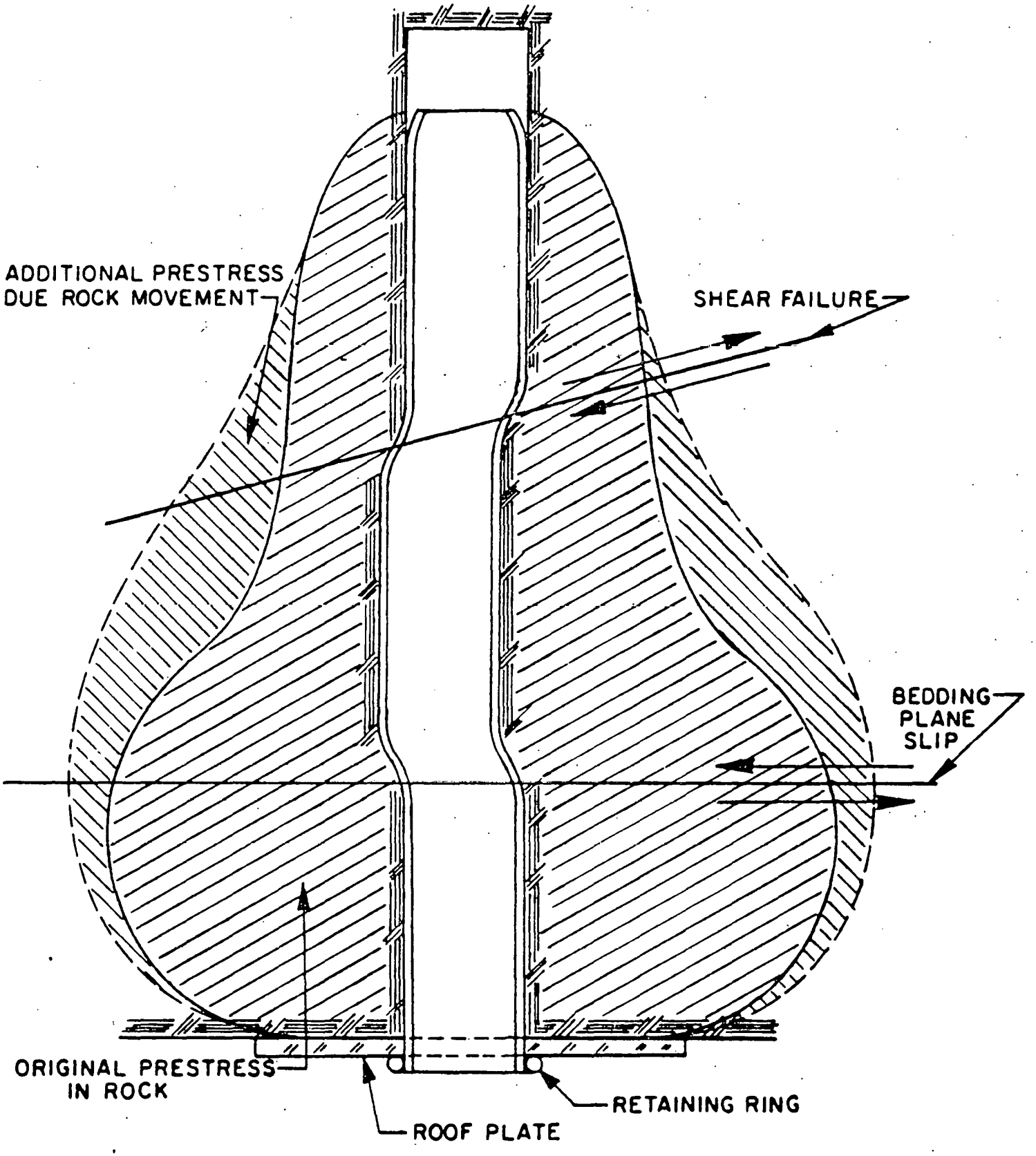



FIGURE 3

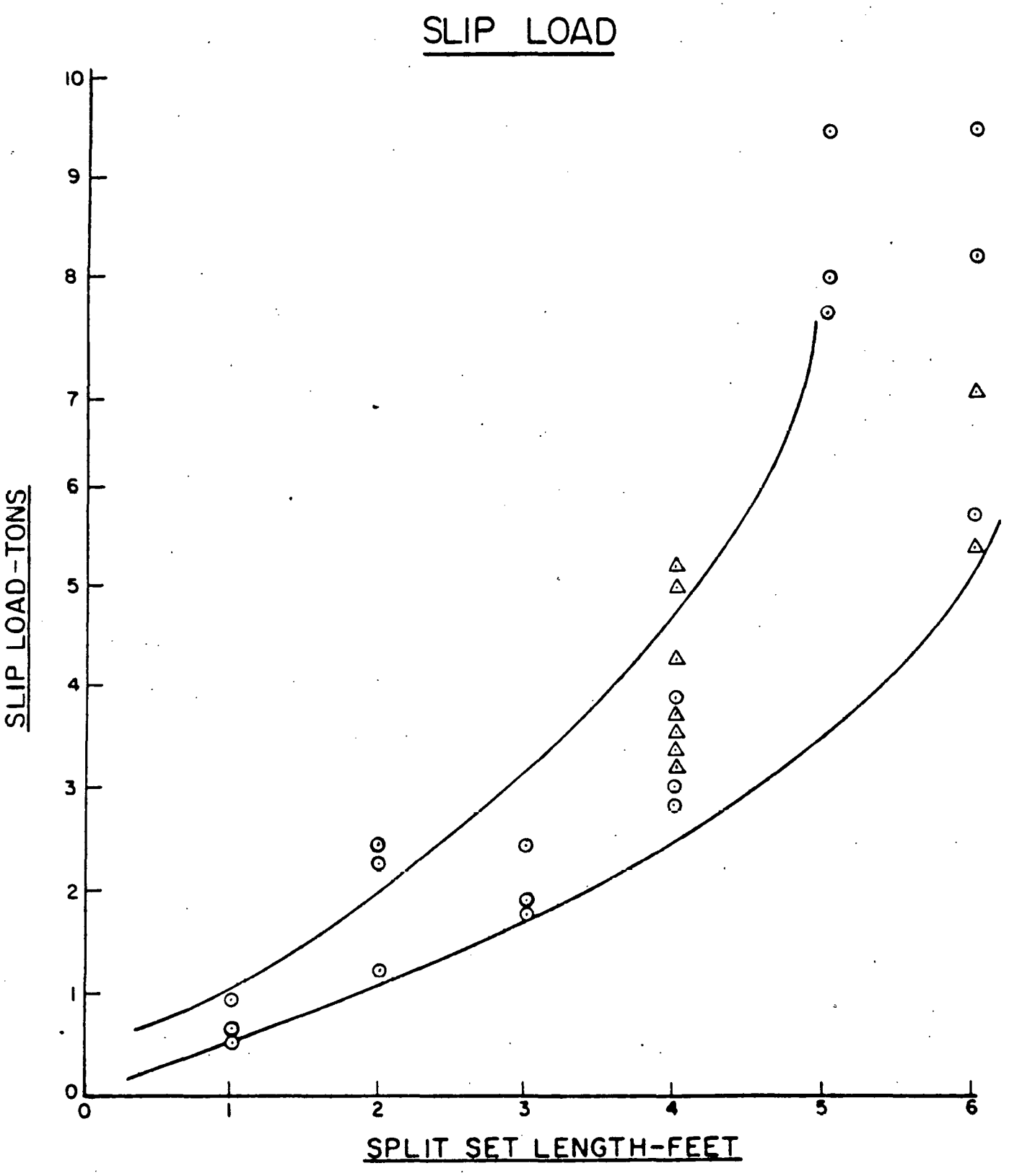
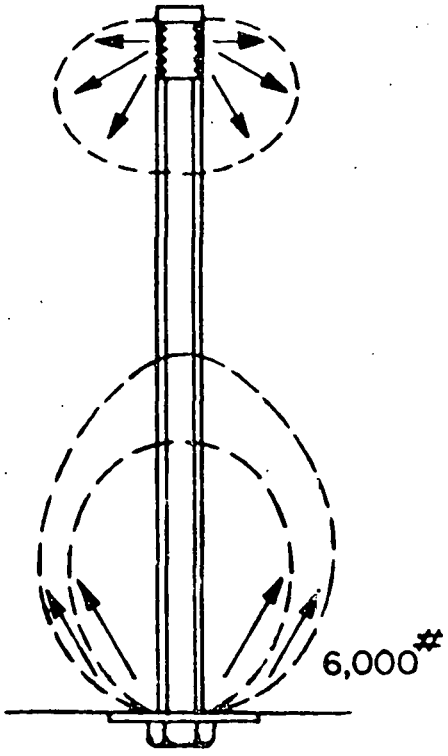
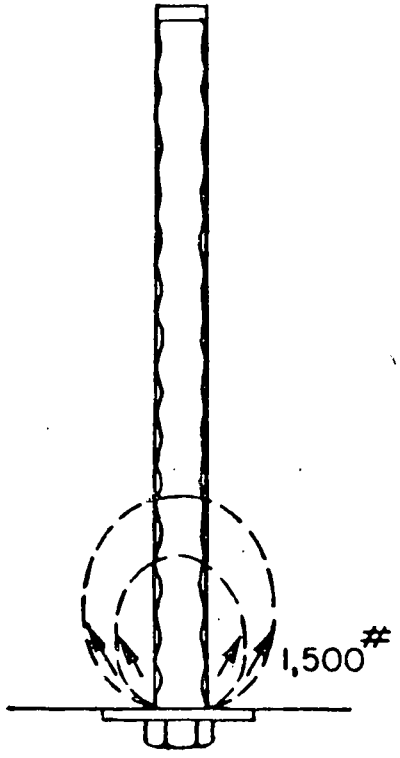



FIGURE 4 Slip Load of Split Set Rock Anchors at various lengths. Hard Shale

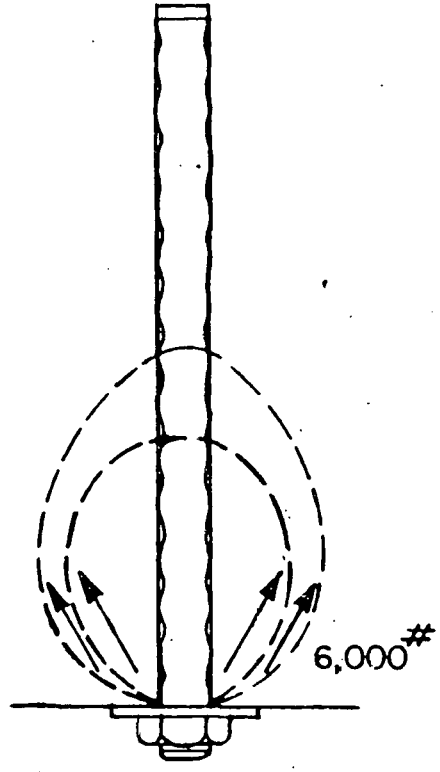
# SUPPORT FIXTURES



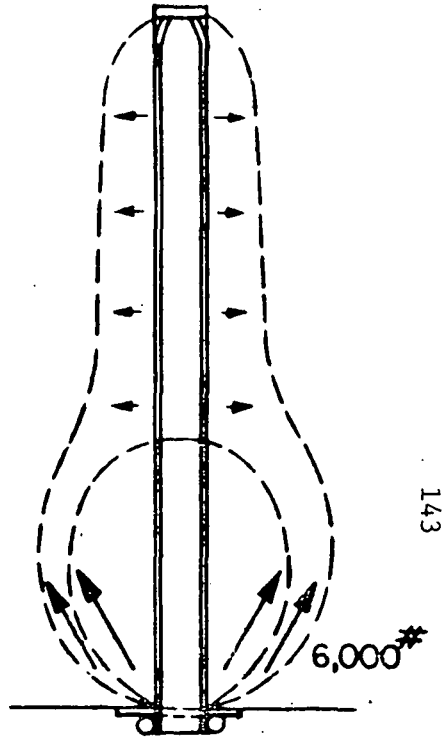

(a) Mechanical




(b) Resin-Head Rebar




(c) Resin-Thread Rebar




(d) Split Set Stabilizer

FIGURE 5

deformed in the Stabilizer. Under the plate there is a pressure bulb similar to the mechanical and to the post-tensioned resin fixture with the added effect of radial compression provided throughout the length of the device. The degree of anchorage per unit length can be adjusted by controlling the size of the hole drilled in the rock. The levels of anchorage are sufficient for holding the rock in place but are not so high that they do not allow rock movement when the total mass is over-stressed, as will happen to rocks in deep excavations.

PULL TEST RESULTS

Pull tests on Split Set Stabilizers, immediately after installation are quite different than those obtained in tests on mechanican or resin or cement-grouted bolts. Figure 6 shows the results on typical tests in hard rock.

Note that both the resin, or cement anchors, and the mechanical anchor systems, have experienced failure at less than 1/2" movement. The Split Set Stabilizer is still intact after 2 1/2" movement and can at any time be redriven and will perform in a similar manner if pull tested a second time. It is this yieldability of the Split Set Stabilizer which makes it unique. It moves with the rock as it deforms but does not lose its holding power. While both the mechanical and resin or cement anchor systems will hold higher initial loads in quick pull tests, they will not yield nearly as much with the rock.

If one examines what happens to interior rock fixtures over their life in the rock mass, a more realistic appraisal can be made of their true worth. Figures 7 and 8 compare a mechanical anchor system with a Split Set Stabilizer system in a geological formation where rock anchors tend to creep.

In Figure 7, a hypothetical case history is presented. Results of several investigators' work is combined to illustrate what could conceivably occur during an actual load history under real conditions, using a standard mechanical fixture. Referring to Figure 7:

A-B Primary creep[7]
B-C Secondary creep[7]
C-D Load increases due to rock fracture and movement[8]
D-E Continued secondary creep
E-F Loss of load due to seismic shock from blasting[9]
F-G Continued secondary creep
G-H Load increases due to rock movement, as for example: a cave line approaching the installation.[8]
H-I Loss of all load and usefulness of fixture if rock breaks from around the plate.

In Figure 8, we see the results of actual field tests on Split Set Stabilizers in various geologic formations. Note that in all cases anchorage increases with time. The rate of anchorage increase, or lock-up, is a function of the need for additional support. For example: Curve A is in a high horizontal stress area where rock tends to shear soon after mining which produces many off-sets or kinks in the Stabilizer.

In Curve C where the Stabilizer is in visco-elastic salt which tends to creep slowly, offsets are not pronounced, but rather, a uniform rate of increase in anchorage is obtained at a slower rate than where pronounced offsets and shearing occur in the high stress area illustrated in Curve A.

FIELD EXPERIENCE[5]

Friction Rock Stabilizers have been used in rocks varying in

# TYPICAL - PULL TEST RESULTS

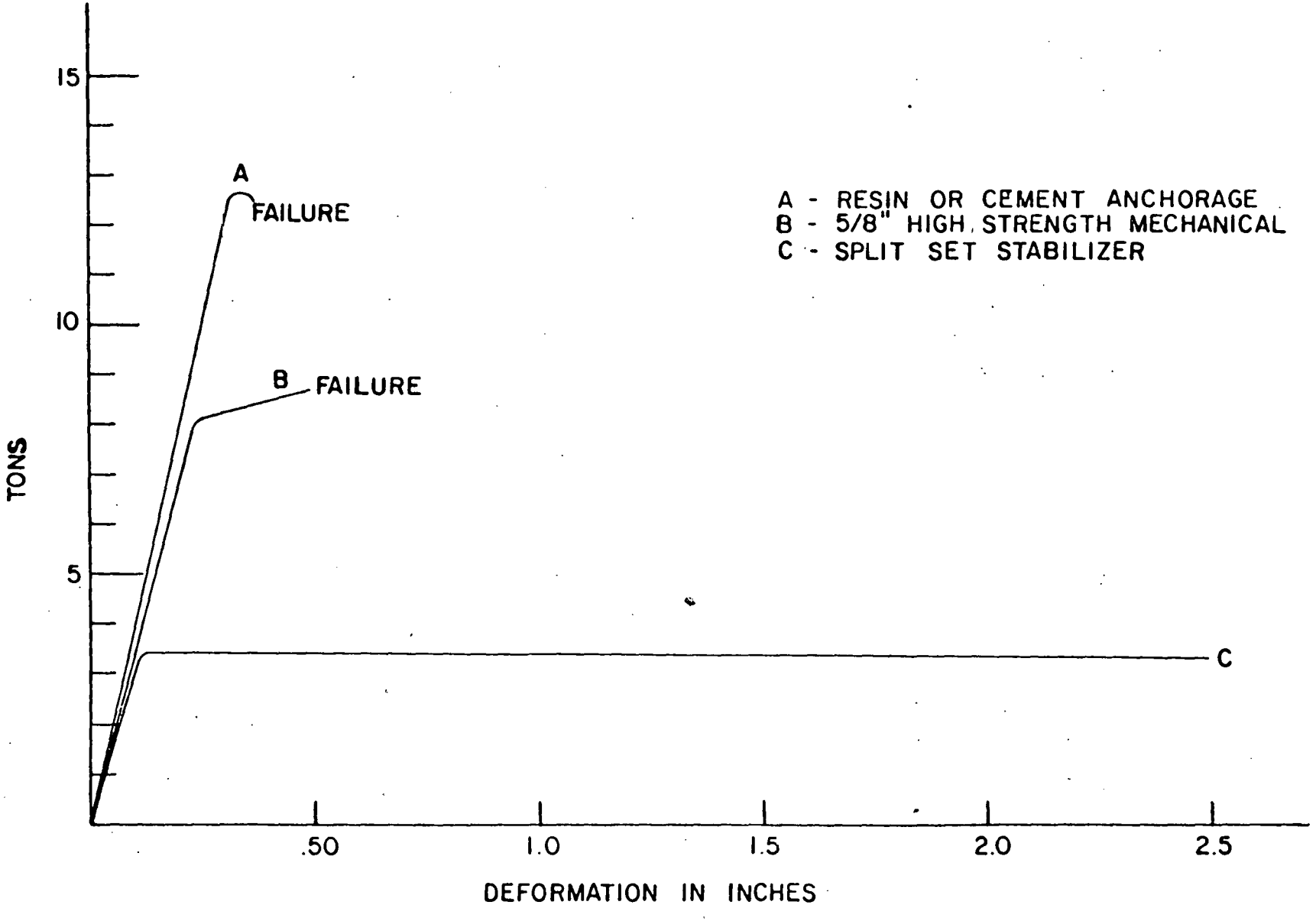



FIGURE 6

# HYPOTHETICAL MECHANICAL BOLT LOAD HISTORY

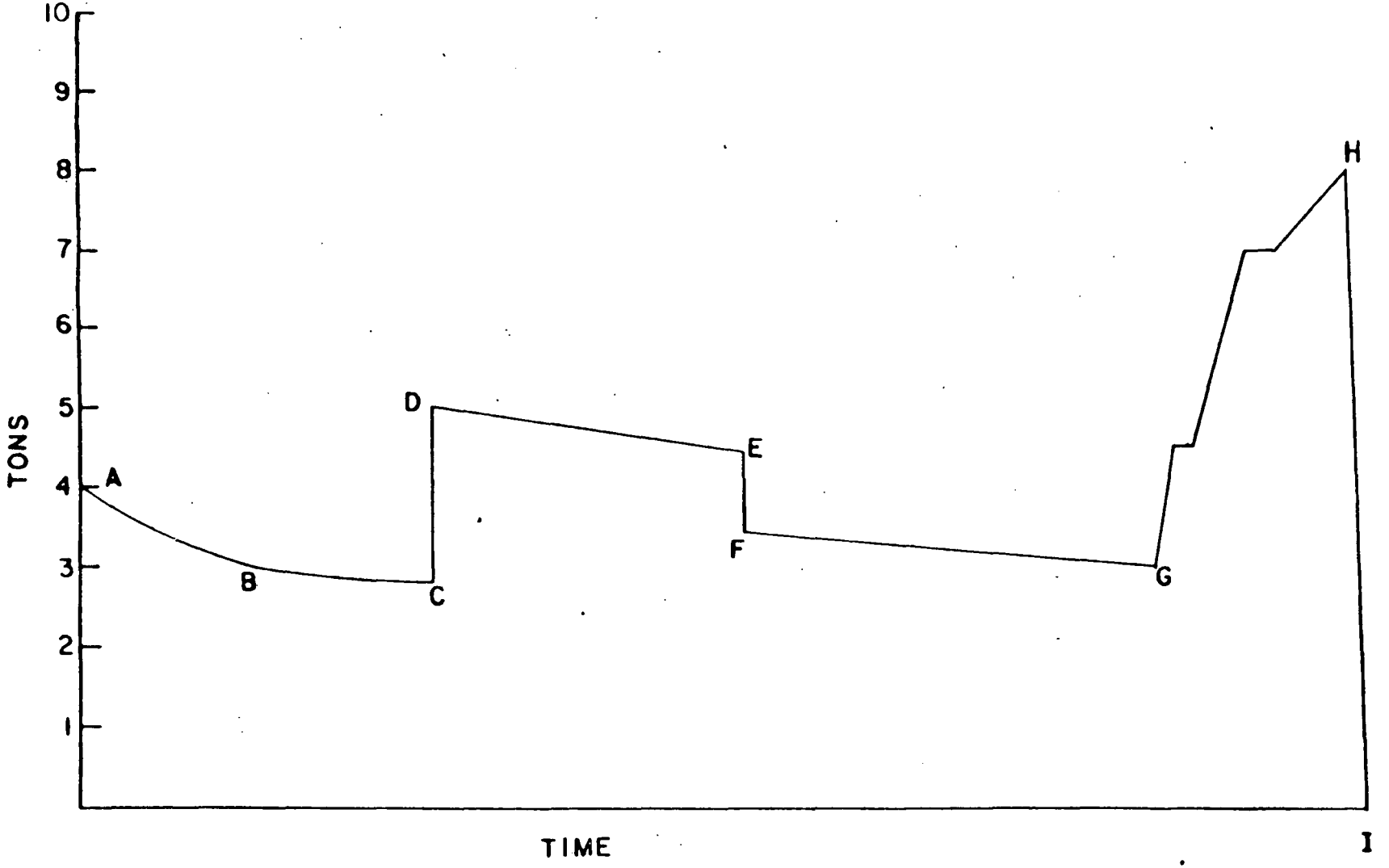



FIGURE 7

# FIELD TEST RESULTS OF PULL TEST ON FRICTION ROCK STABI ZERS IN VARIOUS GEOLOGIC FORMATIONS

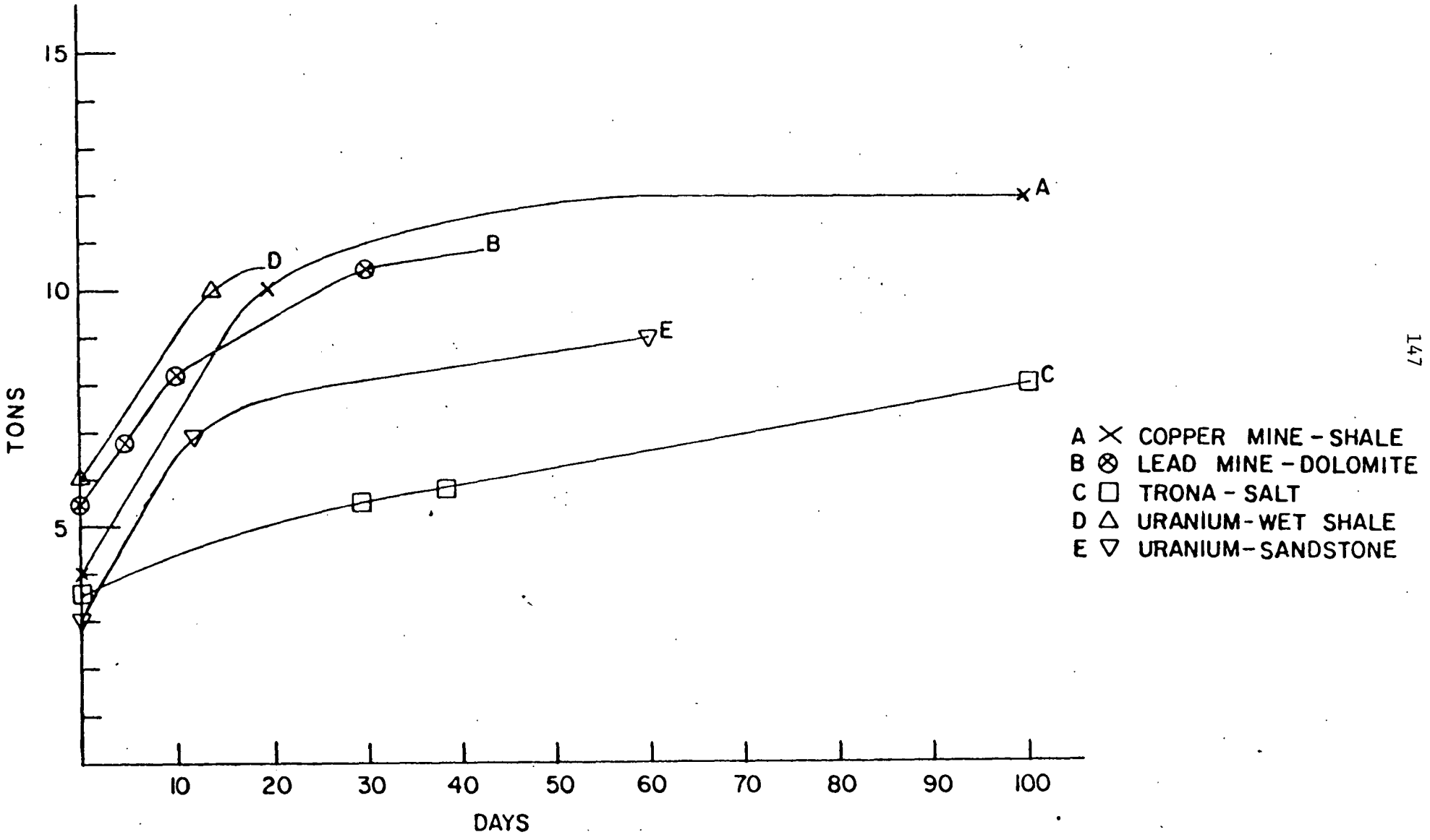



FIGURE 8

strength from 50 psi compressive strength to 60,000 psi. Most field data to date has been in soft formations associated with uranium mining in the United States. Good results have been obtained in the Nonesuch Shale at the White Pine Copper Mine and in the Bonne Terre Dolomite in the lead mines of Missouri, rocks which have compressive strengths near 20,000 psi. Limited documentation of data is available on very hard rock formations such as those found in the Sudbury district of Canada and gold mines in South Africa. The limited data on truly hard rock formations has indicated that performance is excellent.

Following are some case histories where Split Set Stabilizers have worked well and to the advantage of the mine operator:

Case 1. In the Ambrosia Lake area of New Mexico on a deep shaft project, Split Set Stabilizers and wire mesh were used as temporary support for approximately 2 months in a mudstone at a depth of 2800 feet. 5 ft. devices were used in the shaft station in drifts up to 17 ft. wide to allow the opening up of the entire area so that all concrete piers could be poured and steel set and blocked at a later time. Standard mechanical anchor fixtures would not hold tension in this formation. Work was completed without any incidence of ground failure with a major cost saving to the shaft contractor. Studies performed at a later date on the loads on the steel sets showed no loading, indicating that the Split Set Stabilizers and mesh were holding the ground and the steel sets were not loaded.

Case 2. A uranium property in the Powder River Basin of Wyoming has experienced a cost savings of near $200 per foot for development drifts by changing from steel and timber to Split Set Stabilizers and mesh for permanent support. Physical property rock test samples from this mine show that none of the formations test more than 800 psi uniaxial compressive strength and many are as low as 50 psi. It is not uncommon to have soft bentonitic shale squeeze through the 2" x 2" openings in the wire mesh which is used to restrain the surface. Even under these adverse geologic conditions, only a few drifts have experienced failure where Split Set Stabilizers have been used in cycle in the mining operations. All of these failures were directly contributable to encountering water which flowed in and saturated the ground after support placement, changing the rock consistency and strength to a point where failure ensued.

Case 3. A porphyry copper mine in Nevada which used a sublevel caving system and long blast holes, drilled in a fan pattern from sublevels , was having problems with losing access to the longholes due to rock fracture and failure from blasting of previous rings. Mechanical roof bolts with wire mesh and shot-crete were being used for support. A switch to Split Set Stabilizers and mesh solved their problem, eliminating rock breakage around unblasted holes and providing a secure brow. Breakage efficiency increased by approximately 20 per cent in this operation. The rock type was blocky, fractured, altered breccia material.

Case 4. A uranium operator on the Colorado plateau ran a comparative pull test program between standard mechanical roof bolts an Split Set Stabilizers. It was found that near 90 per cent of the roof bolts had anchorages of less than 1 ton while the Split Set Stabilizers consistently showed 3 tons of anchorage upon installation and increased anchorage with time. The increased anchorage seemed to be

a direct function of how much support was needed; that is, the more rock movement, the higher the anchorage became.

Case 5. In the Coeur d'Alene district of Idaho, Split Set Stabilizers have performed well when subjected to rock bursts at depths up to 7900 feet. The rocks are quartzitic in nature with initial anchorages on fixtures of near 3 tons. Anchorages tend to increase with time. In a haulage drift 9 x 10 ft. supported with Split Set Stabilizers and 16 gauge metal mats, with Split Sets placed on approximately 3 to 4 ft. center lines, good support is achieved. When an area of this type is subjected to a medium intensity rock burst, the rock around the drifts fractures but is held in place by the Split Set Stabilizers and mats. Trimming of loose rock and rebolting is sufficient for continued drift useage.

In another case in a shrinkage stope where a Richter 2.0 burst occurred, Split Set Stabilizers held the hanging wall completely intact. At each end of this area the stope was supported by timber and complete failure occurred. Where similar areas have been supported by mechanical roof bolts or timber, such events often necessitated re-mining the area with complete replacement of the support system.

## ROCK MECHANICS DISCUSSION [2]

The previous sections and figures in this paper should serve to demonstrate the uniqueness of the Split Set system; But is it compatible with basic rock mechanics principles?

Ground reaction curves, as developed by Deere and Peck and others[10], Figure 9, indicate that there is little that can be done to prevent primary rock relaxation around an underground opening. Without massive, expensive, pre-loaded supports, this movement cannot be stopped. The practical engineer tries to get support in soon so that primary rock movement loads the support at the same time that the rock is generating interior bending movement and shear stress in an attempt to be self-supporting.

Split Sets are ideal devices to enhance this process. Placed in cycle in a mining operation, they would be present to resist the ground reaction curve but would not provide too much anchorage so as to develop stress concentrations to cause rock breakage around the device. Rather, upon overloading, they will slip a few thousandths of an inch and will again be capable of full load. The rock will continue to generate the maximum in resistance to movement with the compatible Split Set interior support system for assistance. If a stiffer support is needed, the spacing and pattern can be reduced.

With Split Set Stabilizers the rock mechanics engineer becomes truly challenged to determine the reaction process of the geologic material he is attempting to stabilize.

## CONCLUSIONS

Investigations and applications have been carried out to the point where it can conclusively be said that the Split Set system for ground control is a viable system for the mining engineer to consider as a solution to his ground control problems. It should be viewed as one of a family of devices at the rock mechanic engineer's disposal. Standard mechanical rock bolts, resin grouted bolts, cement grouted bolts, trusses, grouted cables, etc., all have their place and it is the author's opinion that Friction Rock Stabilizers, or Split Sets, have reached a point in their development where they should be seriously considered for many ground control problems.

# LOAD REACTION CURVE

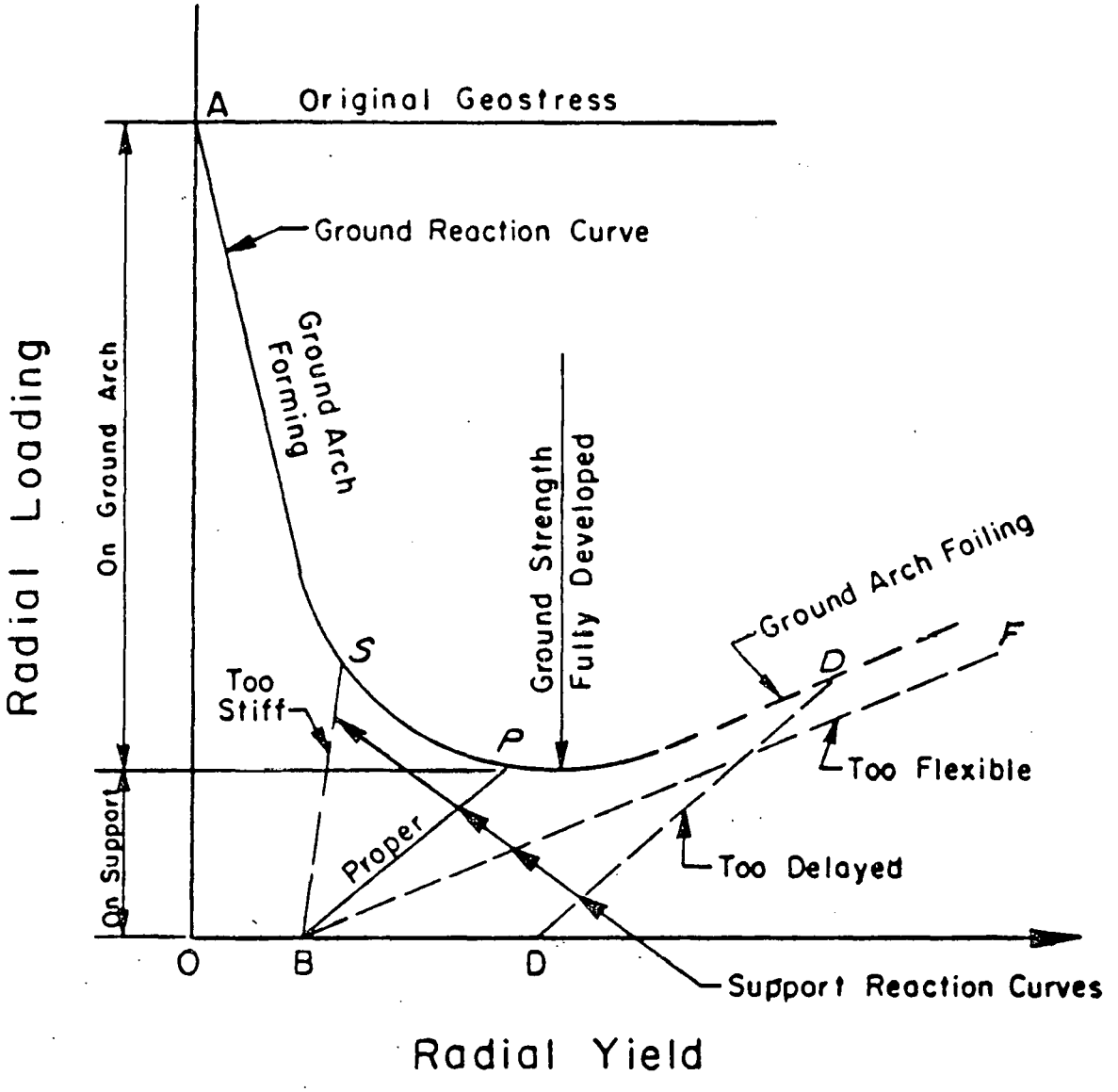



FIGURE 9 Reaction Curve Concept
(adapted from Deere and Peck)

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# SHOTCRETE PRACTICE IN UNDERGROUND CONSTRUCTION

Seymour A. Bortz

Senior Engineering Advisor
IIT Research Institute
Chicago, Illinois

## INTRODUCTION

A major problem in mine safety is that falls of roof, rib, and face account for nearly 60% of the total number of fatalities (about 100 men/year) in underground coal mines. In addition to these fatal accidents, 55 to 60% of the permanent disabilities and approximately 20% of the temporary lost-time accidents are caused by falls of roof rocks. Almost 70% of these accidents occur within 25 ft of the working faces, partly because the immediate roof in these regions is inadequately supported, and partly because the greatest concentration of working personnel is in this area.

Currently, several types of temporary and permanent roof supports are used to prevent the caving in of the roof and sides of mine roadways. Mine supports include systems such as timber posts or sets, hydraulic or friction props, and screw jacks. A post-World War II development is the use of roof bolts. At present, nearly 70% of the nation's underground coal is produced from roof-bolted areas of mines. Unfortunately, all of these tenporary mine supports possess deficiencies for use near the working face because of the erection time involved and the exposure of personnel to rock fall.

Shotcrete has been used effectively in hard rock tunneling for nearly 20 years. It was originally developed in Europe and had since been adopted in Japan, Hong Kong, South America, and finally in the United States. Shotcrete is defined to include pneumatically applied mortar and concrete. Pneumatically applied mortar (Gunite) has been in use as an overlay material for at least 50 years in the United States. It has not been found capable of providing a dependable support in underground excavations, because it has a tendency to loosen as rock surfaces relax, and spall with the greater degree of relaxation of the more incompetent rock areas and members. This could be due to the thickness of individual applications which can vary ±1 in., aggravated by shrinkage induced by a high cement content.

Spray concrete, on the other hand, bond effectively with most rock surfaces, and applied with skill can suppost even a cohesionless soil. Its adhesion to cohesive surfaces is attributed to the peening effect of the coarse aggregate driving its predecessors into the subject surface, and to the high early strengths reached with the aid of a suitable accelerator. It can be applied in layers 2 to 3 in. thick in one pass, thus achieving its supporting function in addition to a seal.

Coarse aggregate shotcrete has proved itself as a support medium over a broad spectrum of difficult underground situations. Used alone or with rock bolts, steel arch supports, or light steel reinforcement as circumstances require, the technique has been found capable of stabilizing virtually all conditions encountered in tunneling. These have included intensive fractured wet rocks; plastic, water-bearing and sealing marls; and cohesionless gravel (Fig. 1).

## PROCESS AND EQUIPMENT

Shotcrete, a pneumatically sprayed concrete, offers one solution to the problem of coal mine ground control. In the shotcrete system, concrete is deposited on a prepared surface with considerable force by pneumatic means. It can be supplied to vertical, overhead, or horizontal surfaces, and is usually deposited in layers; various thicknesses can be obtained by building up several layers. Adhesion between successive layers enables the layers to be regarded as integral with one another. The process is used for lining structures to make them watertight; for repairing concrete which has disintegrated; for linings around dams, mines, and tunnels; and for sealing rock seams and fissures.

The two basic shotcreting processes referred to are the dry mix and the wet mix process. The two methods are outlined:

### Dry Mix Process

1. Cement, damp sand (4-8% moisture), and coarse aggregate are thoroughly mixed. Powdered fastsetting admixture (FSA) is introduced at this stage (Fig. 2).

2. The mixture is fed into a mechanical feeder or gun (Figs. 3 and 4).

3. Material is carried by compressed air through the delivery hose to a nozzle. The nozzle is fitted with a perforated manifold through which water is introduced under pressure and intimately mixed with the other ingredients, Fig. 5. Liquid FSA is introduced at this stage. Fig. 6.

4. Shotcrete is jetted from the nozzle at high velocity onto the surface to be shotcreted, Fig. 7.

### Wet Mix Process

1. Ingredients, including water, are thoroughly mixed.

2. The plastic concrete is introduced into delivery equipment chamber, Figs. 8, 9, and 10.

3. The mix is metered into a delivery hose and conveyed by compressed air or positive pressure to the nozzle.

4. Fast-set admixture is added to the wet mix prior to the nozzle, Figs. 11 and 12.

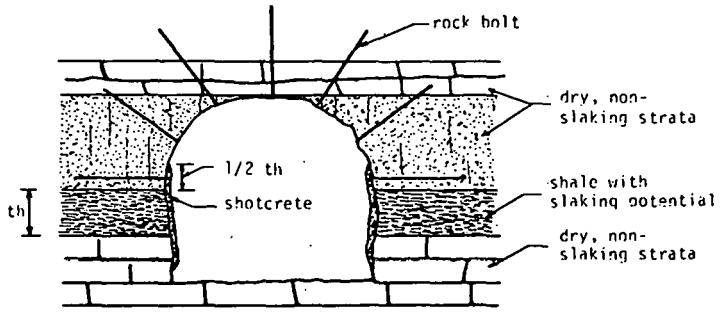
5. Additional air can be injected at the nozzle to increase the velocity, which improves compaction, of the shotcrete,[1] Fig. 13.

6. Shotcrete is jetted from nozzle at high velocity onto surface to be shotcreted.

Shotcrete suitable for normal construction requirements can be produced by either process. Differences in operational features which may merit consideration are given in Table 1.

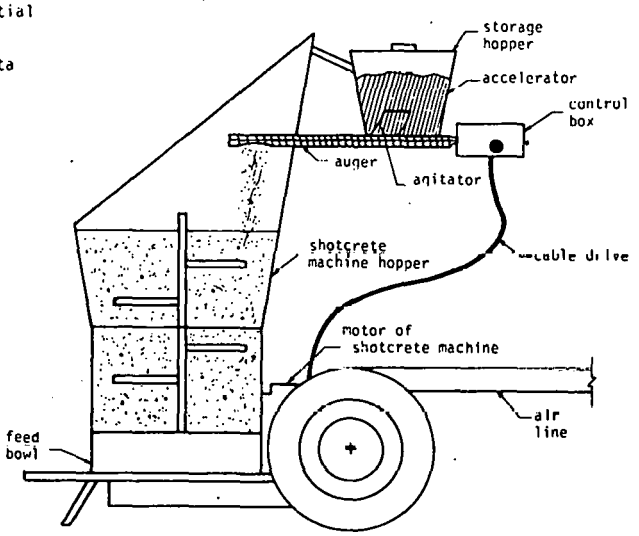
The length of hose used with shotcreting is normally 50 to 150 ft. However, greater lengths of up to 450 ft and, under exceptional conditions, up to 1,000 ft can be used if necessary. The maximum vertical rise above the gun is 200 ft, but a much lower height than this is desirable. The hose for delivering the cement-aggregate mixture is normally 1-1/4 or 1-1/2 in. internal diameter and a 1/2 or 5/8 in. hose can be used to supply water.

Operating pressures are normally between 30 psi and 60 psi, depending on



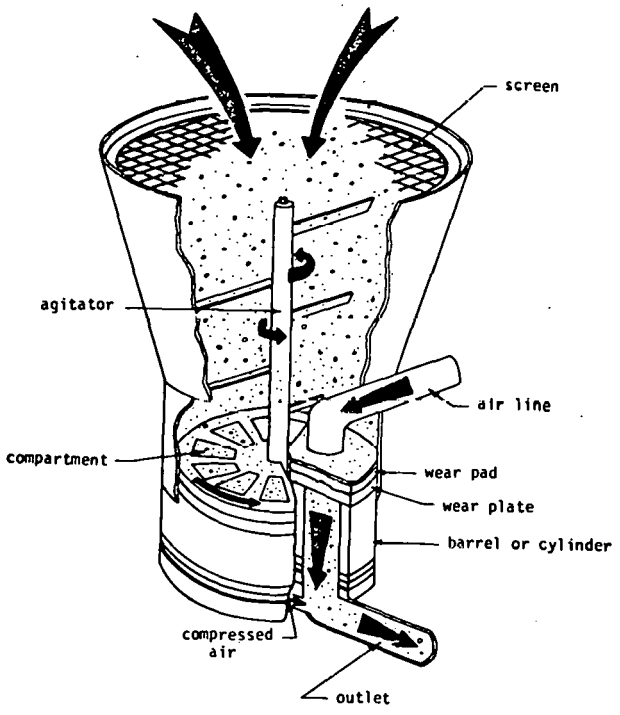

USE OF SHOTCRETE TO PREVENT SLAKING CAUSED BY STRESS RELIEF AND DRYING

Figure 1



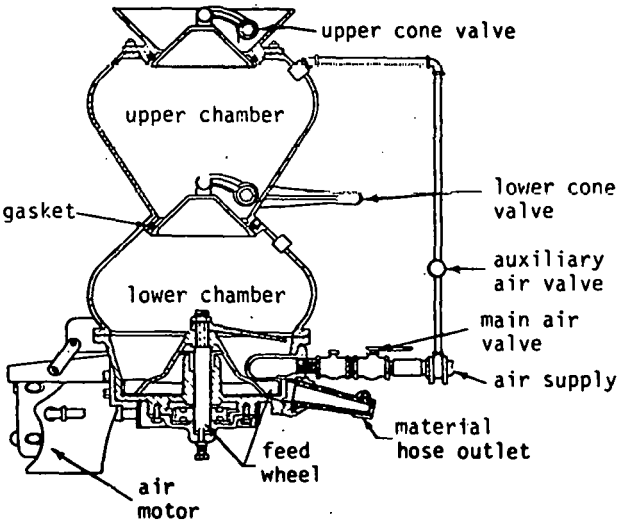

ACCELERATOR FEED TO MATERIALS IN SHOTCRETE MACHINE HOPPER

Figure 2



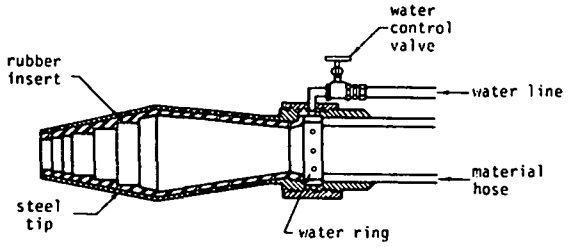

ROTATING BARREL DRY-MIX SHOTCRETE MACHINE

Figure 3

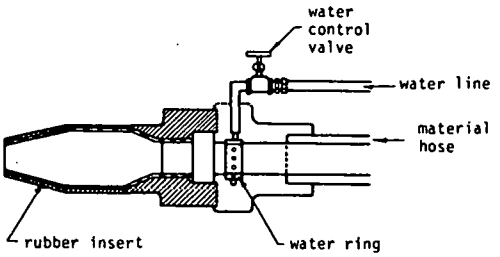



FEED WHEEL SHOTCRETE MACHINE

Figure 4



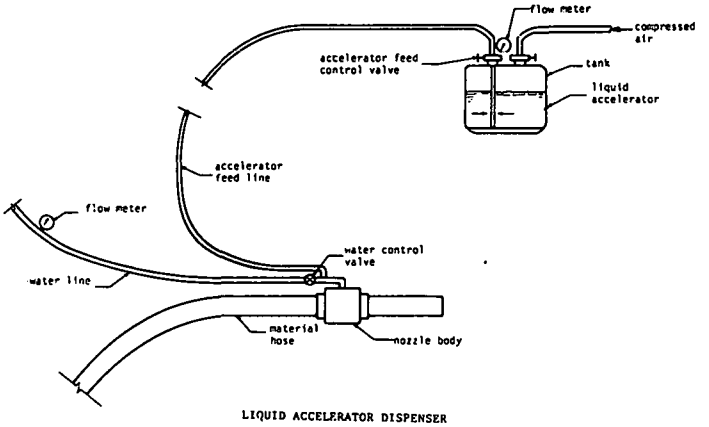

a. Stepped-balloon nozzle




b. Smooth-balloon nozzle

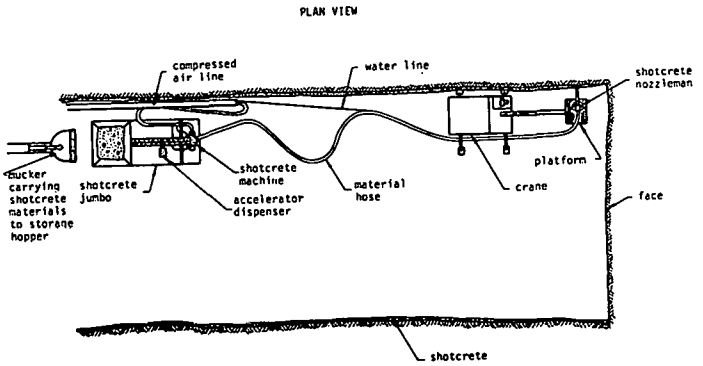
STEPPED-BALLOON AND SMOOTH-BALLOON DRY-MIX SHOTCRETE NOZZLES

Figure 5



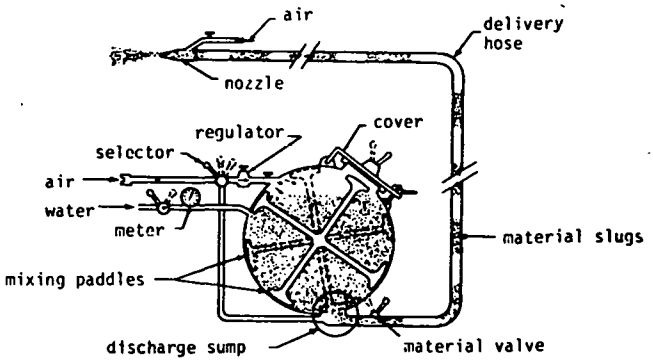

LIQUID ACCELERATOR DISPENSER

Figure 6



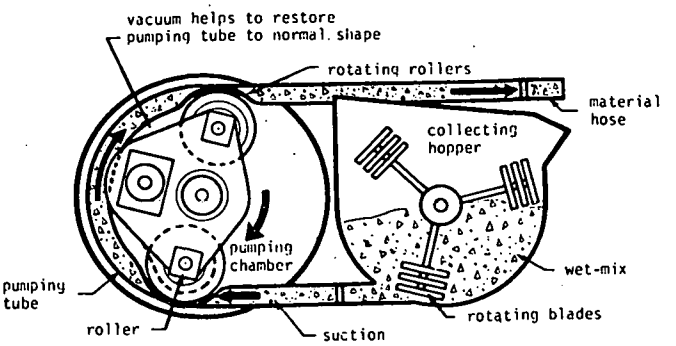

SETUP OF SHOTCRETE EQUIPMENT IN UNDERGROUND CONSTRUCTION

Figure 7



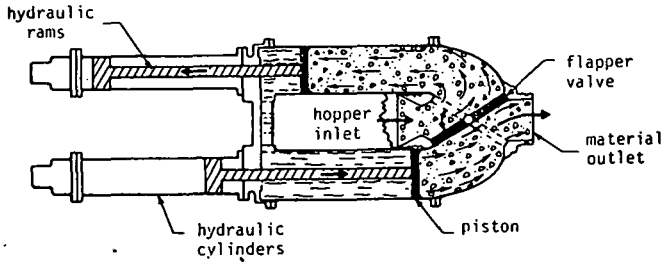

SCHEMATIC DIAGRAM OF THE PNEUMATIC WET-MIX SHOTCRETE MACHINE

Figure 8



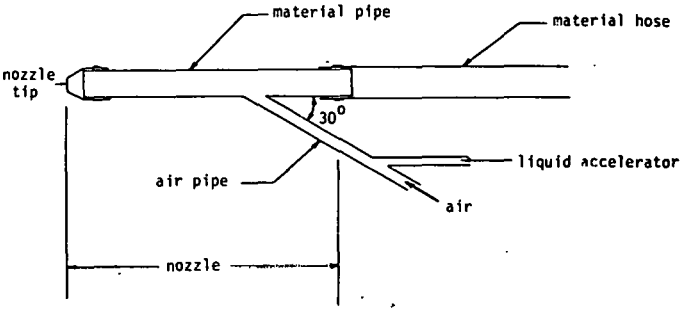

SCHEMATIC DIAGRAM OF SQUEEZE TYPE, WET-MIX SHOTCRETE MACHINE

Figure 9




SCHEMATIC DIAGRAM OF PISTON TYPE, WET-MIX SHOTCRETE MACHINE

Figure 10




WET-MIX SHOTCRETE NOZZLE FOR USE WITH LIQUID ACCELERATORS

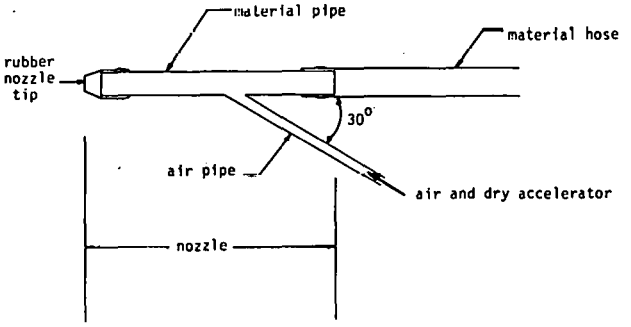
Figure 11

Table 1

OPERATIONAL FEATURES OF DRY AND WET MIX SHOTCRETE PROCESSES

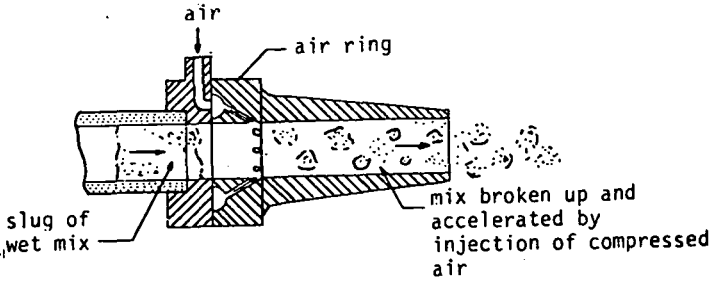
| |
|---|
| **Dry Mix Process** |
| 1. Control over mixing water and over consistency of mix at the nozzle |
| 2. Better suited for placing mixes containing lightweight porous aggregates |
| 3. Capable of longer hose lengths |
| **Wet Mix Process** |
| 1. Mixing water is controlled at the delivery equipment and can be accurately measured. |
| 2. Better assurance that the mixing water is thoroughly mixed with other ingredients. This may also result in less rebound and waste. |
| 3. Less dust accompanies the gunning operation. |

From: "Recommended Practice for Shotcreting," ACI 506 (1966).




AIR TUBE NOZZLE FOR USE WITH POWDER ACCELERATORS

Figure 12




AIR RING NOZZLE FOR WET-MIX SHOTCRETE

Figure 13

the size of the gun. For long horizontal lengths of hose and for vertical lifts higher pressures and a booster jet is required, so that the nozzle velocity of the material is not reduced. Too high or too low pressure will adversely affect the quality of shotcrete and the quantity of rebound produced.

The volume of free air required under normal gunning conditions is 60-225 $ft^3$/min. The water supply to the nozzle should be 15 psi above the air pressure. For good even work, it is important that the pressures selected be maintained at a steady value.(2) Table 2 provides background regarding the interrelations between operating pressure, compressor capacity, hose diameter and nozzle size. The coverage for a given quantity of material cannot be given with precision, but a rough guide is given in Table 3.

The nozzle should be held 3 to 4 ft from and perpendicular to the surface being sprayed and should be kept moving all the time or uneven work will result (Figs. 14 and 15).

Generally, 2 to 3 in. may be deposited in one operation on vertical or gently sloping surfaces when applying shotcrete from top to bottom, but care must be taken to avoid formation of sand pockets as the rebound material does not fall clear of the work as it does with overhead surfaces (Fig. 16). Actual thickness depends on mix design and concentration of FSA.

On overhead surfaces about 2 in. may be the maximum thickness that can be deposited on one pass, again dependent on mix design and concentration of FSA. If greater thickness of material is required, buildup can be accomplished by successive applications, the previous layer being allowed to set before application of the subsequent layer. A thickness of 2 in. is usually sufficient for waterproofing purposes.

In order to make shotcrete more effective for use neat work facings in coal mines, a fast setting time is necessary to reduce interference with the mining operation, obtain rapid strength, stop seepage in coal and rock seams, and to facilitate the placing of bulkheads to seal off mine fires and control explosions. The required rapid set can be obtained through the use of special rapid set cements. Fast set shotcrete properties and methods of application have considerable advantages and potential for use in underground mining:

1. It can be applied almost immediately behind coal cutting operations. This will permit little movement of the roof prior to receiving support.

2. It can act as a sealant which is important in the prevention of swelling or sloughing of shales, one of the causes of room falls.

3. It can also be used under wet conditions.

4. It will limit gradual deterioration of pillars by crushing.

5. It provides continuous support.

6. It may no longer be necessary to leave roof coal, even under poor roof conditions, thus increasing coal recovery.

7. Its thickness can be readily altered to suit underground conditions.

Shotcrete has been observed to develop strengths as high as 10,000 psi after 7 days with a rich mix. However, for the most part, strength is reported in the range of 3000 to 7000 psi, comparable to normal placed concrete.

A well designed and placed shotcrete obeys the normal water/cement ratio strength laws and has the same properties as very high quality concrete: (1) low permeability; (2) good resistance to weathering and chemical attack; and (3) good resistance to abrasion.

A more recent development in shotcrete technology is the development of steel fiber shotcrete. Since its introduction in 1971, steel fiber shotcrete has found

Table 2

COMPRESSOR CAPACITIES
FOR NORMAL OPERATING CONDITIONS

| Compressor Capacity, $ft^3/min$ | Hose Dia., in. | Maximum Size of Nozzle Tip, in. | Operating Air Pressure Available, psi |
|---|---|---|---|
| 250 | 1 | 3/4 | 40 |
| 315 | 1-1/4 | 1-1/4 | 45 |
| 365 | 1-1/4 | 1-1/4 | 55 |
| 500 | 1-5/8 | 1-1/2 | 65 |
| 600 | 1-3/4 | 1-5/8 | 75 |
| 750 | 2 | 1-3/4 | 85 |

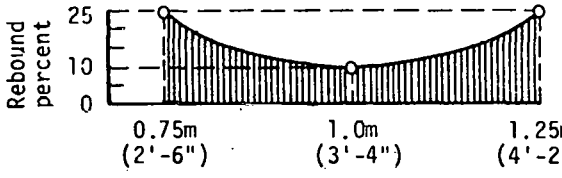
From: "Recommended Practice for Shotcreting," ACI 506 (1966).

Table 3

COVERAGE OF GUNITE FOR STATED
QUANTITIES OF MATERIAL

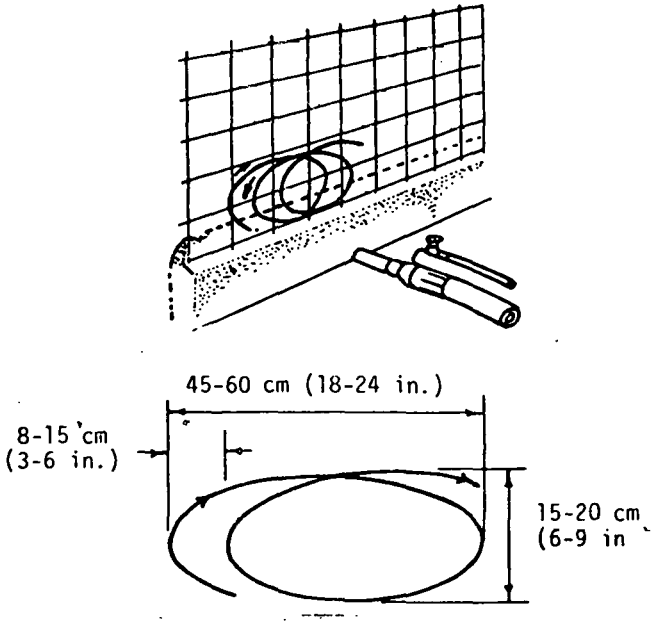
| Mix Proportions, cement:sand | Material Actually Deposited per Bag of Cement, $ft^3$ | |
|---|---|---|
| | Horizontal Surfaces Working Down | Vertical and Overhead Surfaces |
| 1:2½ | 1.5 | 1.3 |
| 1:3 | 1.9 | 1.6 |
| 1:4 | 2.3 | 2.0 |

From: D. F. Orchard, Concrete Technology, Vol. 2, John Wiley & Sons, New York (1962).



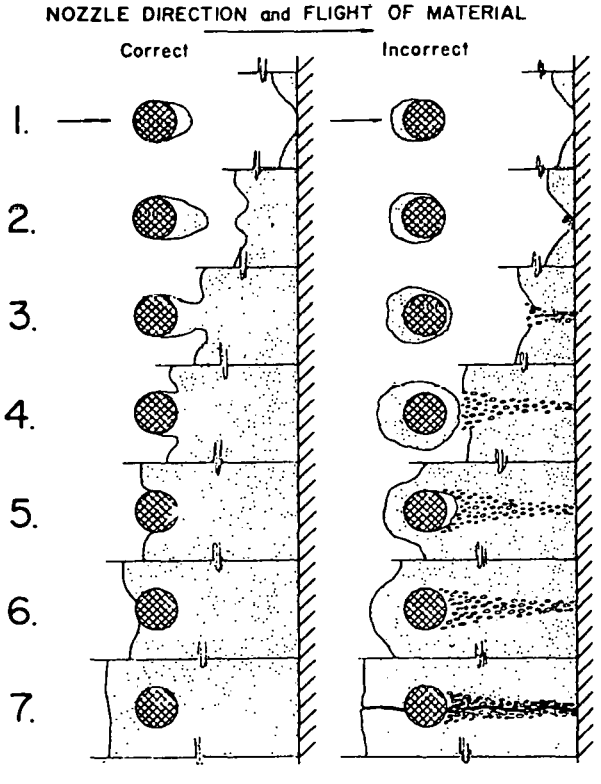

THE EFFECT OF NOZZLE DISTANCE ON REBOUND

Figure 14




SHOTCRETE NOZZLE MOTION

Figure 15




CORRECT (LEFT) AND INCORRECT (RIGHT) METHOD OF ENCASING REINFORCING BAR WITH SHOTCRETE

Figure 16

many interesting and practical uses. One reason for the increasing popularity of this material is its potential to lower the in-place cost of shotcrete by eliminating wire mesh. This is possible because the improvements in properties over those of plain shotcrete are often great enough for fiber shotcrete to do the job without mesh. These improved properties--higher flexural strength, greater ductility, more toughness and higher post-cracking strength--work to especially good advantage in thin-shell applications, mine and tunnel linings and general ground support.

For the dry-mix process a fiber feeder and special nozzle have been developed, Figs. 17 and 18. The fibers are transported from the fiber feeder to the nozzle in a stream of air. To allow introduction of the fibers into the pressurized shotcrete stream, some of the excess air in the cement and sand mixture is bled off before it leaves the nozzle. This type of equipment allows use of longer fibers with higher aspect ratios --that is, length-to-diameter ratios up to about 125--and eliminates the balling that sometimes occurs when fibers are introduced into the shotcrete mix beforehand.

Another innovation for the introduction of steel fibers has been made by Besab. It is a nozzle for the wet-mix process which allows addition of the fibers at the nozzle as shown in Fig. 19. (With the wet-mix process, rebound of shotcrete materials can be reduced to less than 5 percent.) This nozzle has been used to place wet-mix steel fiber shotcrete in a number of applications including underground water tunnels and sewers.

## PROCEDURE

### Mix Preparation

The mix proportions, gradation and quality of aggregate, and condition of equipment and quality of workmanship will effect the quality of shotcrete in place. A preliminary series of test runs must be made prior to the start of the work to check the mix and equipment and to verify quality of the shotcrete that may be expected in the structure.

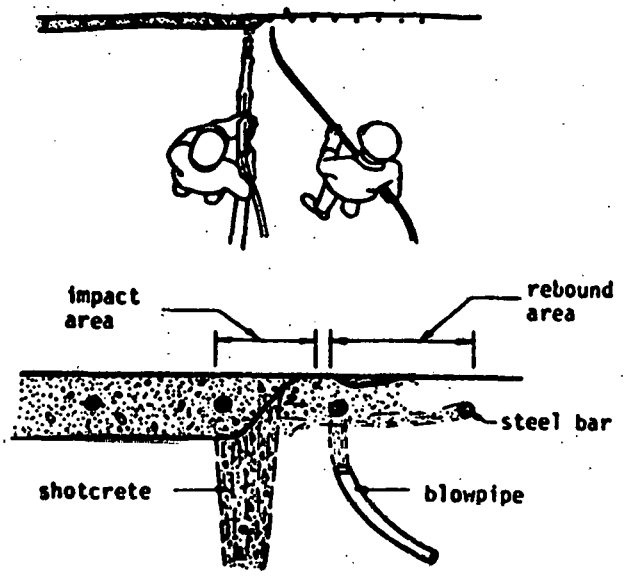
### Fast Setting Mixes

For use as a ground control mechanism in mines, shotcrete must be made to set up quickly with rapid gain in strength so as to effectively support the tunnel and resist the forces from the surrounding strata of rock. The setting time of concrete can be controlled either through the use of admixtures, by special grinding of the cement used, or by new techniques for manufacturing cement.

Where early set or rapid strength development is required, accelerating admixtures are used under certain conditions. Fast setting additives (FSA), in amounts of approximately 2-6% by weight of cement, have been used for this purpose; however, care must be taken when these are used because of strength losses in the concrete. However, various commercial admixtures have been developed which are found to be useful in accelerating the set of shotcrete. Standard admixtures generally found advantageous in regular concrete usually are not found to be useful in shotcrete.

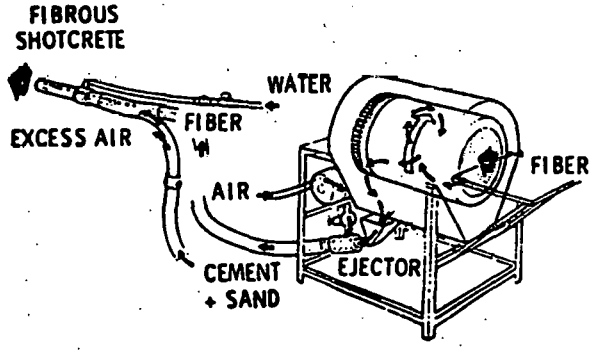
Most additives presently used in the United States are imported from Europe. While all of these are proprietary and no ingredients are listed, they all generally claim to be calcium chloride free. These accelerators are available either in a dry or liquid form, and are used for either the wet or the dry process. Actual proportions should be determined at the site with the materials, mix and temperature prevailing. The differences between cements and even different lot numbers of the same company's cement can vary enough to cause changes in the reaction with the accelerator. So, for accurate setting time control, trial batches must be made to establish the required mixing conditions.

A recent development in the Portland cement field has been the discovery that controlled grinding of Portland cement clinkers will produce cement with controlled setting time.[4] Figure 20 indicates the rapid and early strength obtained with controlled grinding of Type I cement. Regulated Set Portland Cement (RSPC) used in this program is manufactured



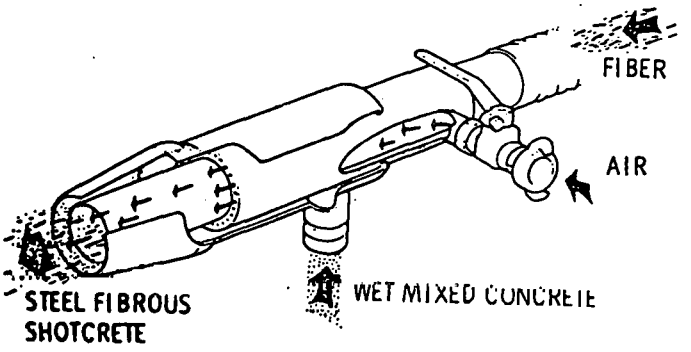

USE OF A BLOW PIPE IN REMOVING REBOUND FOR PROPER ENCASEMENT OF REINFORCING BARS

Figure 17



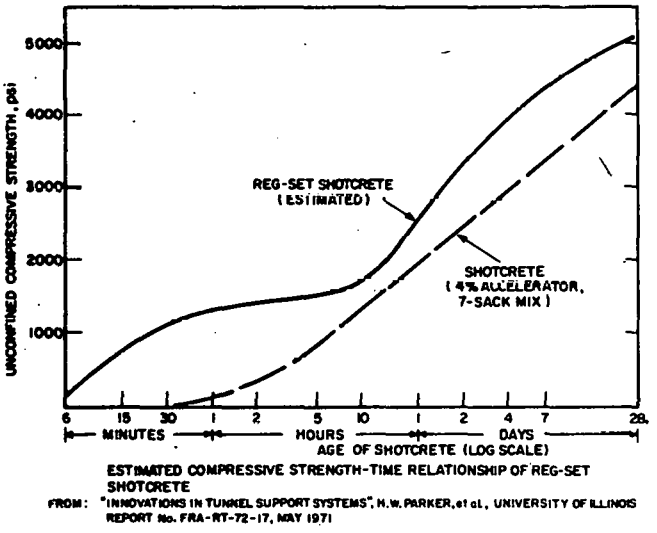

BESAB FIBER FEEDER

Figure 18




BESAB NOZZLE FOR WET PROCESS FIBROUS SHOTCRETE

Figure 19




ESTIMATED COMPRESSIVE STRENGTH-TIME RELATIONSHIP OF REG-SET SHOTCRETE

FROM: "INNOVATIONS IN TUNNEL SUPPORT SYSTEMS", H.W. PARKER, et al., UNIVERSITY OF ILLINOIS REPORT No. FRA-RT-72-17, MAY 1971

Figure 20

with 1-2% fluorite ($CaF_2$) addition as a mineralizer. In this way, and with heated mix water and/or addition of soda ash, the setting time can be controlled with reasonable precision.

## Application

The most essential component of good shotcrete application is a well-trained, experienced crew who has the desire to do a good job. Training and experience are more important to shotcrete work than it is to cast-in-place concrete work for two reasons. First, the shotcrete process contains more variables than the cast-in-place concrete process. Second, there are a greater number of human decisions to be made in its placement. There is an element of art and a degree of skill involved in shotcrete which cannot be overlooked. Many shotcrete failures can be traced to poor application practices. These result mainly from the crew's lack of knowledge or expertise. The nozzleman in particular must be highly trained and he must possess the desire to do a good job. He controls the quality of the finished shotcrete product as well as influencing the economy of the job. High quality and economy go hand in hand when proper application procedures are followed by the nozzleman and his crew (Fig. 21).

The first step in the shotcrete application process is the preparation of the receiving surface. Earth, rock, concrete, masonry, wood and steel may be coated with shotcrete. It is of prime importance that they be prepared properly if good bond is to be obtained. Surface preparation is followed by the mixing, shooting, trimming, finishing and curing steps. All of these are individually important and neglecting any one can adversely affect the properties of the finished shotcrete. Proper attention to each of these will insure a produce with all the bond, strength, and durability that is characteristic of well-placed shotcrete.

## Rebound and Coverage

The high velocity of the shotcrete at the time of impact with the surface to be covered causes a fair amount of rebound, and more than 50% of the material may be lost in this way. The amount of rebound depends on the water/cement ratio and the direction of spray. It is less for higher water-cement ratios and more for vertical and overhead work than for horizontal and sloping surfaces. The rebounding material consists of coarse sand particles and aggregate and smaller amounts of cement. All rebound material should be cleaned off surfaces to be covered (Fig. 17), and the sequence of work should be such that any rebound material will fall clear of surfaces to be covered. For this reason, vertical surfaces should be shot from the bottom up (Fig. 22).

Rebound can be reduced if a pozzolan is added to make the mix more plastic without adding water.[3] Also, rebound can be reduced if the grading of the sand is not too coarse and the finest grading is within the limits set up in Table 4.

A mix of one part of cement to 3½ parts of aggregate gauged by loose dry volumes is used for almost all purposes. For work under dry conditions or where great strength and impermeability are not required, a 1 to 4 or 1 to 4½ mix may be used. The mix proportions of the concrete, as deposited, will be slightly richer than described due to rebound of sand and aggregate, and for this reason a mix richer than 1 to 3 should not be used. A 1 to 3-1/2 mix may, after it is deposited, become 1 to 3 or 1 to 3-1/4. Aggregate size normally will gange from 1/4 to 3/4 in. in size. Suitable gradings are shown in Table 4.

The sand should not be absolutely dry but preferably will have a moisture content between 4% and 8%. The quantity of water added is controlled by a tap at the nozzle and is determined by the judgment of the operator. Too much water will cause the shotcrete to run down or slump on vertical surfaces, or drop down from overhead surfaces. If the mixture is too dry, it will be weak and will not adhere properly. The correct quantity of water usually causes a sheen to appear on the finished work.

Strength tests on shotcrete samples are the most commen means for checking shotcrete quality. These tests give an

Table 4

GRADING OF AGGREGATE FOR USE IN SHOTCRETE

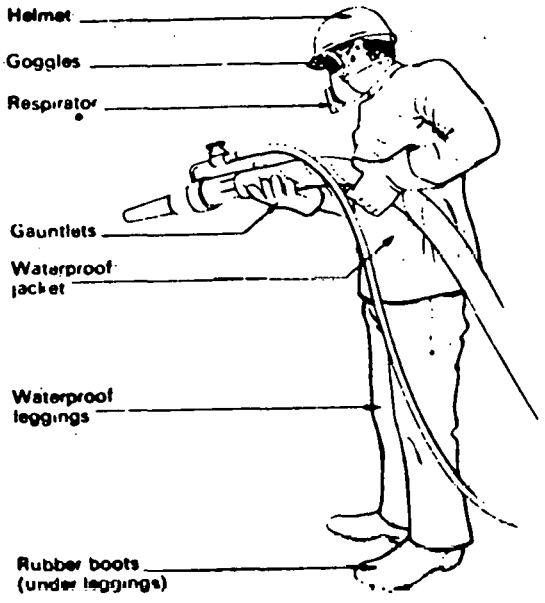
A. Fine Aggregate

| Sieve Size, U.S. standard square mesh | Percent Passing, by weight |
|---|---|
| 3/8 in. | 100 |
| No. 4 | 95-100 |
| No. 8 | 80-90 |
| No. 16 | 50-85 |
| No. 30 | 25-60 |
| No. 50 | 10-30 |
| No. 100 | 2-10 |

B. Coarse Aggregate

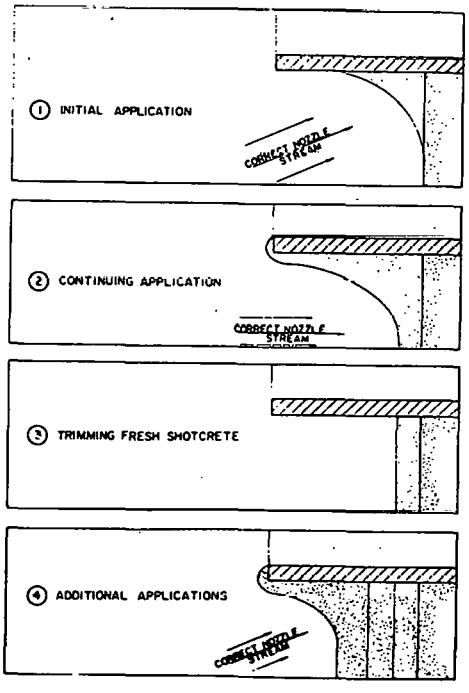
| Sieve Size, U.S. standard square mesh | Percent Passing Individual Sieves, by weight | | |
|---|---|---|---|
| | No. 8 to 3/8 in. size | No. 4 to 1/2 in. size | No. 4 to 3/4 in. size |
| 1 in. | -- | -- | 100 |
| 3/4 in. | -- | 100 | 90-100 |
| 1/2 in. | 100 | 90-100 | -- |
| 3/8 in. | 85-100 | 40-70 | 20-55 |
| No. 4 | 10-30 | 0-15 | 0-10 |
| No. 8 | 0-10 | 0-5 | 0-5 |
| No. 16 | 0-5 | -- | -- |

From: "Recommended Practice for Shotcreting," ACI 506 (1966).




THE WELL-DRESSED NOZZLEMAN

Figure 21




ALTERNATE METHOD OF SHOOTING A FORMBOARD OR ANY WALL PROJECTION. AS EACH LAYER IS APPLIED, THE MATERIAL ON THE PROJECTION IS CUT BACK FLUSH WITH THE PLANE OF APPLICATION EACH SUCCEEDING APPLICATION BEGINS IN THE CLEANED CORNER

Figure 22

indication of both the load-carrying capacity and overall quality of the in-place shotcrete. Strength is a good index of the manufacture, placement, and most of the properties of the resulting shotcrete, such as durability. The results obtained from strength tests are sensitive to sampling, methods of testing, and actual test procedures with differences of more than 50% compression being obtained. Test procedures consist of flexure, pullout (Fig. 23), rebound hammers (Fig. 24), and penetration tests (Fig. 25). Figure 26 shows samples which are representative of shotcrete cut from a test panel. It is mandatory that samples receive the same curing as that of the in-place shotcrete.

## CONCLUSION

Some of the most important factors in the effective use of shotcrete for underground support are the practical considerations relating to application. These involve the techniques of application in order to establish a stable support system in a practical manner.

It must be understood that shotcrete is not the only answer to every ground condition needing support. It is only a tool to be used where practical and advantageous. Questionable past results using shotcrete for ground support may be related to lack of knowledge and practical experience as to equipment performance, application techniques, and materials design. It is the aim of this paper to provide some of the necessary background so that the designer and contractor may use shotcrete in a safe, useful, and economical manner for underground applications.

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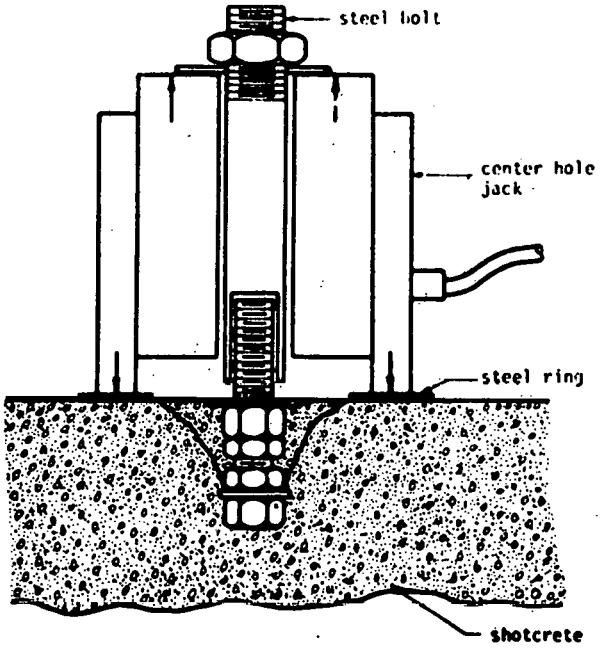
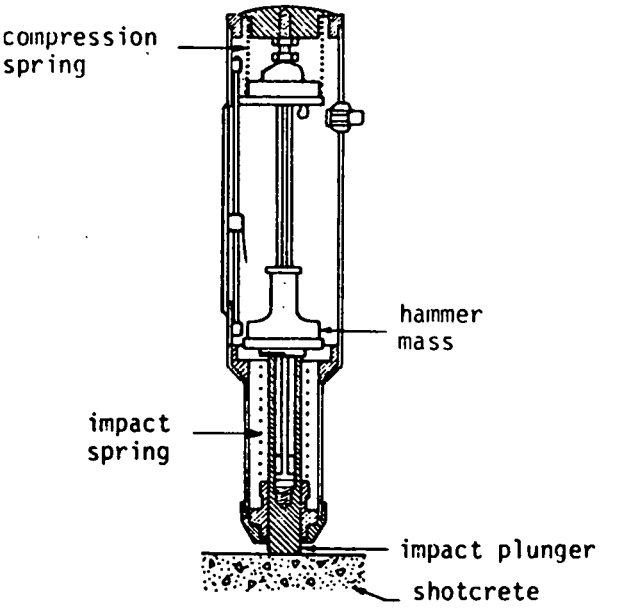
6. Ibid, Ref. 2, p. 428.



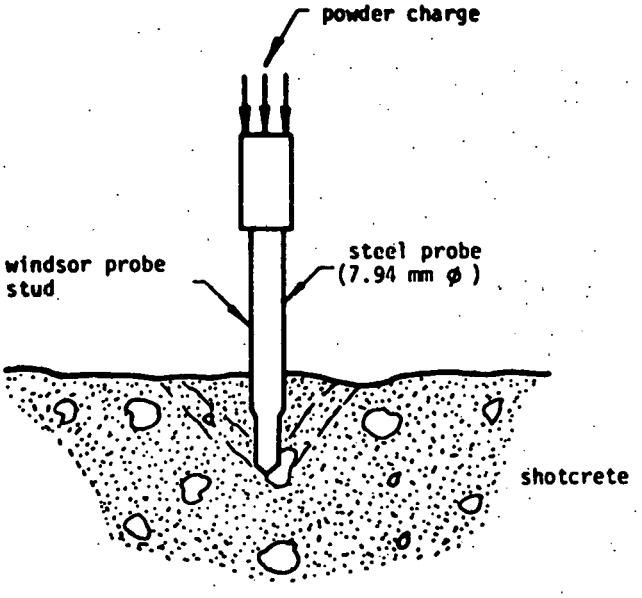

PULL-OUT TEST ON SHOTCRETE

Figure 23



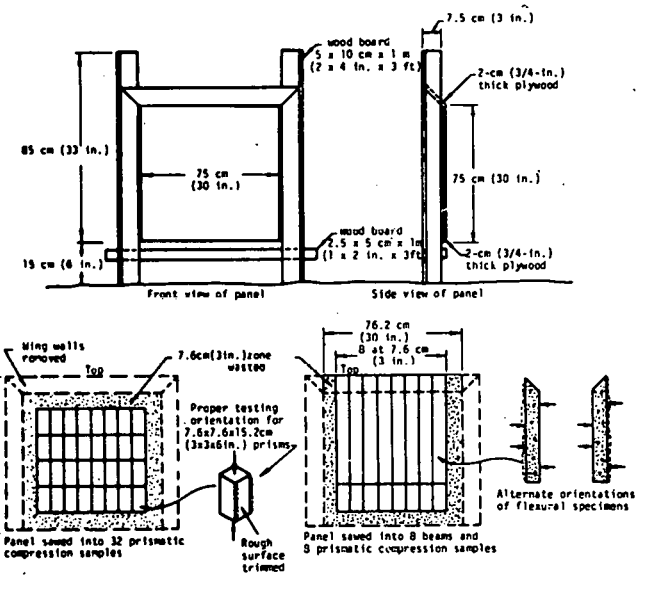

TYPE L SCHMIDT TEST HAMMER

Figure 24




WINDSOR PROBE TEST

Figure 25




TEST PANEL METHOD FOR OBTAINING PROPERLY-OEIENTED TEST SPECIMENS

Figure 26

# SESSION IV

# LONGWALL MINING

Session Co-Chairpersons:

Mr. P. Conroy, Dames and Moore, Chicago, Illinois

Mr. J. Janes, Old Ben Coal Company, Marion, Illinois

Dup

# DESIGN OF LONGWALL MINING SYSTEMS

Ernest A. Curth

Mining Engineer
United States Department of Energy
Pittsburgh Mining Technology Center

## INTRODUCTION

The Illinois coal basin contains significant reserves and is one of the important coal producing provinces in the United States in close proximity to the consumers. Room-and-pillar methods are prevalent in Illinois with the result that the average recovery of coal approximates 50 percent and roof control is difficult. The alternative is longwall mining with the potential of better ground control, generally easier compliance with safety standards, improved productivity and higher resource recovery. However, attempts at longwall mining the Herrin No. 6 coalbed were unsuccessful in the past (1). If problems such as instability at the face and hazardous ground control are solved, longwall mining could be reconsidered.

The Department of Energy's Coal Mining Research Program aims to lower the economic and social cost of coal mining by developing full extraction systems such as longwall mining. Problems in design and operation of safe and efficient faces are to be solved by improved technology. In line with this objective the Department of Energy became a partner in a cost-sharing endeavor with Old Ben Coal Company, to demonstrate that the Herrin No. 6 coalbed can be mined safely by longwall methods.

The design of the Old Ben longwall operation emphasized risk reduction and personnel safety and included the following:

1. A premining study.
2. Choice of system.
3. Panel design.
4. Environmental aspects.
5. Equipment selection.
6. Coal clearance and other services.
7. Monitoring and analysis.

Longwall mining was reintroduced to the Illinois coal basin in the Fall of 1976 and now in 1979 four longwalls with roof shields are in operation.

ACKNOWLEDGEMENTS

The author acknowledges the permission granted him by Dr. D. S. Choi to use figures 11, 12, and 14 (15, 16); E. Jones to use figures 10 and 13 (13); J. Jansky to use figures 3 and 4 (5); T. Gales to use figure 18 (21), Emory Ayers Associates to use figures 7 and 8; P. Conroy to use figures 15 and 14 (18).

## PREMINING INVESTIGATION

The investigation includes core drilling to provide data for developing coal thickness maps and isopachs of various strata intervals and the overburden thickness (fig. 1). Physical rock properties are determined either from the cores (2) or directly by geophysical logging to obtain indicators of rock mass behavior. Estimates of roof support mean load density requirements are made based on strata separation and cantilever action in stiff strata.

Seismic techniques adapted from oil and gas exploration yield information on tectonic structures (3,4) (fig. 2). Satellite imagery and aerial photography are used to interpret linear ground features as viewed from satellites, Skylab and high altitude aircraft at distances from 570, 270, and 13 miles, respectively (5, 6). These lineaments trace shear zones in fractured rock that reach down from the surface through the strata and indicate unstable ground (figs. 3, 4).

Looking into the history of failure in past attempts at longwall mining, evaluating photographs of damaged supports and performing stress analyses on salvaged roof supports can provide significant equipment selection criteria. For example, the selection of shield-type roof supports would solve problems of instability caused by a lateral thrust inferred from the failure pattern.

Such existing horizontal in situ field stresses at the level of the coalbed can be determined by the borehole deformation method (7) or by hydraulic fracture techniques adapted from the oil industry (8, 9, 10).

Where the underground site is accessible, roof and floor rock bearing capacity tests can be performed to determine contact area requirements on roof supports and cutting patterns (11) (fig. 5). For example, a soft underclay which fails at 300 psi when wet must be protected by leaving a sufficiently thick layer of coal to prevent sinking of the supports. Conversely, soft friable roof material may be kept from spalling by leaving a sufficiently thick layer of coal. Consequently, continuous miners during the development work and the shearer-loader on the longwall face leave floor or roof coal or both as the conditions require to maintain ground stability.

## CHOICE OF SYSTEM

Standard longwall practice in the United States is retreat mining, while advancing longwalls are prevalent overseas. Reports from the West German mining industry in 1977 indicate the share of production from retreating, advancing and Z-systems as 29.5, 54.3, and 16.2 percent, respectively. The tendency in Europe is toward increasing the use of retreating longwalls. The main reasons why U.S. operators pre retreating longwalls are:

1. Longwall panel development by multiple headings is the same as for traditional room-and-pillar development.

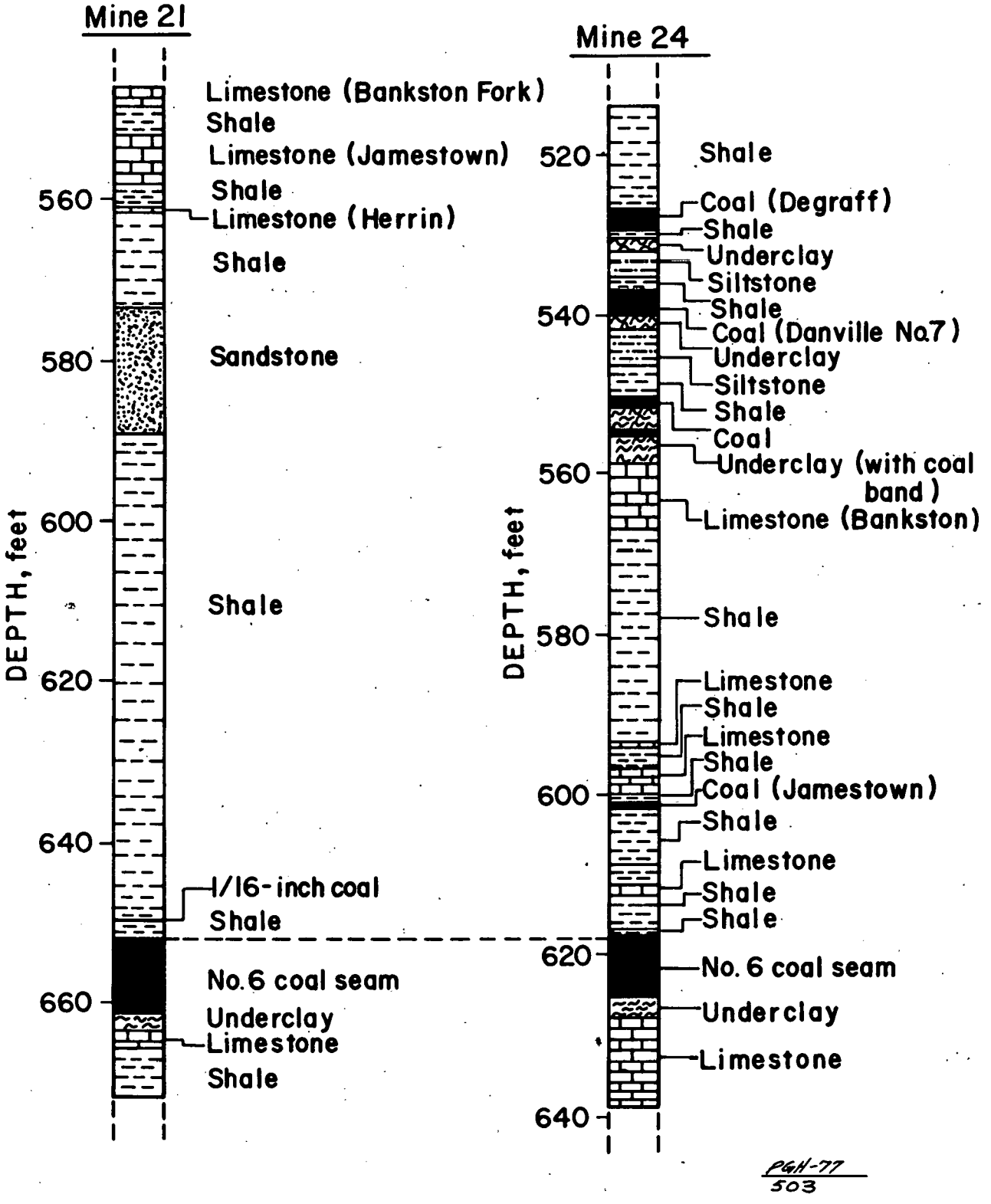



FIGURE 1 - Geologic columns according to cores.

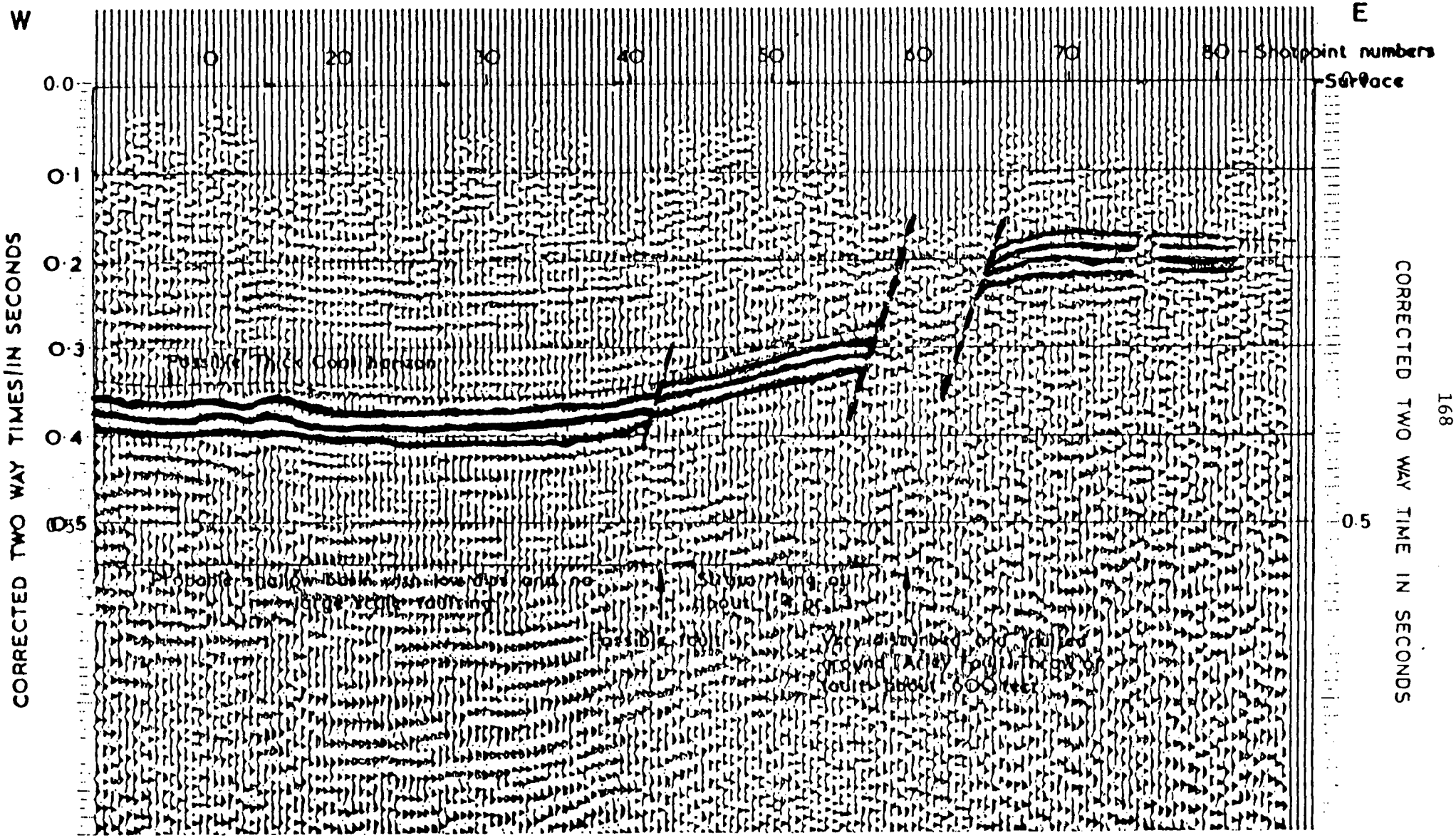



FIGURE 2. - COMPUTER PRINTOUT OF SEISMIC SURVEY

FIGURE 2 - Computer printout of seismic survey.

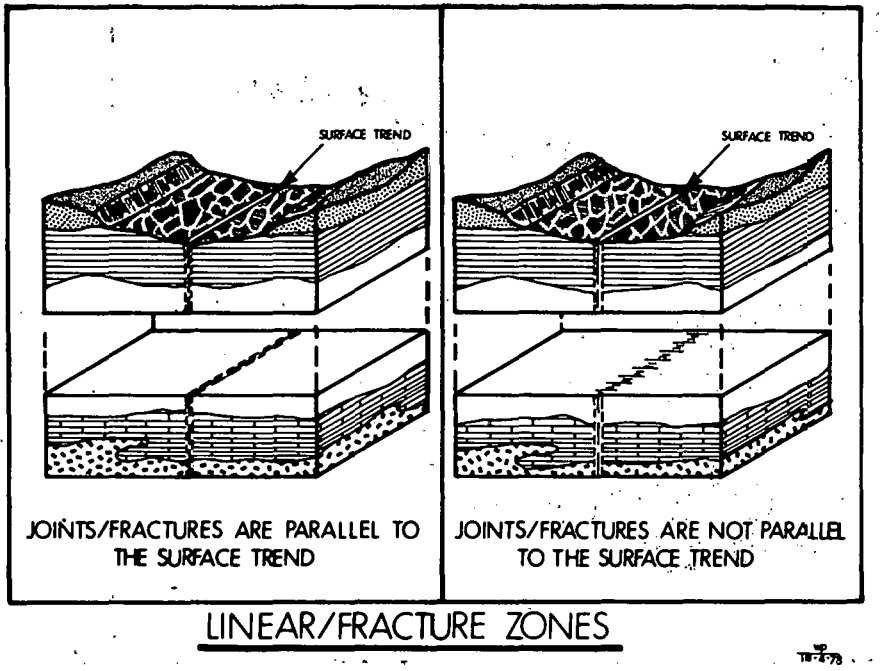
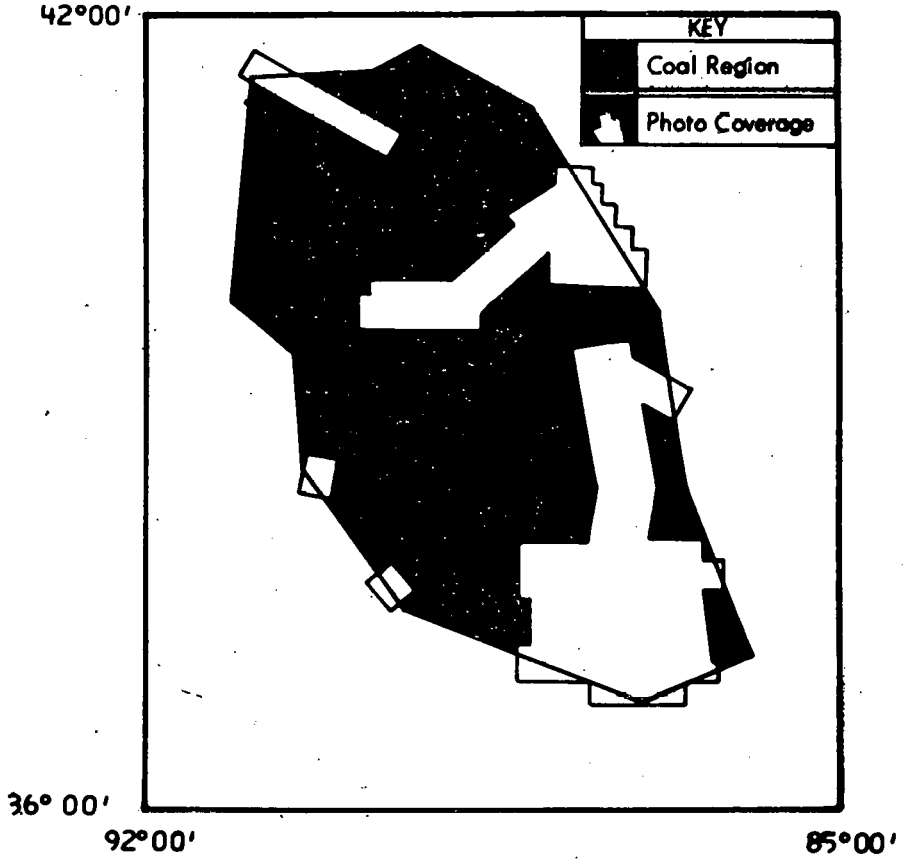



FIGURE 3 - Linear fracture zones.

# PHOTO COVERAGE

## Total Pictures
## 447




## Number of Flights

| 72 | 73 | 74 | 75 | 76 | 77 |
|---|---|---|---|---|---|
| 5 | 9 | 1 | 2 | — | — |

# EASTERN REGION
# INTERIOR PROVINCE

FIGURE 4 · Overflight of Eastern Region - Interior Province.

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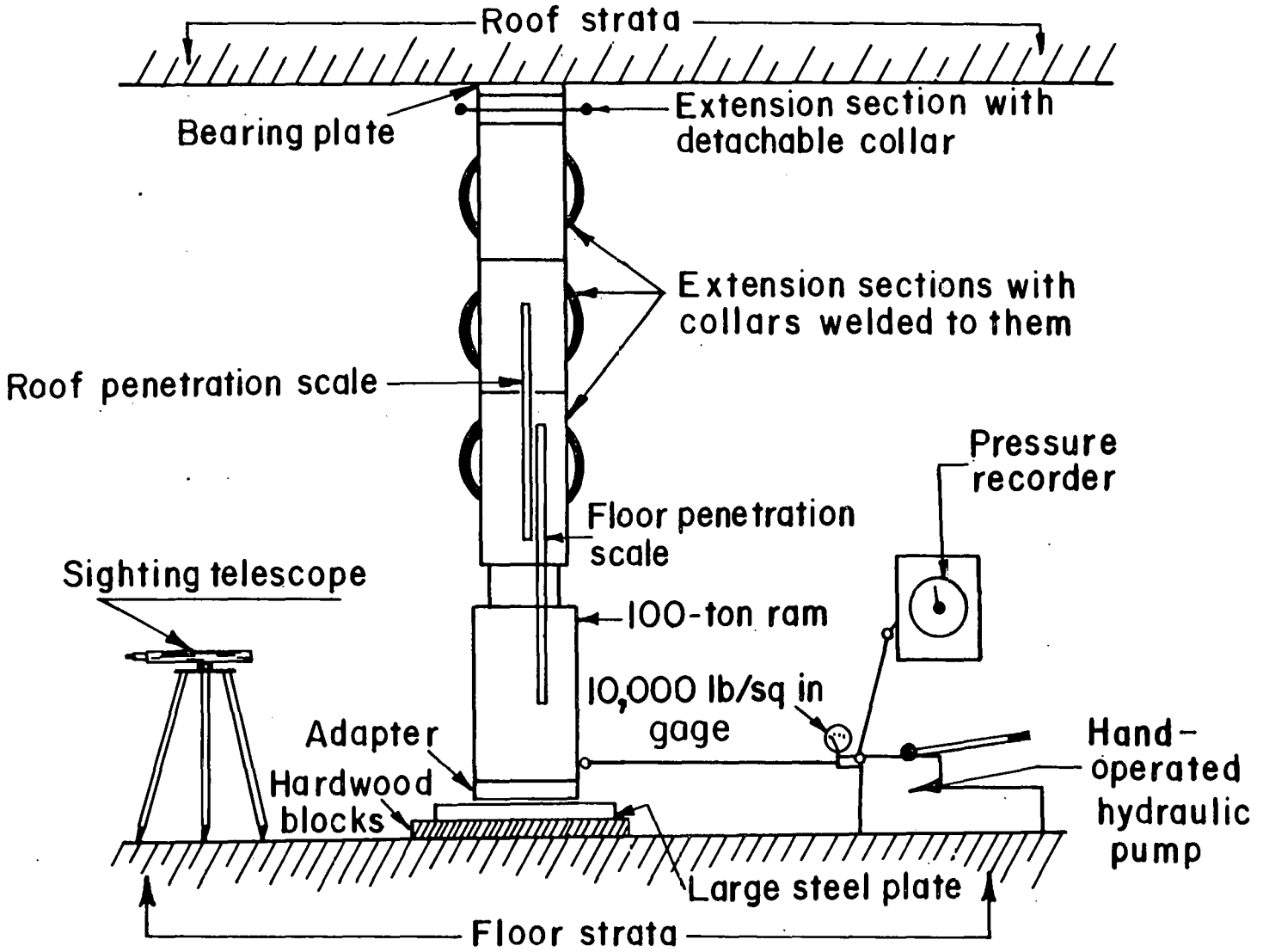


FIGURE 5 - Bearing test apparatus.

Bearing test apparatus

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2. Exploration prior to extraction reduces the risk of encountering unknown geological hazards.

3. Face operation is not tied to roadway driving and, therefore, face end formation is simplified (fig. 6).

Chief advantages of the advancing longwall method are:

1. Immediate face production and, hence, a more favorable cash flow.

2. Single entry instead of multiple entry development allows full recovery of the valuable resource without leaving chain pillars which go to waste and can cause "punch" effects in adjacent coalbeds.

3. Improved methane drainage through boreholes slanted over the gob.

There is only one advancing longwall currently operating in the western United States in an area where difficult strata conditions and high methane liberation preclude development by multiple entries (fig. 7). Plans for an advancing longwall face in an eastern mine did not materialize (fig. 8).

Z-systems feature one existing gate road and the other gate road formed in conjunction with the face (fig. 9). Such a system is in compliance with MSHA Mandatory Standards when the ranging arm shearer can be used to profile the roadway in that there is only one machine in a split of air. But the legal requirement of one intake, one return and a neutral belt entry to the face is not satisfied. The return air from the face must be coursed through the belt entry. Such a ventilation system calls for a special permit. Both advancing and Z-systems depend on the efficient installation of packwalls placed by hydraulic or pneumatic means, a technology developed overseas during the last decade, but still in its infancy in the United States.

## PANEL DESIGN

The objective of panel design is to maintain ground stability in the gate roads under the following considerations:

1. Leaving large chain pillars. Wide pillars of approximately 80 feet to 100 feet in width are left next to the tailgate roads. The rationale here is to create conditions for each sequence of panels similar to those obtained on the first panel where the least adversity is expected (12, 13). This design results from the findings of Salamon and Munro (14) who determined a Factor of Safety S by the following formula:

$$S = 1320 \times w^{.46}/h^{.66} \times (1-e)1.1H$$

where w = pillar width, feet

h = mining height, feet

e = extraction ratio, decimal

H = depth of overburden, feet

The extraction ratio e considers the whole area including longwall face and chain pillars (fig. 10).

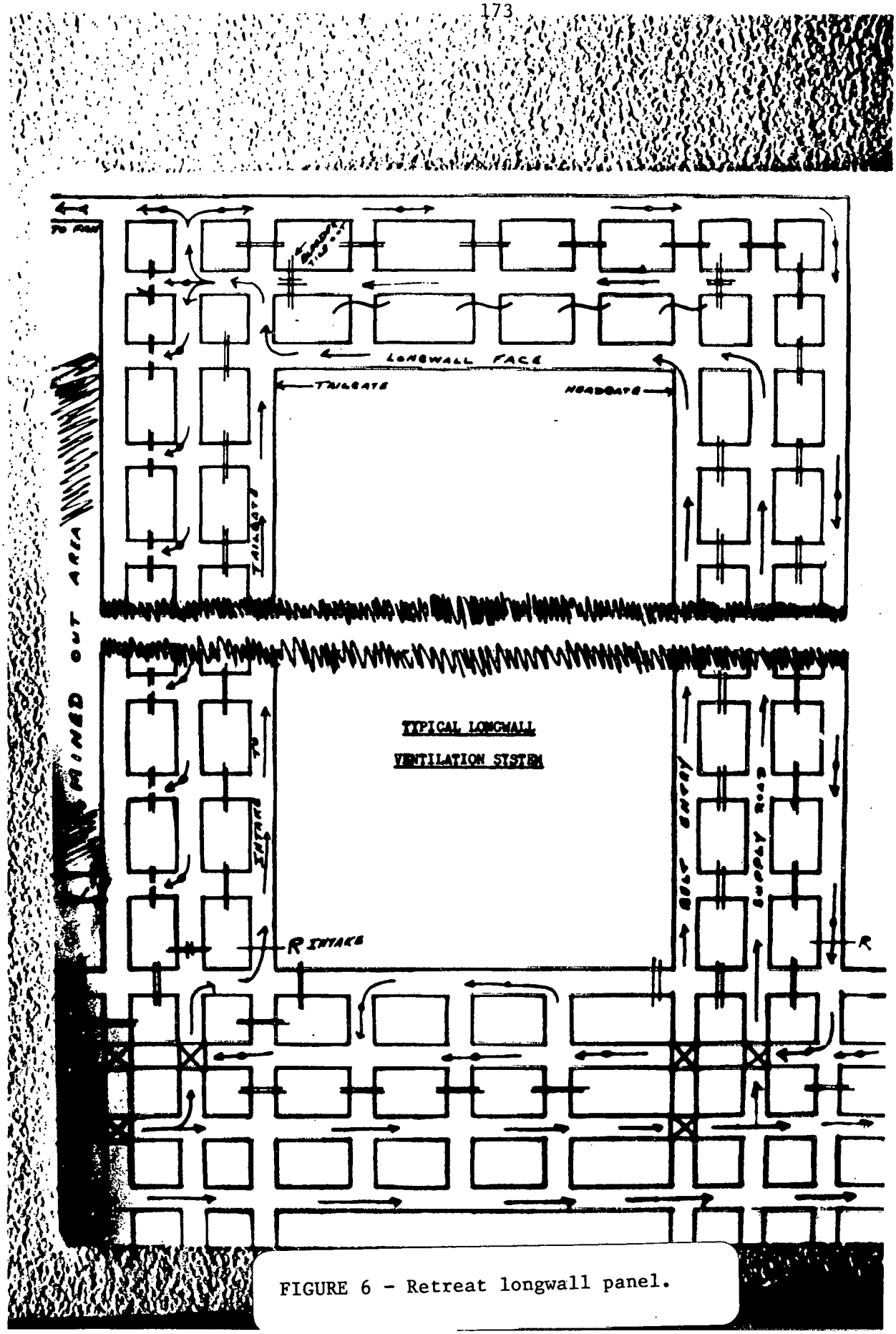


FIGURE 6 - Retreat longwall panel.

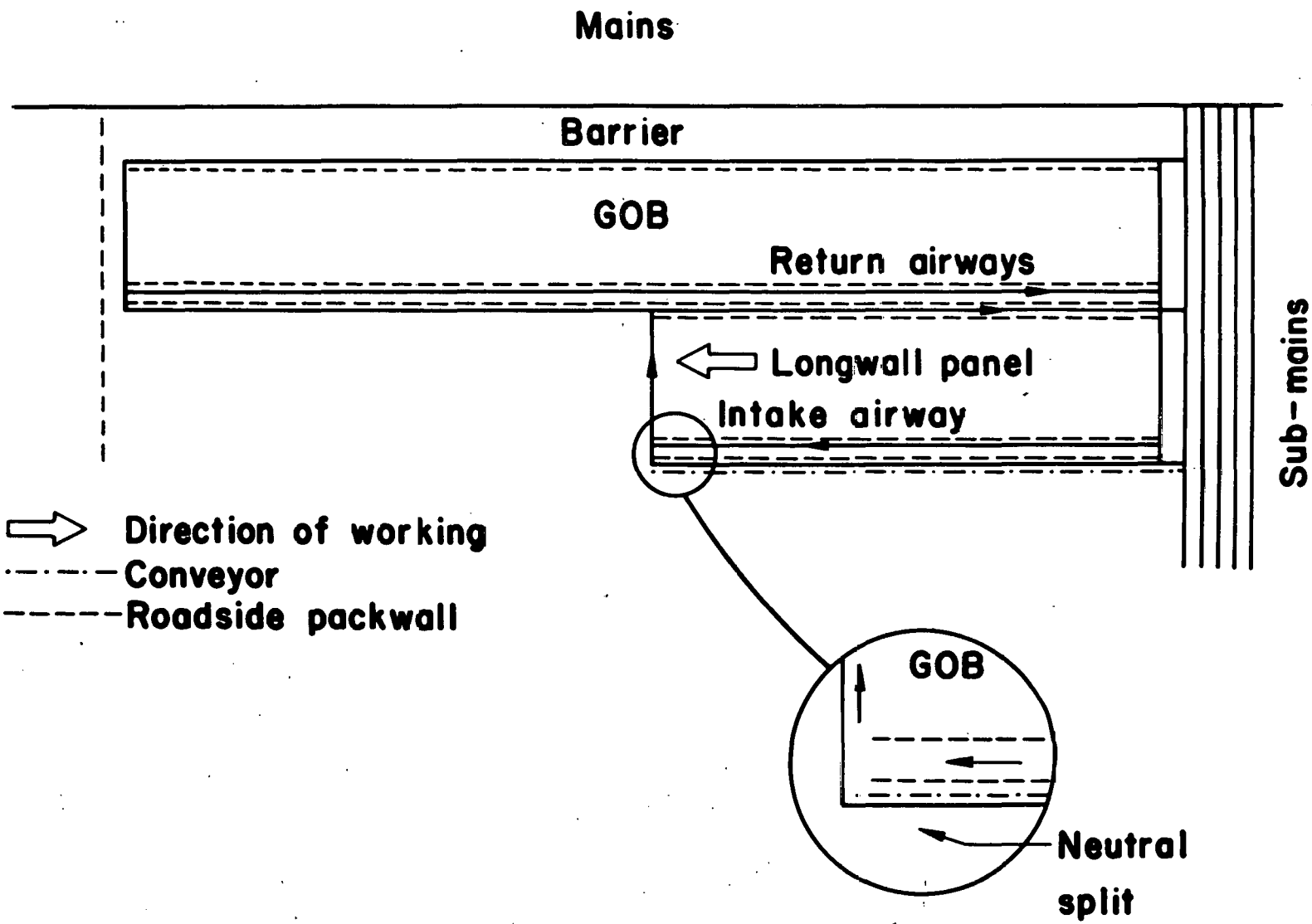


FIGURE 7 - Advancing longwall - Divided Entry.

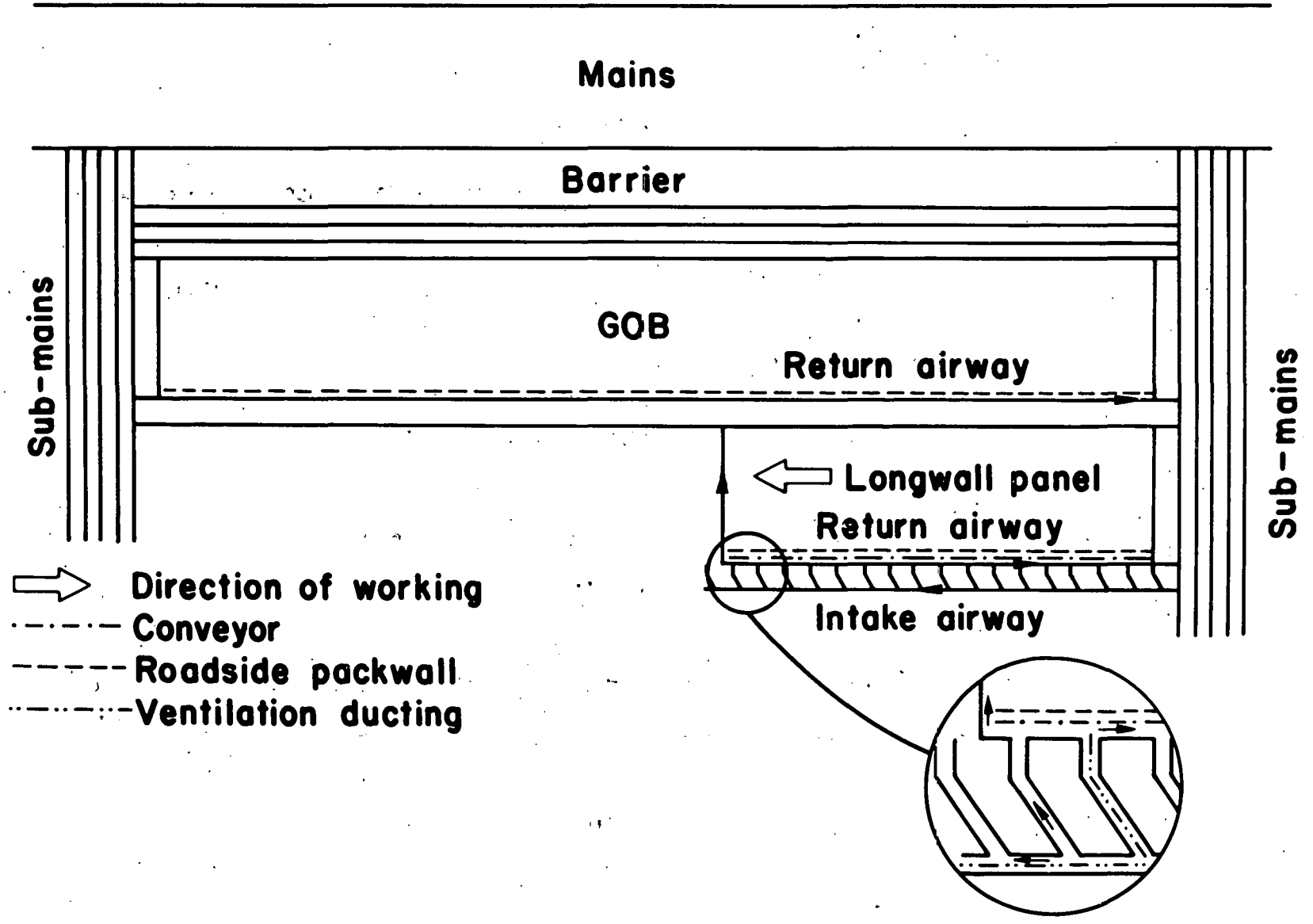


FIGURE 8 - Advancing longwall - Double Entry.

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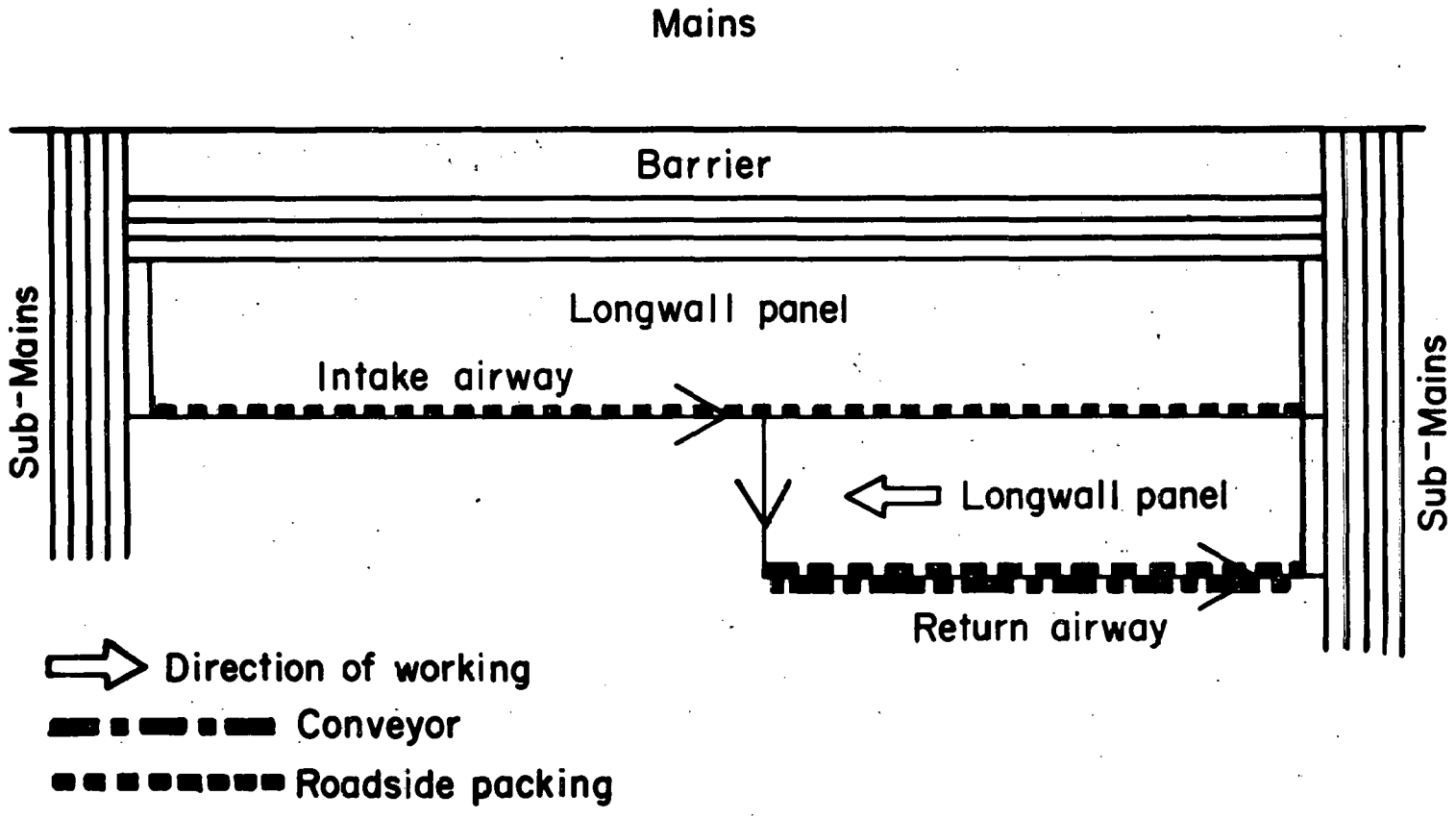


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FIGURE 9 - Z - system.

# PILLAR DESIGN FOR FREEPORT SEAMS IN CENTRAL PENNSYLVANIA

4 Entry System - 72" Coal Height

| Heading Centers | Pillar Width | Length of Face | (1 -c) | Depth of Cover 600' | 500' | 400' | 300' | 200' |
|---|---|---|---|---|---|---|---|---|
| 100 X 100 | 80' | 300' | .310 | 1.426 | 1.711 | 2.139 | 2.852 | 4.278 |
| | | 350' | .287 | 1.320 | 1.584 | 1.980 | 2.640 | 3.960 |
| | | 400' | .267 | 1.228 | 1.474 | 1.842 | 2.456 | 3.684 |
| | | 450' | .249 | 1.146 | 1.375 | 1.719 | 1.970 | 3.438 |
| | | 500' | .234 | 1.077 | 1.292 | 1.616 | 2.154 | 3.231 |
| 90 X 90 | 70' | 300' | .249 | 1.077 | 1.292 | 1.616 | 2.154 | 3.231 |
| | | 350' | .230 | .995 | 1.194 | 1.492 | 1.990 | 2.985 |
| | | 400' | .213 | .922 | 1.106 | 1.383 | 1.844 | 2.766 |
| | | 450' | .199 | .861 | 1.033 | 1.292 | 1.722 | 2.583 |
| | | 500' | .186 | .805 | .966 | 1.208 | 1.610 | 2.415 |
| 80 X 80 | 60' | 300' | .193 | .778 | .934 | 1.167 | 1.556 | 2.334 |
| | | 350' | .177 | .713 | .856 | 1.070 | 1.426 | 2.139 |
| | | 400' | .164 | .661 | .793 | .992 | 1.322 | 1.983 |
| | | 450' | .152 | .613 | .736 | .920 | 1.226 | 1.839 |
| | | 500' | .142 | .572 | .686 | .858 | 1.144 | 1.716 |
| 70 X 70 | 50' | 300' | .142 | .526 | .631 | .789 | 1.052 | 1.578 |
| | | 350' | .129 | .478 | .574 | .717 | .956 | 1.434 |
| | | 400' | .119 | .441 | .529 | .662 | .882 | 1.323 |
| | | 450' | .110 | .408 | .490 | .612 | .816 | 1.224 |
| | | 500' | .103 | .382 | .458 | .573 | .764 | 1.146 |

$$\text{Factor of Safety} = \frac{1320\ W^{.46}}{H^{.66}} \times \frac{(1-E)}{1.1H}$$

W = Width of pillar

H = Height of coal

E = Extraction decimal

H = Depth below surface

FIGURE 10 - Safety factors in panel design.

2. Choi, Dahl, and Von Schönfeld (15) recommend that the large chain pillars be located at the head gate entry of a panel and that the small yield pillars be located at the tailgate entry for the next panel. This configuration will provide reduced deformation and improved stability around the tailgate; it should also induce a cleaner break at the edge of the gob and improve overall opening stability by transferring a significant portion of the overburden weight to the gob. Figure 11 shows a finite element model designed to determine loads on the chain pillars. Recently, Choi and McCain (16) developed a chart for the Pittsburgh coalbed in northern West Virginia for a coal with a support capability of 2,200 to 2,500 psi. The chart shows the chain pillar width in terms of panel widths for several overburden thicknesses (fig. 12).

3. The general design practice in the United States is to adopt the same time proven panel geometry as that used in the room-and-pillar system in the individual mines. Following simple example shows how a Safety Factor for ground support is computed:

Assuming pillar dimensions of 50 x 50 ft and 20 ft wide entries and crosscuts, the extraction ratio is (17):

$4900\ ft^2$ = total area

$-2500\ ft^2$ = left in pillars

$2400\ ft^2$ mined = extraction ratio of 49%

Under 650 feet of cover the weight of the original burden is distributed over 51 percent of the coal left in situ after first mining. Hence, the unit load on the coal rises to 650/0.51 = 1,275 psi. Assuming the compressive strength of the coal to be 2,000 psi, the Safety Factor is

$$S = 2{,}000/1{,}275 = 1.6$$

The minimum number of panel entries for longwall development to comply with MSHA mandatory standards is three, except for some western mines where exceptions for double entry and single entry (with appropriate dividers) development were approved. Ventilation requirements may dictate a minimum of four development entries. In southern Illinois, the longwalls are developed by sets of 3 entries on 60 and 100 foot centers (fig. 13). The chain pillar on 60 foot centers adjacent to the panel is mined along with the longwall face, thus adding to its length. This practice is unique to longwall mining and is designed to increase overall coalbed recovery. The three gate roads leading to the face are the headgate with the belt, the supply gate for men and supply transportation, and the tail gate.

Keith of the University of Missouri, Rolla, performed a finite element analysis of the Old Ben longwall development entry system (18). Input data were provided by Dames & Moore. Fig. 14 shows the entry geometry and Figure 15 the results of the analysis. Roof rock, coal and underclay floor are introduced as elastic materials without any tensile strength whose assigned physical properties are the critical variables in the rock mechanics applications of finite elements analyses. However, the underclay floor is modelled to act plastically after exposed to a deviator stress in excess of 1200 psi.

Case No. 1 in figure 15 shows the premining condition. Only minor insignificant local fracture zones in brittle material are indicated. Case No. 2 shows the condition during mining. The roof shields in the excavated panel area apply a maximu support of 125 psi = 9 tons/$ft^2$ which is introduced as a boundary stress into the model. A zone of fracturing slightly larger than in the premining setting appears in the main gate rib adjacent to the mined panel. Although the underclay floor

# Finite Element Mesh

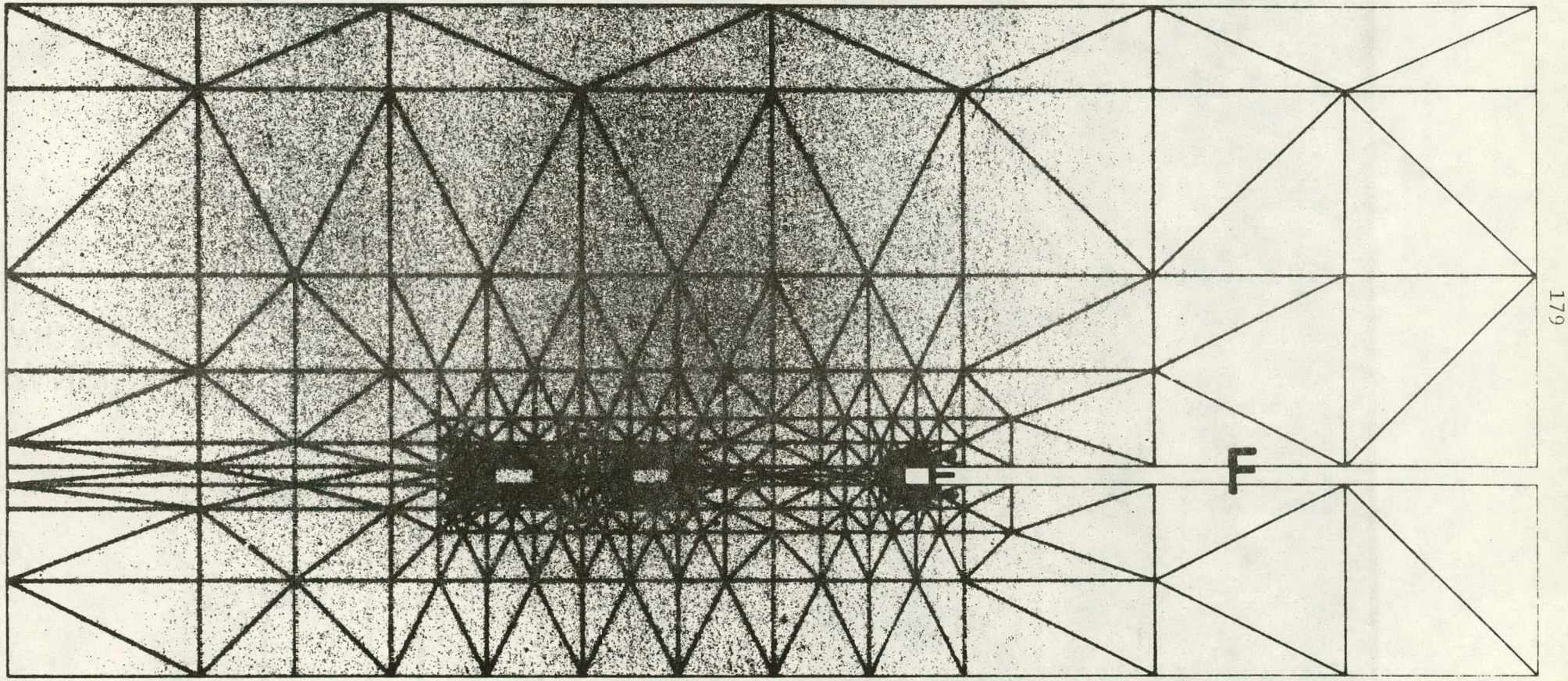


FIGURE 11 - Finite element model.

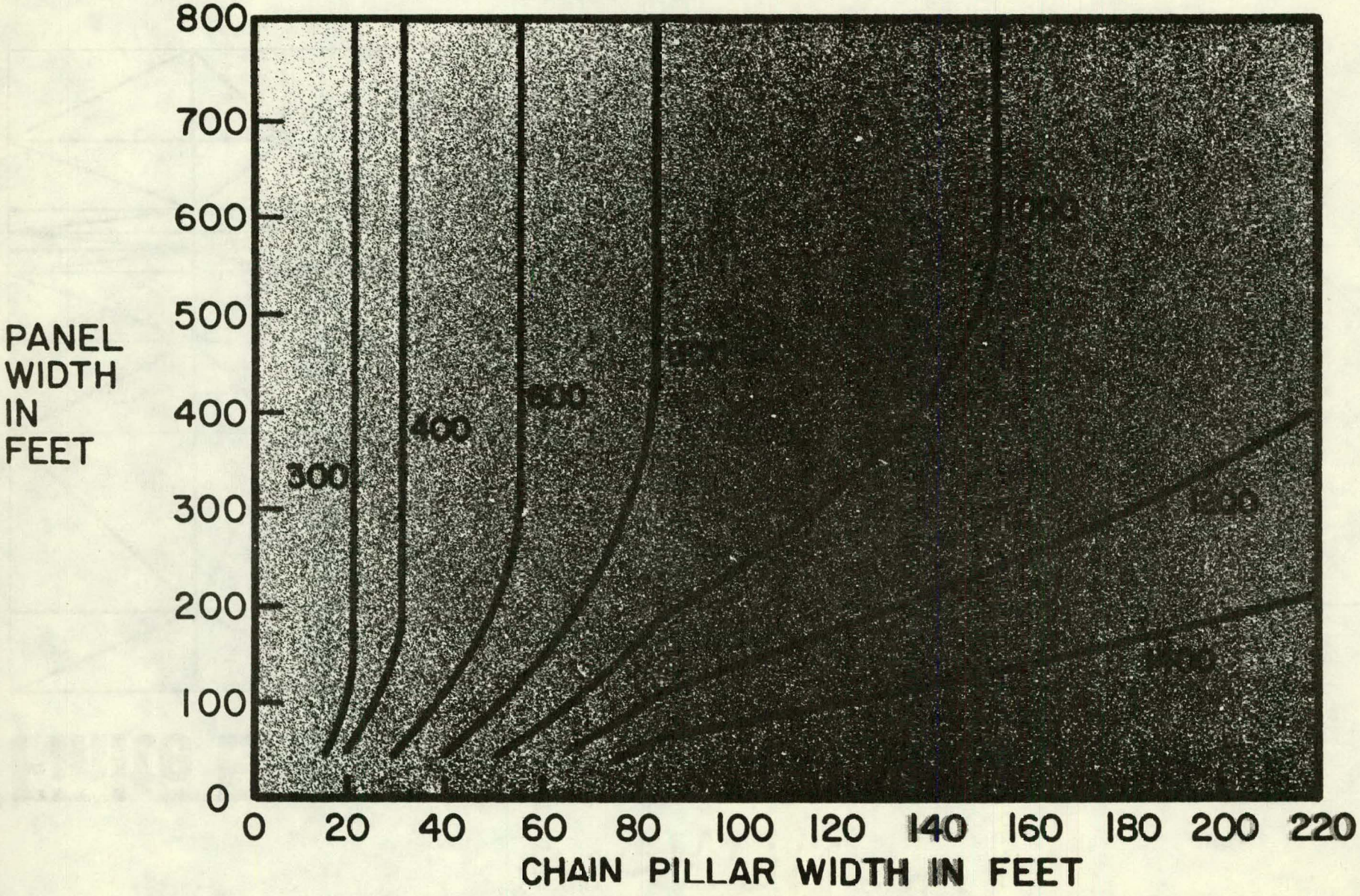


FIGURE 12 - Chain pillar width in terms of panel width and overburden.

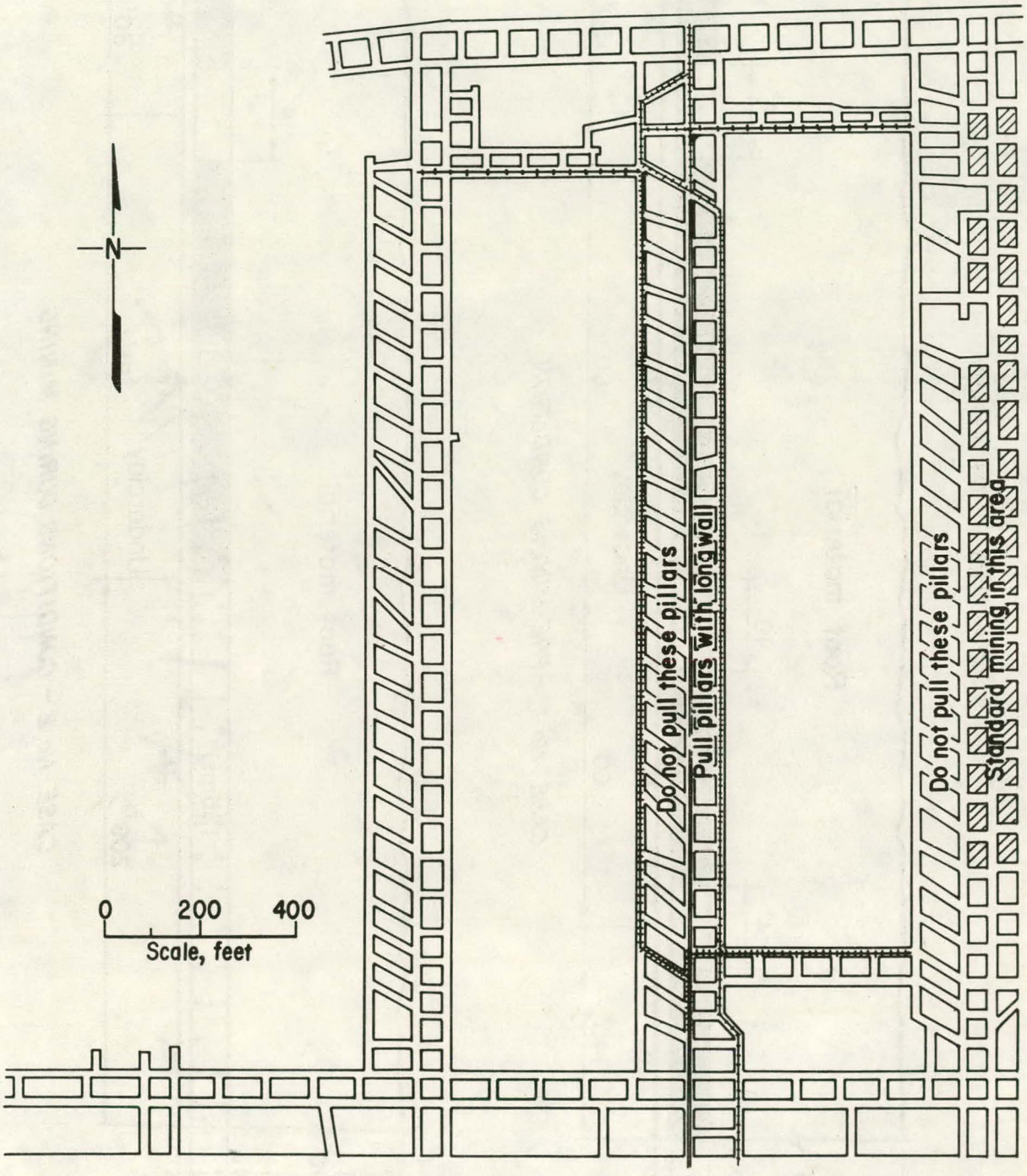


FIGURE 13 - Panel layout.

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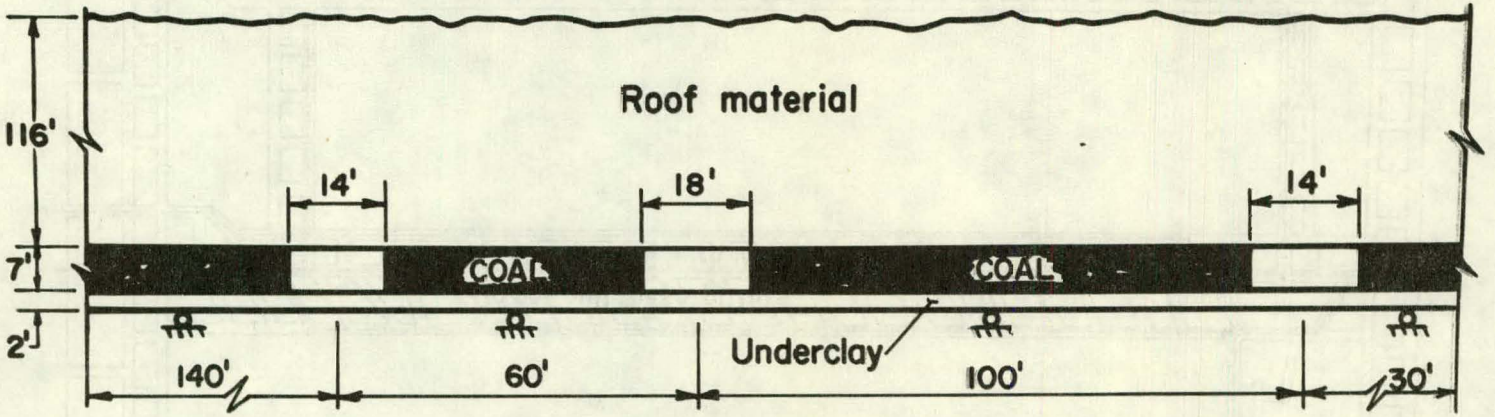
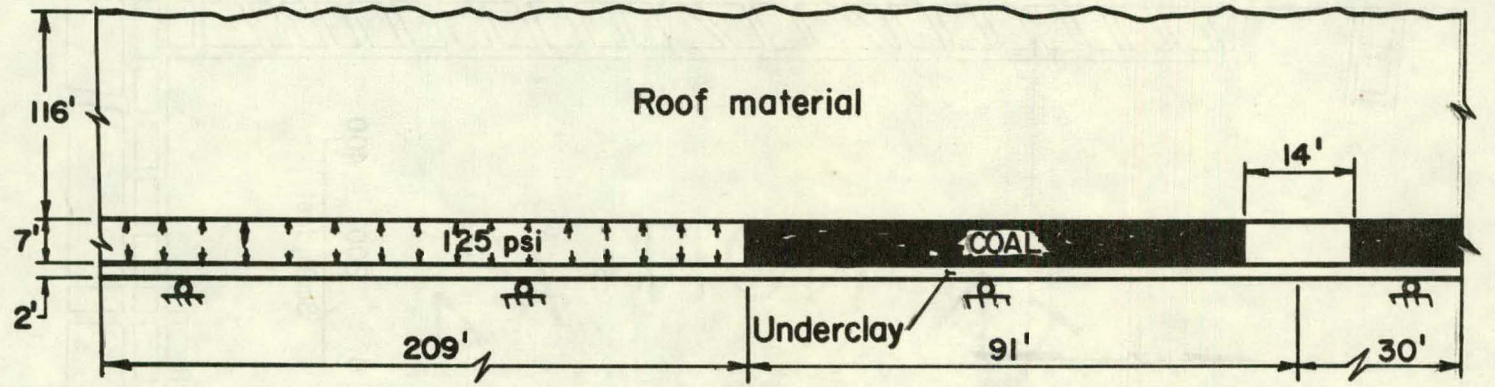
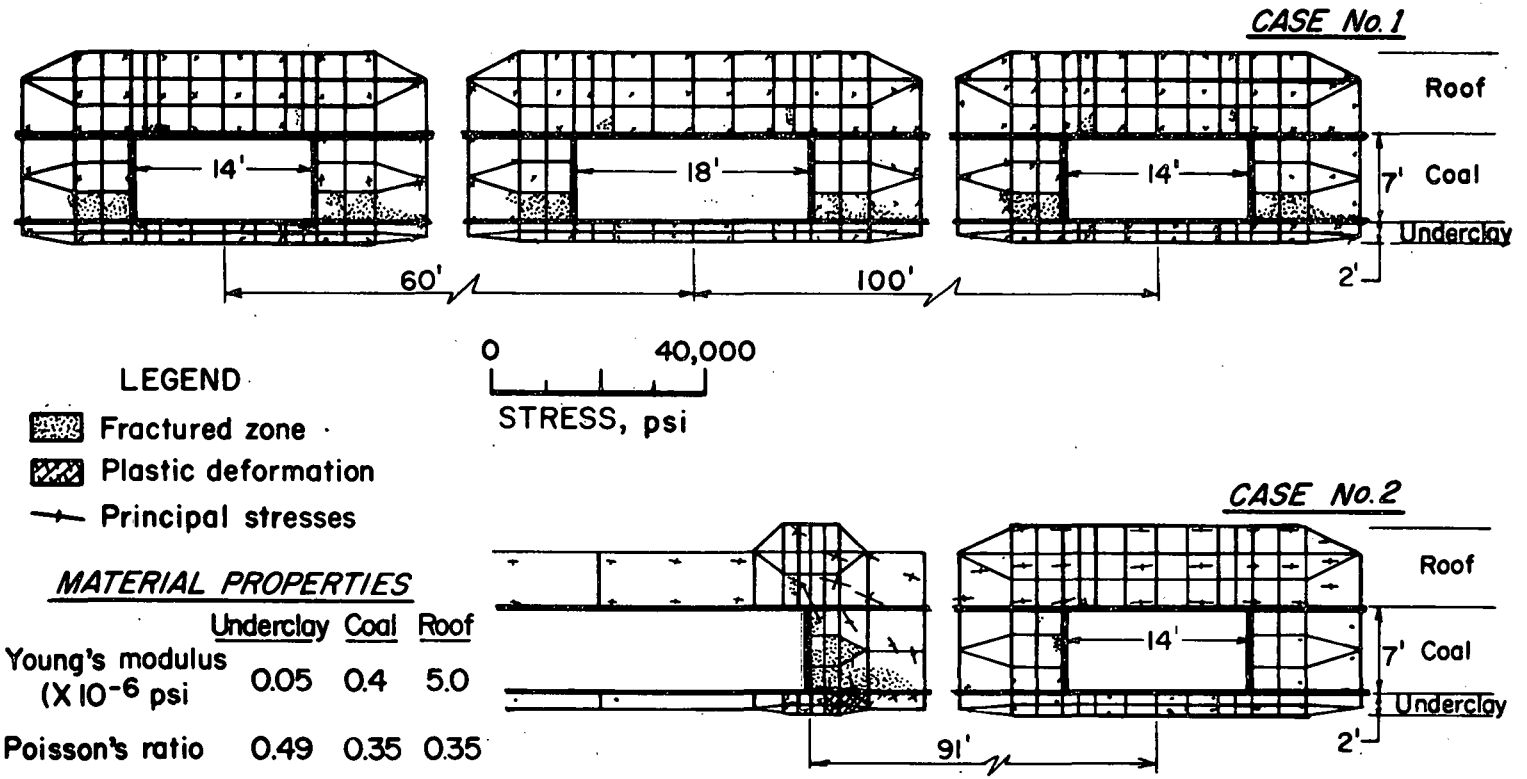



CASE No. 1 – PRE-MINING CONDITION




CASE No. 2 – CONDITIONS DURING MINING

FIGURE 14 - Entry geometry. 10-12-79 L-17082




*MATERIAL PROPERTIES*

| | Underclay | Coal | Roof |
|---|---|---|---|
| Young's modulus ($\times 10^{-6}$ psi | 0.05 | 0.4 | 5.0 |
| Poisson's ratio | 0.49 | 0.35 | 0.35 |
| Yield Strength psi | 1200 | --- | --- |

FIGURE 15 - Finite Element Analysis
by H. D. Keith. 10-12-79 L-17081

yields to a small extent, the integrity of the gate road in the model is not threatened and near horizontal stresses in the roof even enhance the stability of the openings.

Experience in the Illinois coal basin has shown that entries wider than about 15 feet cannot be maintained even when the roof is reinforced with resin bolts and coated with a protective sealant. The belt entries or longwall head gates were originally driven 18 feet in width to accomodate the hydraulic powerpack and electrical power center along the conveyor belt. However, falls occurred in the headgates during retreat mining. Therefore, the belt entry width was reduced to 15 feet, and the powerpacks and power centers were modified to fit into the narrower entry configuration. The width of the stage loader, too, was reduced by relocating the drives. Belt entry roof support by resin anchored trusses has been introduced recently.

The pressure arch theory, developed in Britain to explain rock mass behavior, assumes an arch of destressed material forms above the excavation and relieves it from the overburden pressure buildup at the ends of the arch which follows the outline of the longwall panel (19). Critical ground stresses may develop at the face ends where abutment forces along the face combine with those along the gate roads. They impose a heavy load on the roof support system while the longwall faces passes through (fig. 16).

Single hydraulic props, by virtue of early bearing capability, are used to control convergence in gate roads. They are set in addition to cribbing which relies on rock deformation for building up support strength. The props are connected to the hydraulic face supply system and can extend to exert a thrust of 30 tons, if properly blocked. They yield at 44 tons (fig. 17).

The effect of face length on strata control is uncertain. Investigations in Great Britain have shown that longer faces do not experience more roof failures than shorter faces and mining research in West Germany found that, in strong roof strata, caving was improved on longer faces. Although the pressure arch theory has been applied to yield pillar techniques in panel design (20), no relationship between optimum longwall face length and strata control has been developed.

Therefore, selection of face length is guided largely by economic considerations. Cost of equipment ownership increases with face length (20), but the cost of panel development per unit of mined coal decreases as face length increases (fig. 18). An equation was developed to obtain $C_s$ = cost per ton of coal mined (fig. 19) (22).

$$C_s = \frac{C_T}{P_T}$$

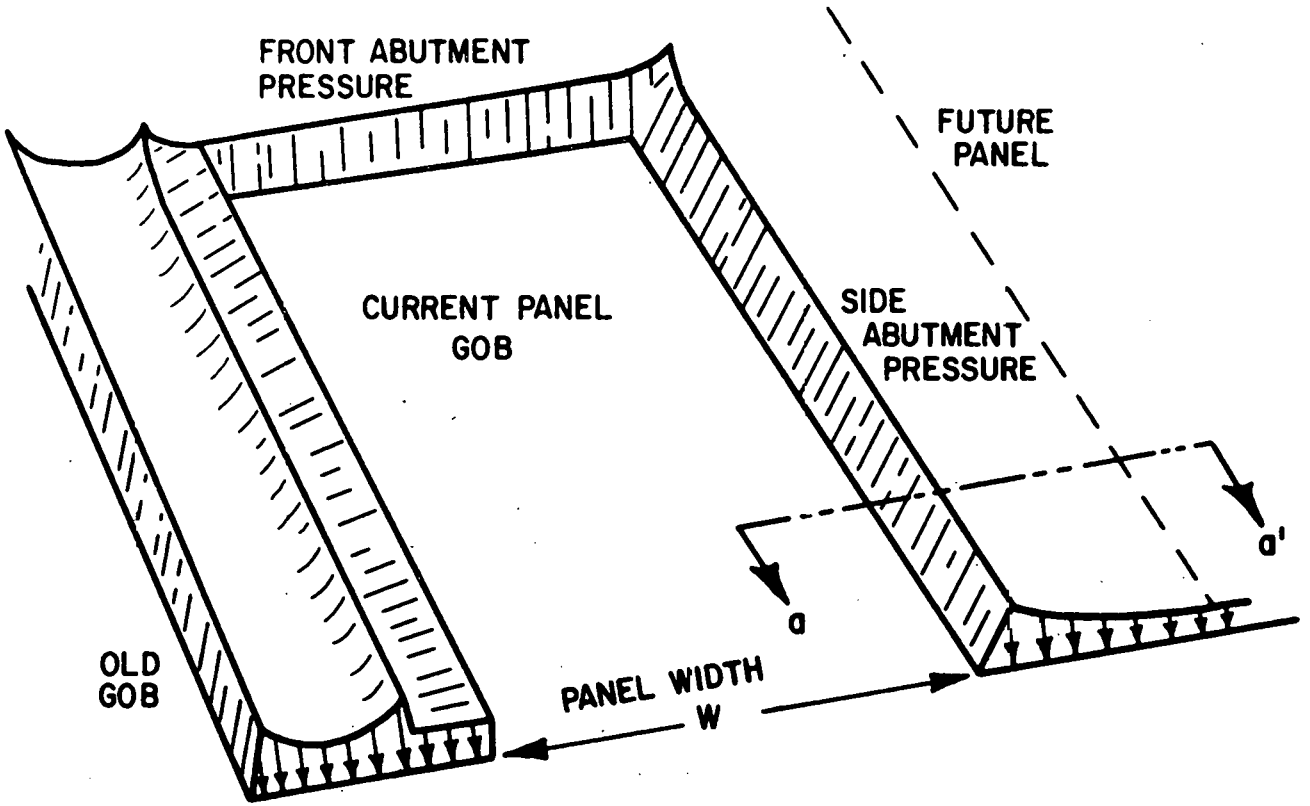
$$\frac{L_f(C_{WF} + C_{HFC}) + L_f^2 C_{HFL} + \frac{P_F C_{WG}}{P_G}(C_{WG} + C_{HG})}{P_F(L_f + C_{WG})} \qquad (2)$$

Differentiating the equation and setting the first differential quotient equal to zero results in the optimum face length.

$$\frac{dC_s}{dL_f} = 0$$

$$L_f max = \left( \sqrt{1 + \frac{\frac{P_F}{P_G}(C_{WG} + C_{HG}) - (C_{WF} + C_{HFC})}{C_{WG} C_{HFL}}} - 1 \right) C_{WG} \qquad (3)$$

PRESSURE REDISTRIBUTION IN THE MIDDLE OF COAL SEAM AROUND A WIDE CAVING MINING PANEL



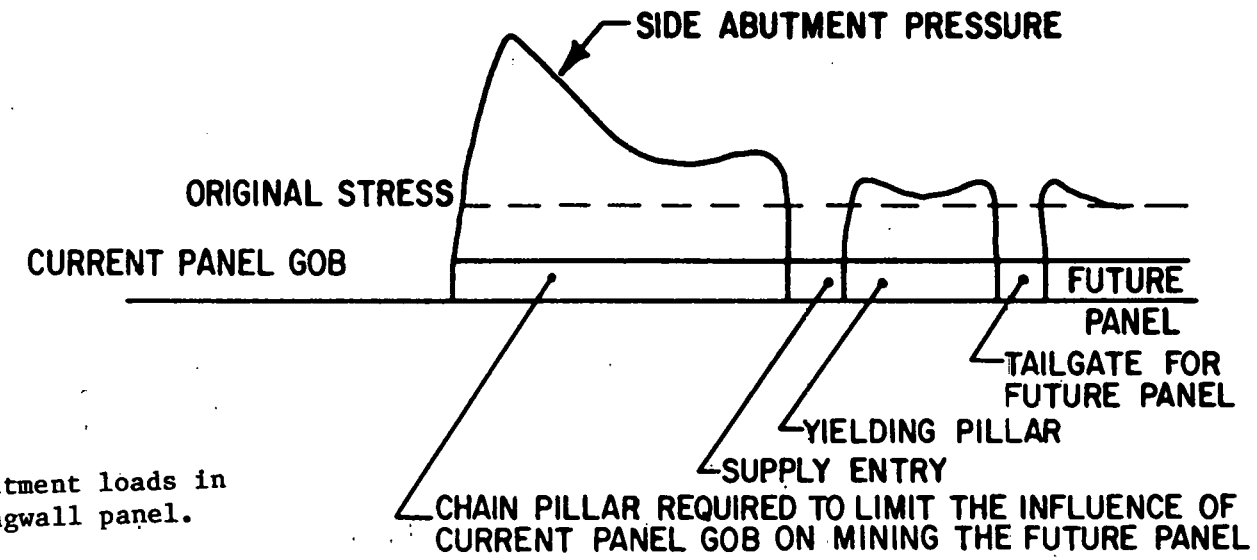




FIGURE 16 - Abutment loads in longwall panel.

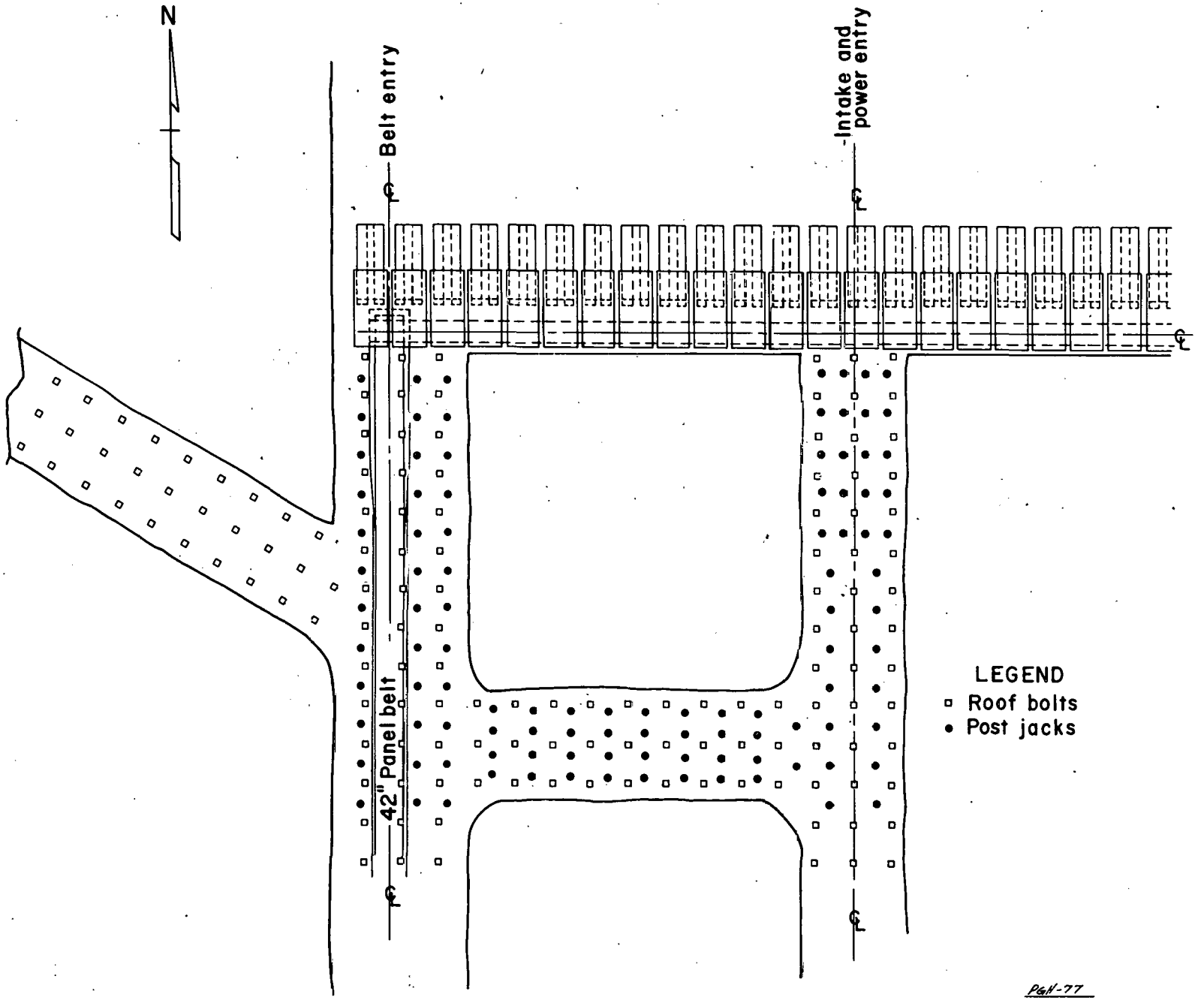



FIGURE 17 - Roof support for gateroads.

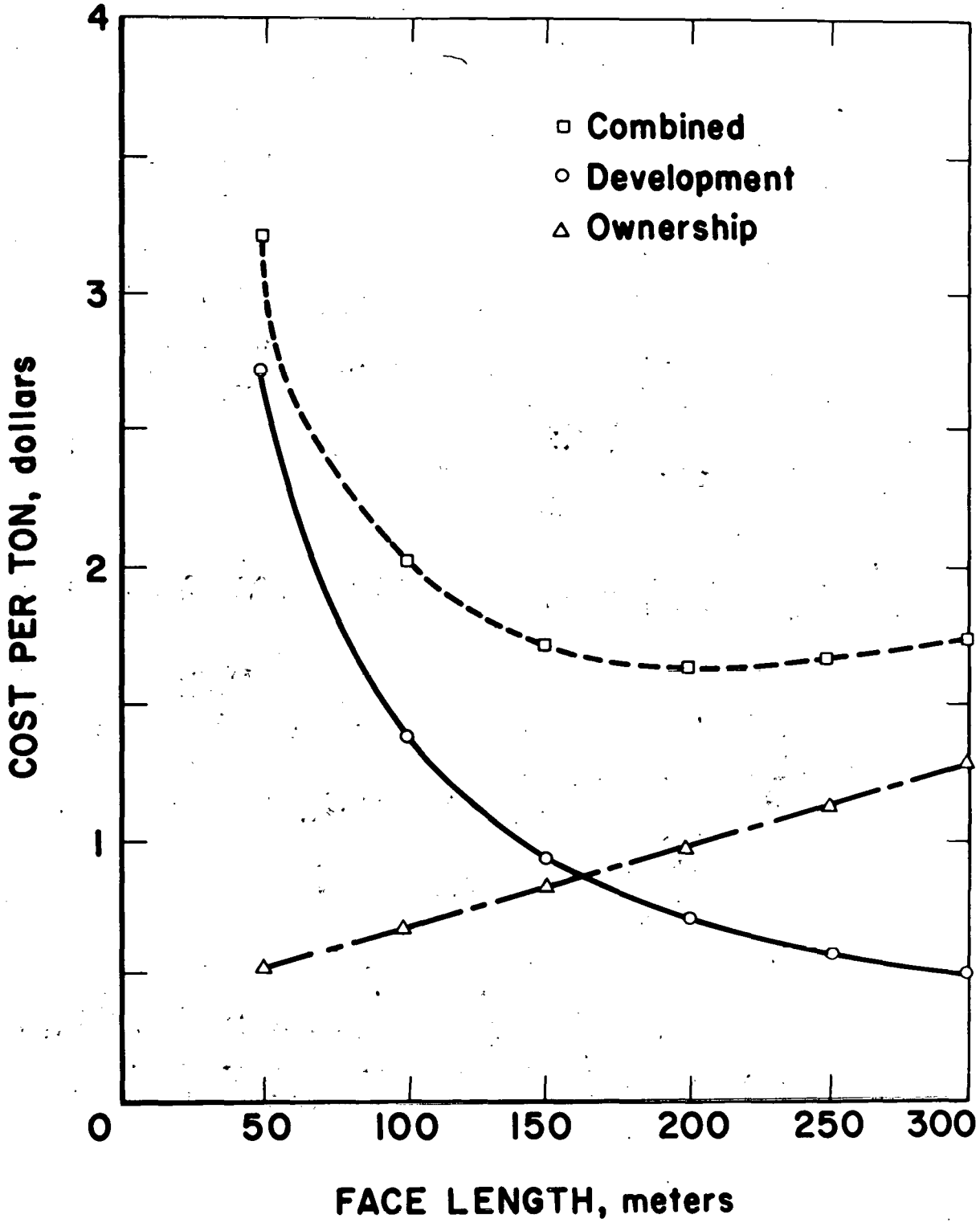
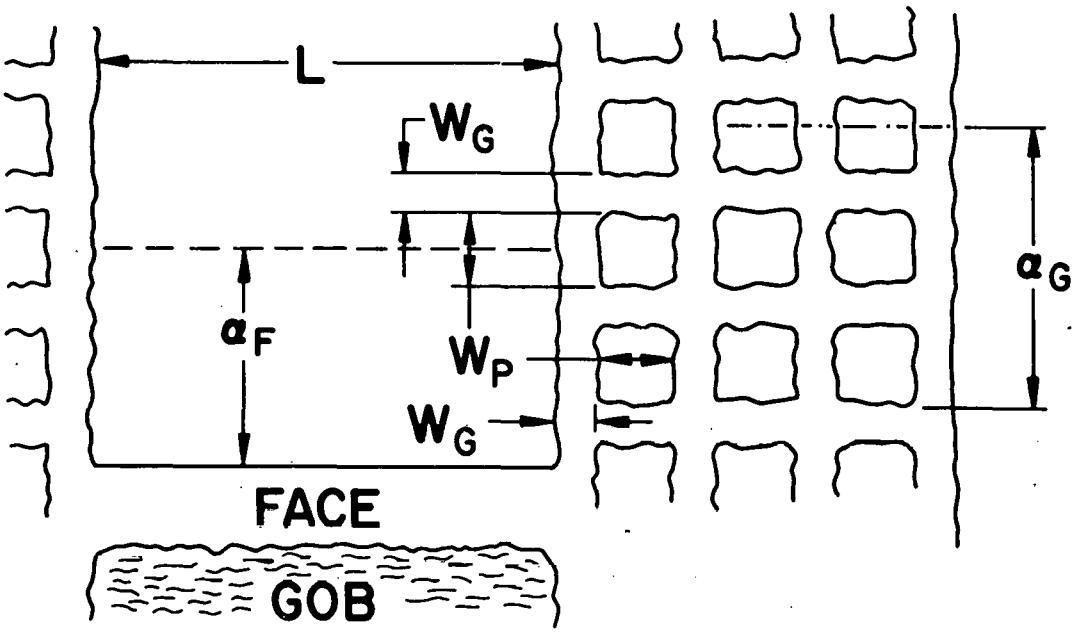



FIGURE 18 - Face length in terms of ownership and development costs. 24-79 L-16877




$C_S$ =COST PER TON OF COAL MINED.

$$C_S = \frac{C_T}{P_T} = \frac{L_f(C_{WF}+C_{HFC}) + L_f^2 C_{HFL} + \frac{P_F C_{WG}}{P_G}(C_{WG}+C_{HG})}{P_F(L_f + C_{WG})}$$

FROM $\frac{dC_S}{dL_f} = 0$ FOLLOWS:

$$L_f\,MAX = \left( \sqrt{1 + \frac{\frac{P_F}{P_G}(C_{WG}+C_{HG}) - (C_{WF}+C_{HFC})}{C_{WG}\,C_{HFL}}} - 1 \right) C_{WG}$$

FIGURE 19 - Determining optimal face length.

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Where

$P_T$ = total production of the face including face and gate, tons/day

$P_F$ = production of the face, tons/day

$P_G$ = production of the gate development, tons/day/continuous miner section

$L_f$ = face length, ft

$C_{WF}$ = labor costs for face crew, $/day

$C_{WG}$ = labor costs for gate crew, $/day

$C_{HFC}$ = costs of hardware in face remaining constant, $/day

$C_{HFL}$ = costs of hardware in face changing with face length, $/day

$C_{HG}$ = costs of hardware in gate during developing, $/day/continuous miner section

$C_T$ = total cost of face and entry system, $/day

$C_S$ = specific cost of production, $/ton

Economic face lengths in the United States range from 450 feet in thick coalbeds to 600 feet in thin coal beds. Obviously, larger blocks (face length times panel length) mean the wasted resource including chain pillars, recovery stalls and bleeder pillars will represent a smaller portion of the total coal developed and costly equipment moves will be less frequent.

Factors which affect panel length are (22):

Surface effects

Local geology

Interaction with previous mine workings

Oil and gas wells

Boundaries of coal reserves

Gate road maintenance cost

Longwall panels in the United States average 3000 feet in length. A few exceed 5000 feet (23).

Several new coal mines have been developed for longwall mining with panel blocks as large as limiting factors allow. They are projected to have two active longwall faces and as many continuous mining units as are required for development and extraction.

## ENVIRONMENTAL ASPECTS

Environmental considerations which affect the planning of a longwall face are surface utilization and hydrologic information such as hydrostatic formation,

fluid pressure, and aquifer permeability (24). Proposed Rules in the implementation of the Surface Mining Control and Reclamation Act of 1977 (PL.97-87) include buffer zones for protection of public buildings, aquifers of the ground water supply and water impoundments. In addition, the Act requires subsidence monitoring by the Office of Surface Mining Reclamation and Enforcement (OSM) and advanced notice to owners of surface property above or adjacent to planned mining operations (25). This implies identification of surface premining damages prior to mining-induced subsidence. Also, oil and gas wells may affect the panel design.

## EQUIPMENT SELECTION

Based on data resulting from the premining investigation, selection criteria for roof supports are developed. Shields are being preferred over chocks since they have been introduced in America in 1975. Decisive factors for favoring shields over chocks are:

1. A sheltered working space requiring minimum cleanup work.

2. Structural stability which allows advancing without delay even with brushing roof contact.

Figure 20 shows how the support forces can be evaluated for two-legged caliper type and lemniscate shields. The canopy tip of the caliper-type shield describes a circular arc when the shield is raised or lowered. Hence, the critical span between canopy tip and face widens with increasing height of the shield unless compensated by an extension. Caliper shields also develop a horizontal thrust towards the face when yielding. On lemniscate shields the span between canopy tip and face barely changes during vertical travel. The first shields introduced in the United States and installed in mines in New Mexico and West Virginia in 1975 were caliper-type units. The lemniscate shield was introduced first in Illinois in 1976.

Figure 21 shows the side view of such a lemniscate shield in the one-web-back position with a shearer-loader and face conveyor. A side view of an advanced type of shield installed in an Illinois mine in 1978 illustrates how the joint between the gob end of the canopy and the gob shield has been redesigned to eliminate the dead corner where debris could accumulate (Fig. 22). Figure 23 shows a similar shield with a large ram between the canopy and the gob shield for stabilizing the joint. Figure 24 shows a four-legged shield of a type which is used in several mines in Pennsylvania, Ohio, and West Virginia. These shields provide a high tip load.

Favorable roof support selection criteria include:

1. A large hydraulic extension range so that roof cavities can be supported quickly.

2. An adequate mean load density to support a rock column considering strata separation and cantilever action (26). Wade (27) developed the following formula for mean load density:

$$MLD = (1 + \sum_{n=1}^{5} C_n) \times 4\gamma h \text{ tons/ft}^2 \qquad (6)$$

where h = mining height, ft

$\gamma$ = specific weight of rock, tons/ft$^3$

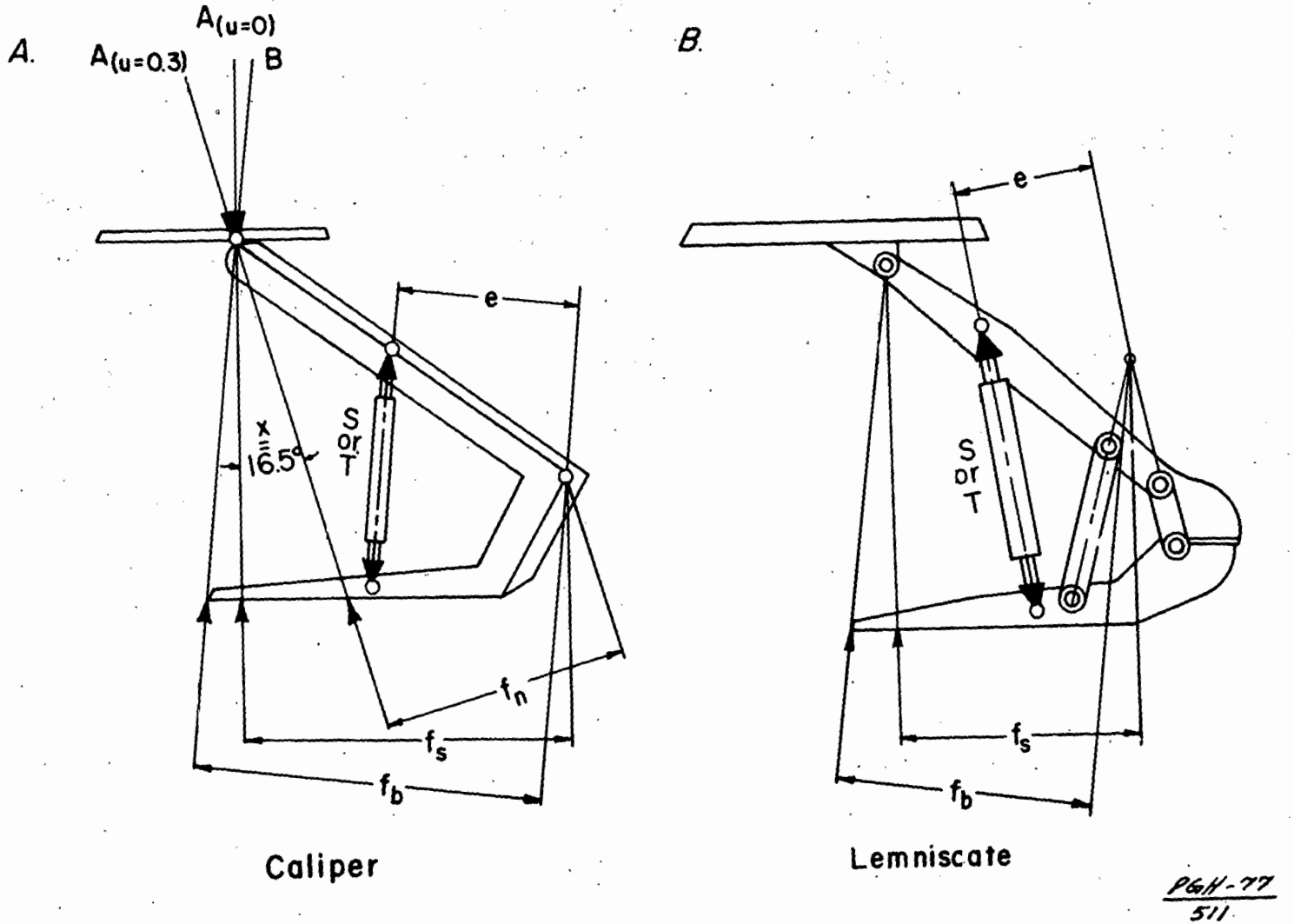
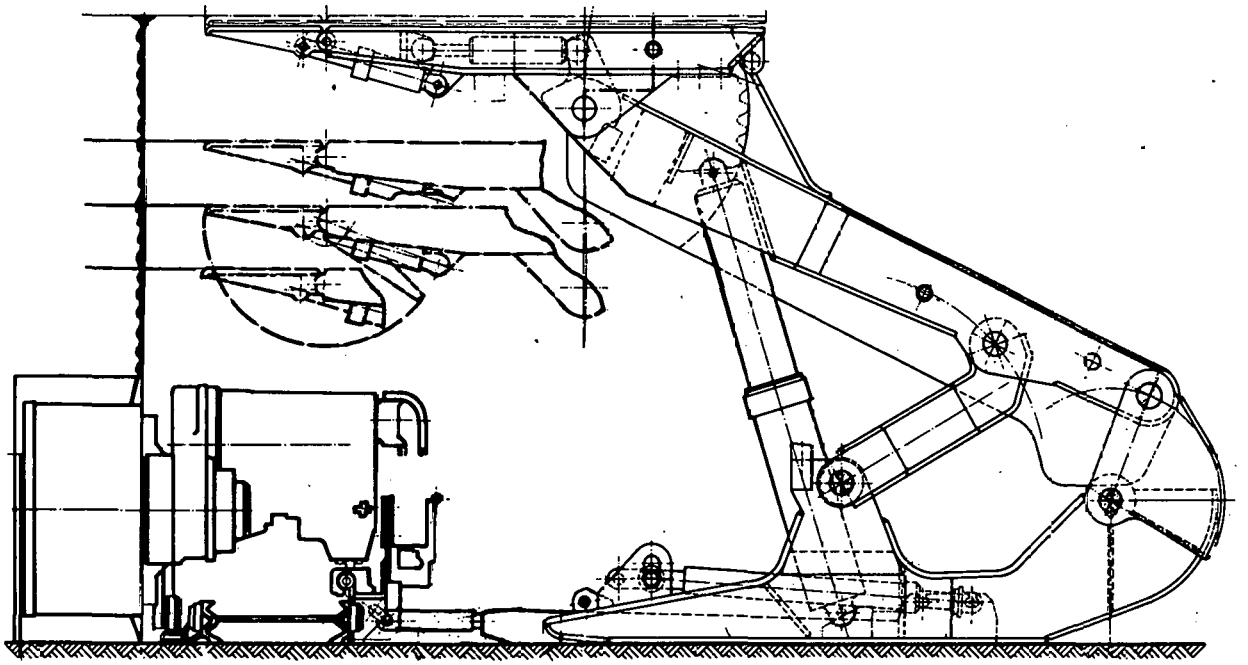
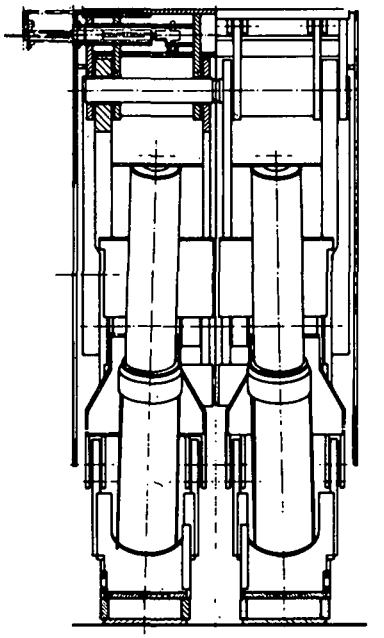


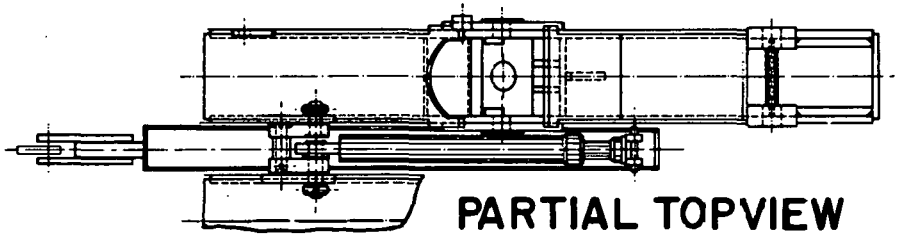

FIGURE 20 - Shield diagrams.



SIDEVIEW



FRONTVIEW



PARTIAL TOPVIEW
OF BASE

PG11-77
517

FIGURE 21 - Roof shield 6'/10'.

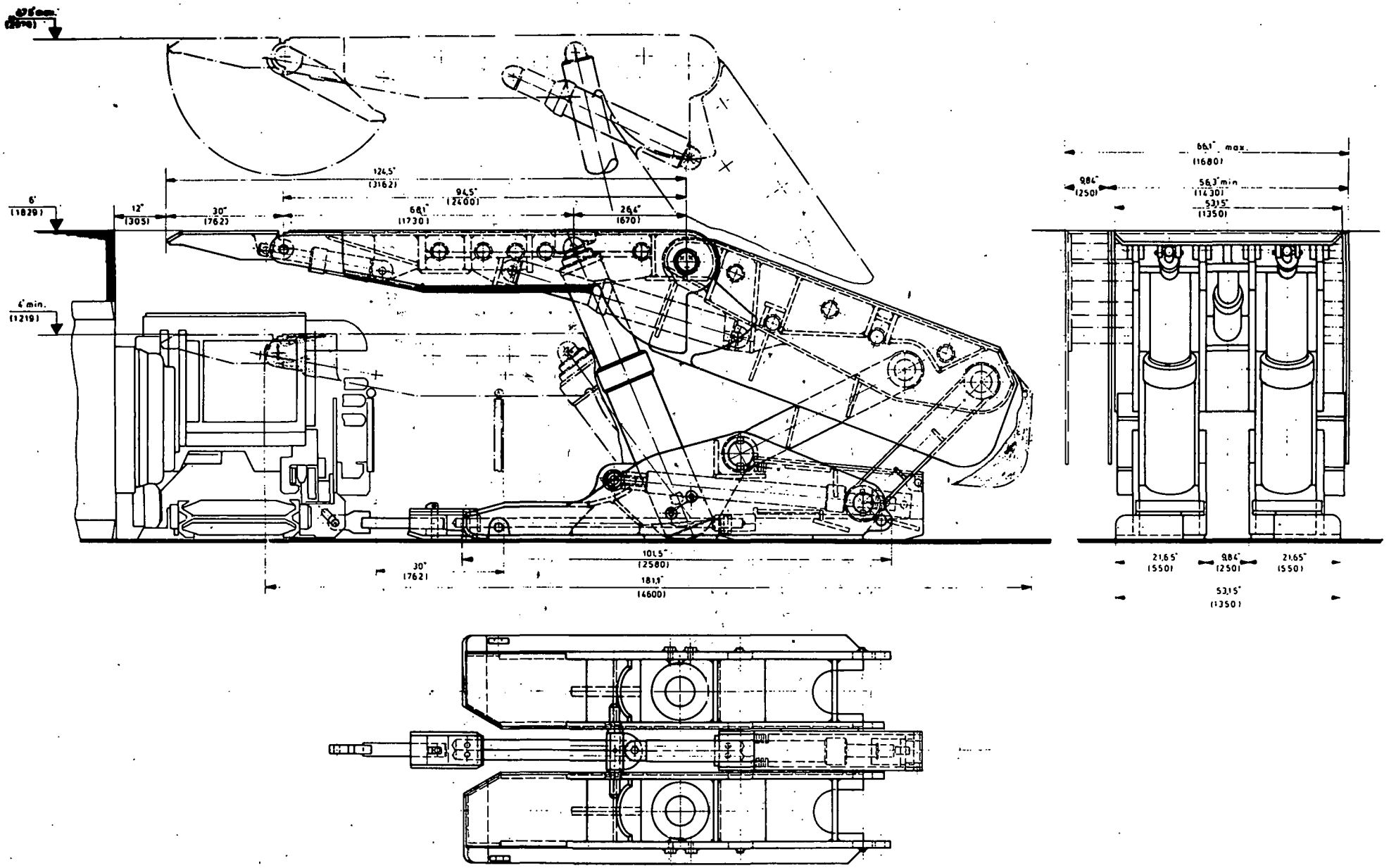
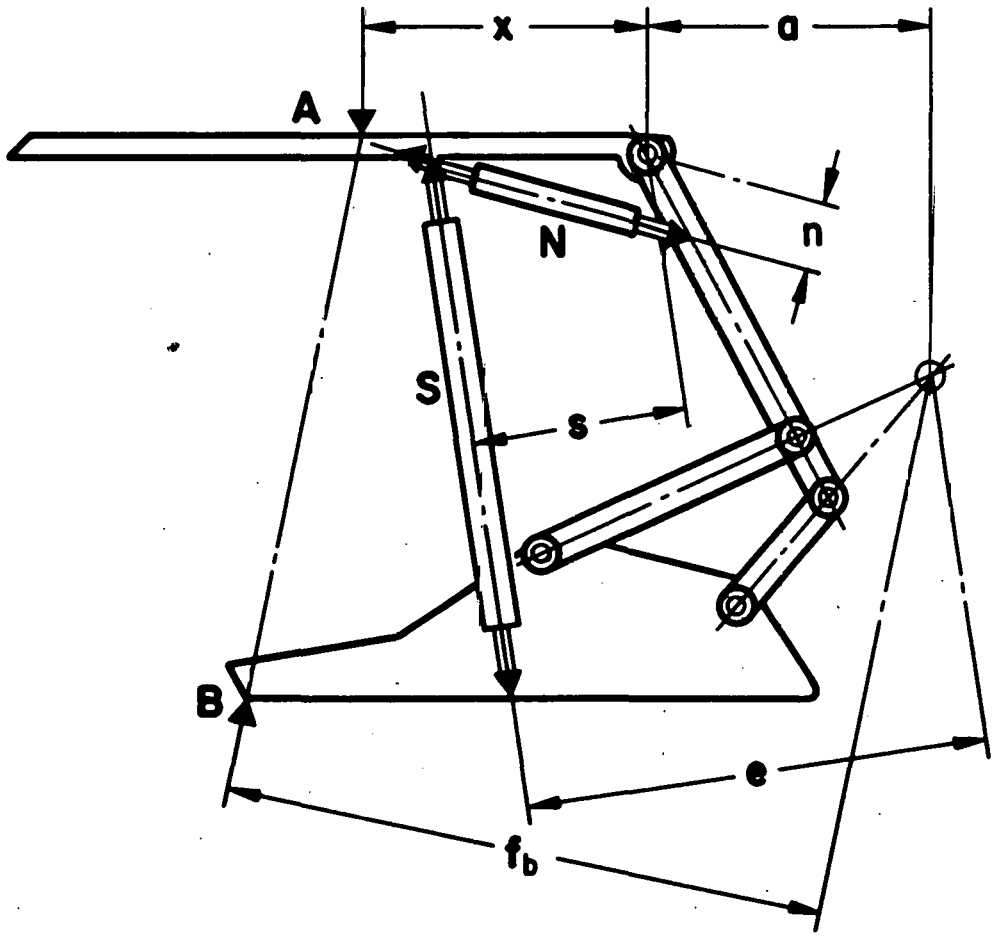



FIGURE 22 - Roof shield 4'/9.75'.




Lemniscate-2

FIGURE 23 - Lemniscate 2.

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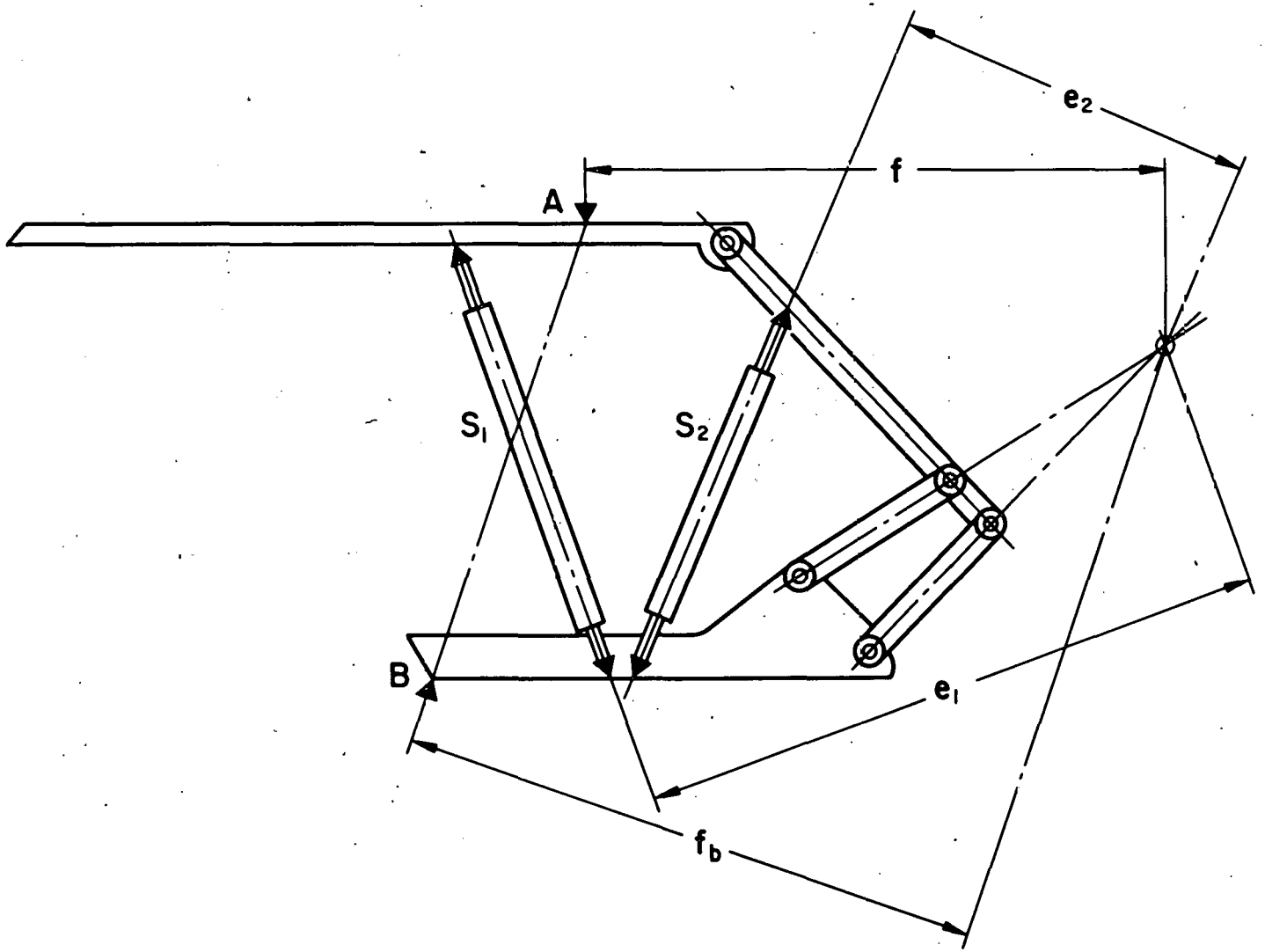


Four - legged

FIGURE 24 - Four-legged shield.

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$C_n$ = magnification coefficients

Magnification coefficients are identified as hanging of immediate roof, local face activity, bridging of immediate roof prior to first fall, main roof weight and extended downtime.

A mechanical disadvantage caused by the inclination of the shield legs reduces the vertical force exerted against the roof.

3. A short span of at least 12 inches between canopy tips and face before shearer passes.

4. A canopy designed to maintain stable roof contact during the shield advance by the action of a ram which swings it against the roof and by having a ratio of the front to the rear portion of the canopy not greater than ?.

5. Short length of shield canopies. Shields take fewer steps walking from gob to face than chocks with longer canopies. This reduces the "trampel" effect. German miners use this expression to describe roof deterioration above powered supports caused by frequent setting and lowering.

6. Upswept canopy extensions which provide close roof contact and, when retracted, allow an immediate change from the one-web-back to the non-web-back mode if required for roof control. The one-web-back operation provides more traveling space and immediate forward roof support. A non-web-back mode minimizes roof exposure in advance of the front props.

7. A working pressure equalling 80 percent of the support yield pressure to maintain a strong thrust against the roof immediately after exposure to prevent bed separation. The hydraulic system must have sufficient capacity to coordinate the speed of the shield advance with the shearer haulage speed and, thus, to minimize the delay of supporting the roof after exposure.

8. A yield pressure not to exceed 7,000 psi for reliable yield valve function.

9. A safe travelway.

10. A divided base which allows each half of the support to be lifted individually so that floor obstacles can be overcome and which is selfcleaning by allowing debris to pass through into the gob.

11. Control of each shield from the adjacent unit to protect the shield operator.

12. Having the double-acting ram mounted in an inclined position counteracts the tendency of the shield to dig into the floor. The ram acts through reverse linkage to raise the base slightly and to exert the full force of the piston end when the shield is pulled up to the conveyor.

13. Automatic sprays mounted on the canopies to control dust originating from the gob during shield advance and sealing of the working space against dust by sideplates fitted between the shields and kept tight by springs and hydraulic rams.

14. Collapsible for movement under trolley wire.

Roof supports must pass an intensive testing program in an appropriate laboratory which includes the following investigations (11): Forces exerted on the roof and floor, resistance against racking loads, support resistance, bending moments in the floor skids, penetration of the floor skids into the floor, strength of the shield

at one to three loading cycles, durability under pulsating loads, function and operation mode of the canopy, protection against flushing, width and height of the travelway, behavior of the shields on uneven floor and sideways mobility of double acting ram.

Roof supports should be equipped with load indicators, gages and recorders to trace the loading profile in the roof support systems by measuring hydraulic pressures.

Recovery of longwall roof supports is always a difficult task and requires circumspect planning and execution. Shield recovery poses an unusual problem because part of the structure is under the caved rock. Therefore, during the last 10 passes across the face, wire mesh is stretched over the top of the shields. A recovery room is then driven by a continuous miner along the shield line (fig. 25). The space is further secured by a movable bulkhead formed by two shields moving side by side along the recovery room, while a shield is removed from the shield line and taken off the face (fig. 26). This is an important safety feature. Another precaution taken is to build cribs in place of a removed shield. A third precaution taken is to advance shields from the line of supports under their own power. Under these conditions, shields can be set against the roof quickly should immediate support be needed (fig. 27).

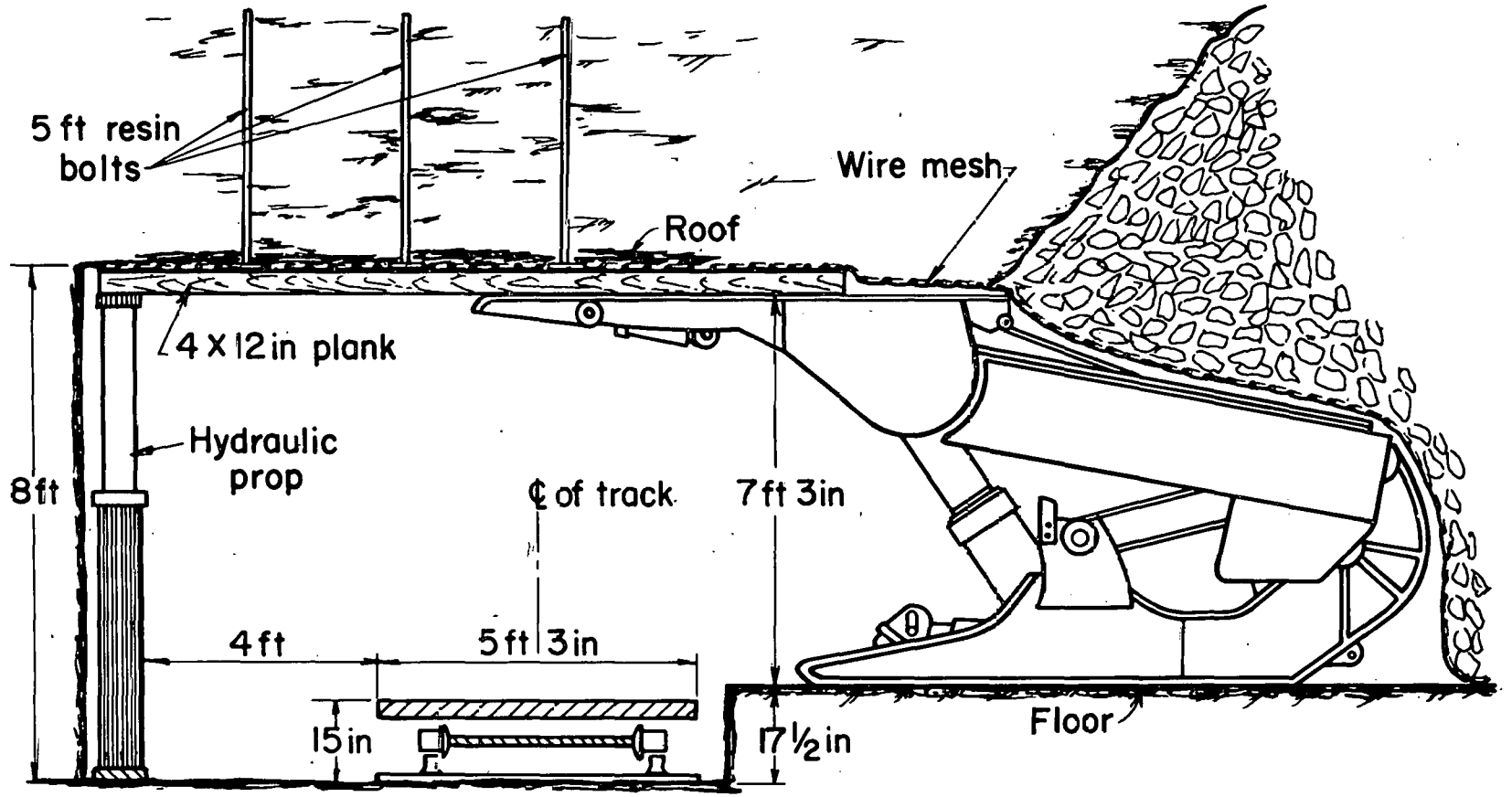
The shearer-loader is used predominantly on U.S. longwalls to cut and load coal. Only 25 percent of the longwall faces were equipped with plows in 1977, though the equipment is simple, the product is coarser and the operation is less dusty than with shearers. Shearers are used in firm coalbeds that do not lend themselves to plowing. The shearer operation also offers a better horizon control where the floor is soft, although the manufacturer of a novel "Gleithobel" plow claims that adequate horizon control and cutting performance in firm coal can now be achieved.

The double-ended ranging drum shearer provides face end formation without entering the gates. The machine frame is safely trapped by guide tubes on the gob side of the panline and rides on rollers that run on the L-shaped ramp plates on the face side. Novel in-seam shearers which ride on the floor ahead of the face conveyor are designed to cut coal as low as 30 inches.

Shearer haulage chain tensioners are mounted on the head and tail terminals of the conveyor and are connected to the hydraulic pressure system of the roof supports. The tensioners compensate for the elastic stretch of the chain which may amount to as much as 2.1 feet for a 470-foot face and a 26 millimeter chain. They provide uniform chain loading regardless of the postion of the shearer along the face. Chainless haulages have recently been introduced and promise benefits in both safety and efficiency.

Most shearer drums have two-start spiral vanes, are 30 inches in width and cut a 27- to 28-inch web. A wider web is desirable where roof control can be maintained. Each drum can be equipped with a cowl to clean ahead of the conveyor. Hollow drum shaft ventilation is recommended where methane liberation is high. Water sprays are integrated with the drums and also mounted on the machine frame. Fewer picks, slower drum rotation and picks as long as 4 inches are favorable factors in dust control.

However, the control of respirable dust often constrains the production from high capacity cutter-loaders. Additional methods available to allay respirable dust at the source include water with wetting agents, the use of on-board scrubbers and homotropal ventilation modes where coal and air travel in the same direction to reduce the relative air velocity and, thus, prevent aerodynamic entrainment of respirable dust.




Recovery area.

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FIGURE 25 - Recovery.

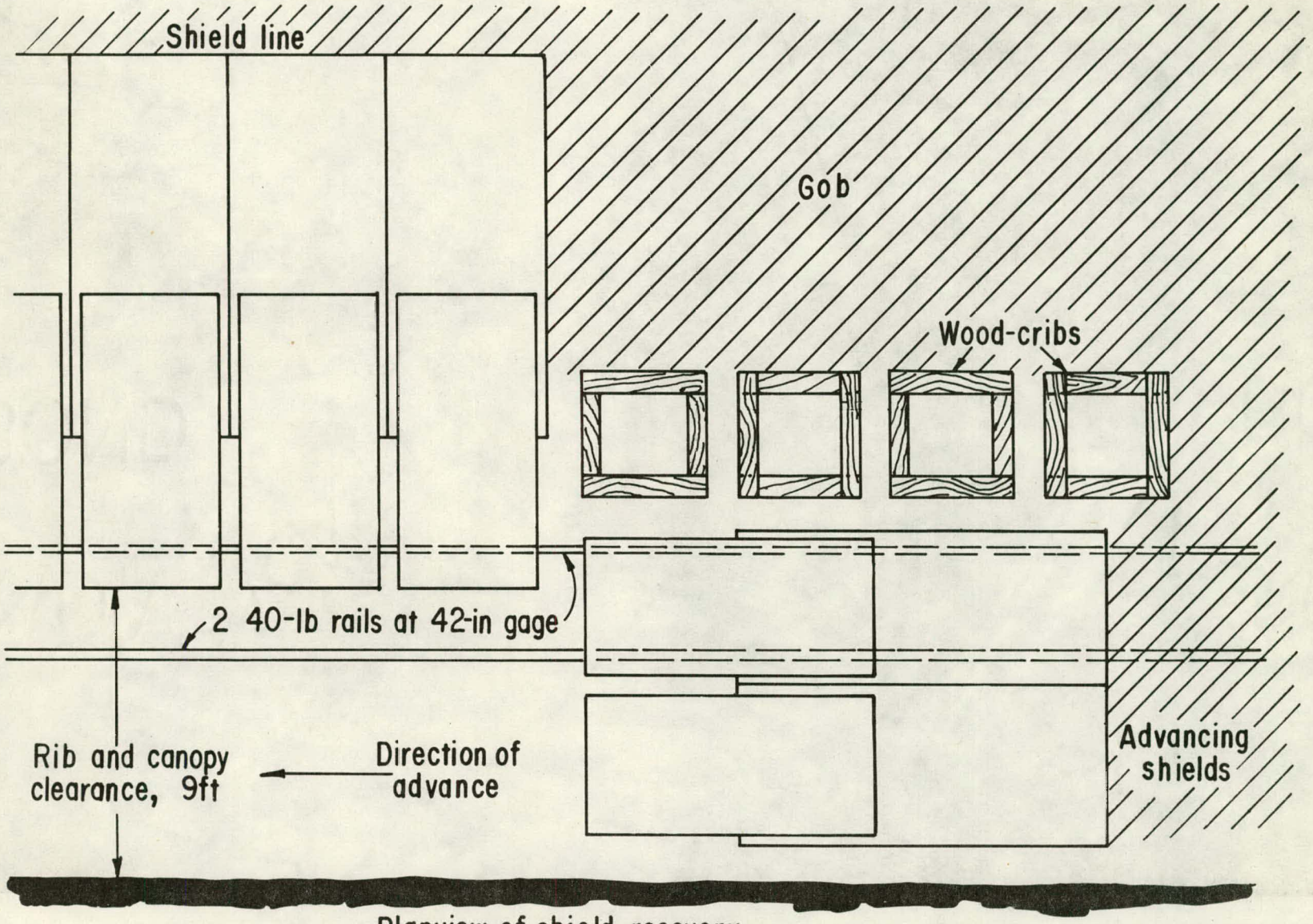
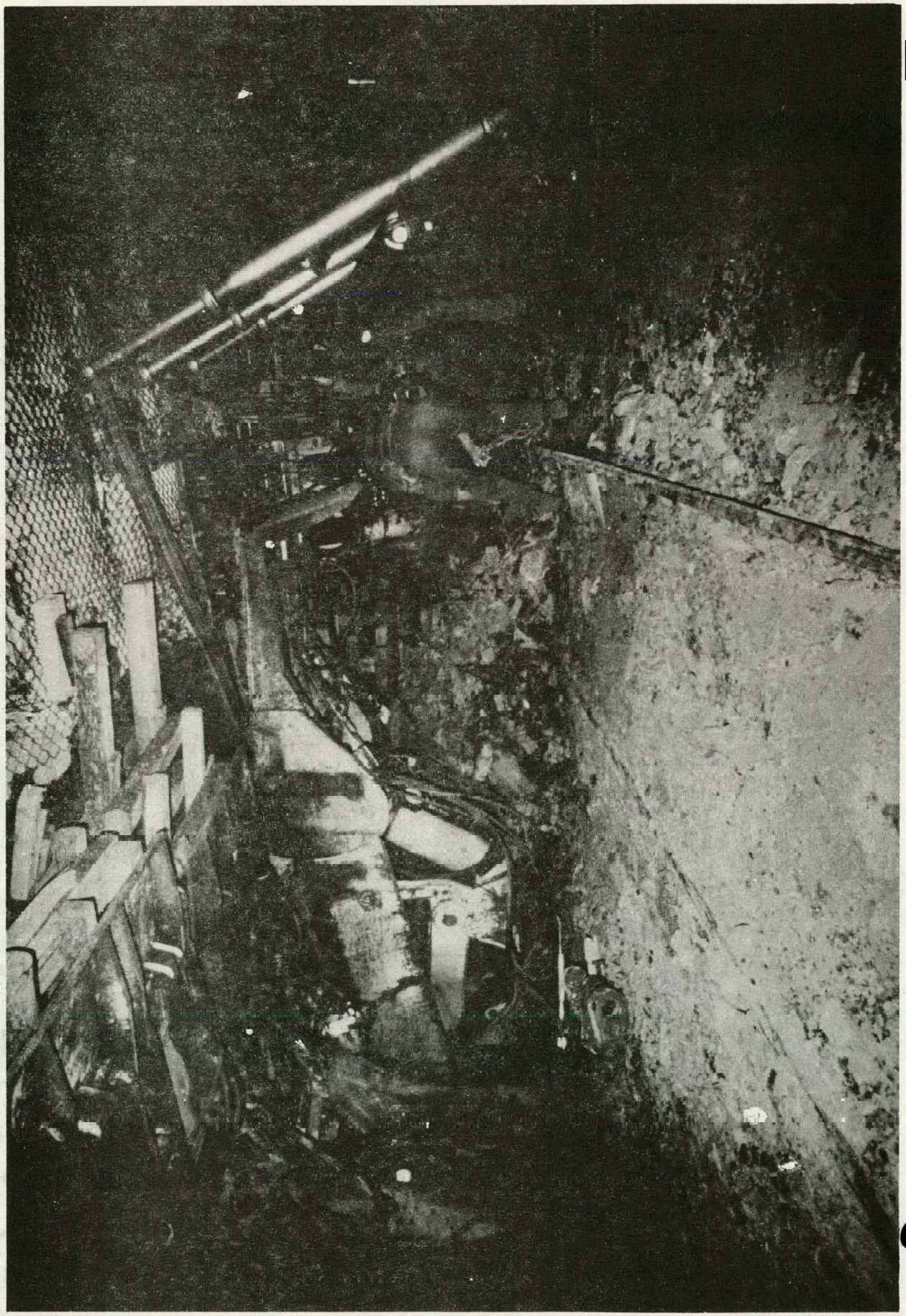



FIGURE 26 - Shield recovery.



High air velocities result from the restricted cross sectional area available along a face equipped with powered ruoof supports. The ventilation cross sectional area, A, is a function of mining height according to an empirical formula developed in Europe (28).

$A = 12.5\ (M-1)$ for chocks and four-legged shields, $ft^2$ (4)

$A = 10(M-1)$ for two-legged shields, $ft^2$ (5)

M = mining height, ft

Obviously, two-legged shields are at a disadvantage relative to those with chocks or four-legged shields in terms of face ventilation.

The unidirectional mode of operation practiced in Illinois is designed to keep crews in the intake air while the shearer is cutting. The shearer cleans its track on the non-cutting return pass (fig. 28).

High capacity armored face conveyors have single center, double inboard or outboard chains. Each standard pan is 59 inches (1.5 meter) long to accomodate one shield so that the middle of the pan can be attached to the double-acting ram, thereby minimizing the stress at the pan connections due to eccentric forces when pushing the conveyor forward or pulling up the shield. The conveyor snaking procedure which sumps the shearer into the face and involves about 12 pan lengths or 60 feet also stresses the pan connections. Damage to pan ends can be avoided by having connectors attached to the conveyor furniture rather than placing them on the pans (29). The furniture includes ramp plates on the face side and spillplates with the guide tubes, the cable handling trough and brackets for attaching the rods of the double-acting shield rams on the gob side.

Standard conveyor chain speed is 181 feet per minute. Power at the terminals is transmitted through fluid couplings and must be adequate to haul the coal away from the cutter loader. The National Coal Board's Bulletin on "Heavy-Armored Flexible Conveyor" offers power requirement calculations (30).

Stageloaders bridge the gap between the face conveyor and the panel belt by providing flexibility at both the receiving and delivery ends. The receiving end must be designed to prevent spillage from being carried back under the return strand of the face conveyor (29), where accumulation of debris can cause stoppage and chain breakage. The delivery end rests on a swivel dolly which rides on the belt tail piece. A lump breaker may be integrated into the stage loader.

To assure the safe performance and reliability of the shields and other face equipment, components and entire assemblies should be subjected to a battery of tests in an appropriate laboratory. In the United Kingdom, equipment to be used in National Coal Board mines must meet rigorous tests of performance and durability at their Bretby Rotary Laboratory and Swadlincote test site. Such tests also provide data on the adequacy of their exports.

Compatibility of a longwall system composed of equipment from several sources requires a special engineering effort to coordinate the installation and functioning of products not only from several manufacturers but also originating in different countries.

## COAL CLEARANCE AND OTHER SERVICES

Coal clearance from the faces affects production directly. Therefore, the use of

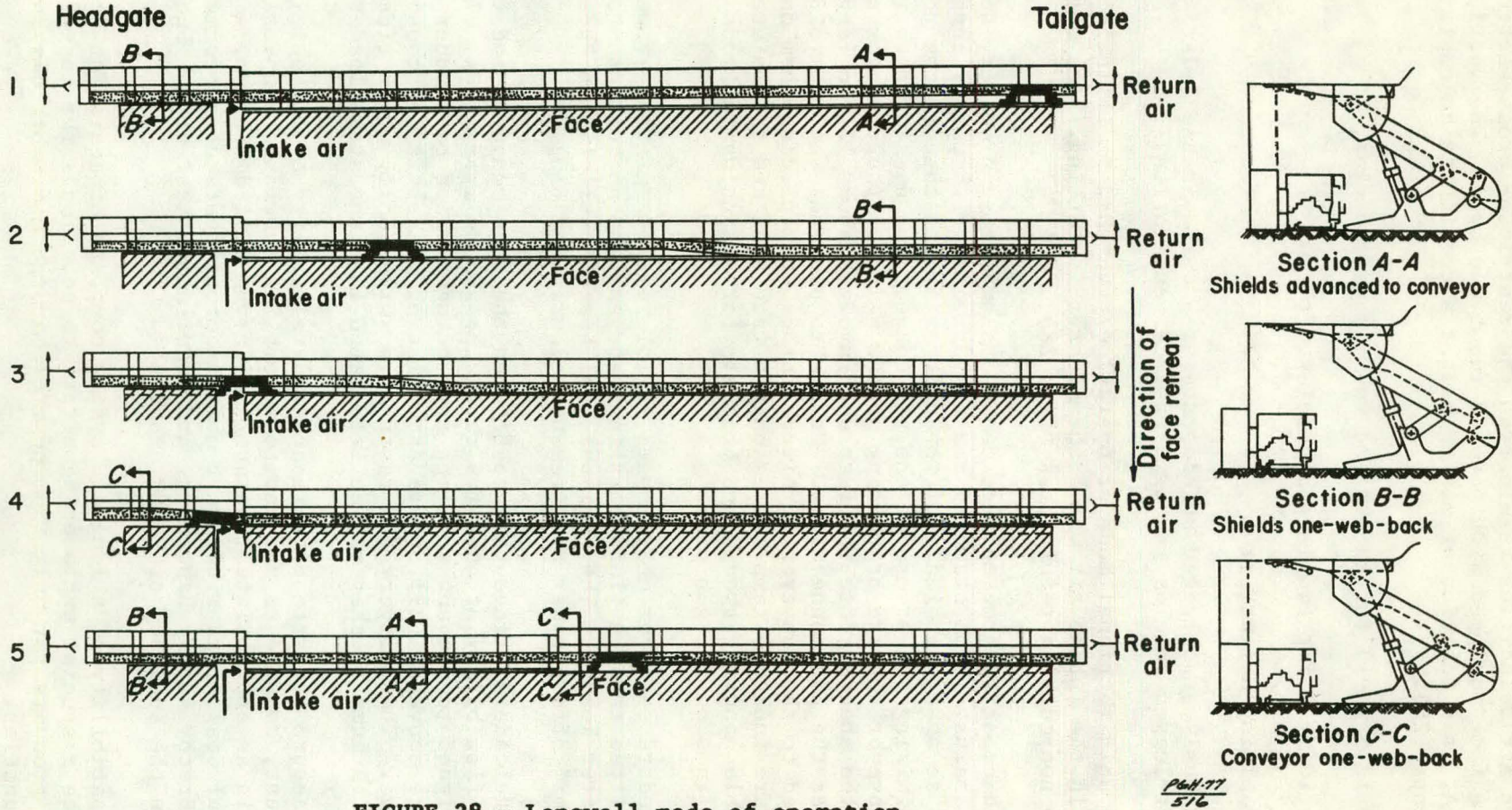



FIGURE 28 - Longwall mode of operation.

high capacity, 42-inch wide retractable sectional belt conveyors are common. They are equipped with mobile tail pieces and have solid structures suspended from the roof by chains. The combination of a sectional belt with main line track haulage has proven to be a very flexible system. In all-belt mines, the external belt system often presents a bottleneck to face production. Insertion of bunkers into the system can provide the needed flexibility. Such storage system are widely used in European mines such as the Bevercotes mine in the United Kingdom and the Haus Aden mine in Germany.

Remote monitoring and control can improve the availability of all belt systems. The system includes three elements:

1. Monitoring of fault conditions at transfer points.

2. Transmission of signals.

3. A central control console for collecting, storing and displaying data and issuing commands.

Monitoring by closed circuit television also has proven effective (31). The National Coal Board in the United Kingdom has developed computer assisted Mine Operation Systems (MINOS) (32) wherein an operator from his desk in the control room controls conveyors and bunkers and can call on stored information to provide a performance summary.

Track is installed in many mines solely to accomodate men and supply transportation. Some newer mines designed for extraction by longwall have trackless services. Power supply, 995 volt for face equipment where permitted by State law, must be adequate to match production goals. Face communication systems consisting of paging phones are placed along the full length of the shield line.

Face lighting systems must meet mandatory requirements and be approved by an MSHA Statement of Test and Evaluation. The required minimum luminous intensity is 0.06 fL (foot Lambert). Incident photometers read the incident light in fc (foot candle). Each luminaire in a longwall sheds light on a space bounded by surfaces of the roof, floor, coal face and the shield. The foot candle average of the photometer readings must satsify the formula:

$$\text{mean face measurement} \times \text{light loss factor (0.77 = 77 percent)} \times \text{reflectance of underground surface (0.04 = 4 percent)} \geq 0.06 \text{ fl} \quad (7)$$

$$\text{Minimum incident light required: } \frac{0.06}{0.04 \times 0.77} = 1.95 \text{ fc}$$

For example, to illuminate a longwall space having a roof span from face to gob of 8 feet, a floor span from face to back of 12 feet, a mining height of 7 feet, and luminaire spacing of 10 feet, the area A to be lighted is: (10 x 8) + (10 x 12) + (2 x 10 x 7) = 340 sq ft and minimum lumens required are:

$$A \times fc \times 2 \text{ (nonuniformity of coverage factor)} = 340 \times 2 \times 2 = 1{,}360 \text{ lumens} \quad (8)$$

A system with mercury vapor lamps has been developed and introduced at Illinois longwall operations (33). It features a minimum of luminaires and electric cables, is rugged and easily accessible, and the components including lamps, ballasts and powerpacks do not obstruct the travelway. The luminaires illuminate the total area but do not cause glare to the miners.

## MONITORING AND ANALYZING

Operational data extracted from shift, maintenance and supply records are compiled into monthly summaries. The type of delays and repairs required can be code for immediate computer input.

A ground control monitoring program is designed to achieve the following objectives:

1. To provide early warning capability by measuring strata movement and pressures during mining so that corrective measures could be taken quickly, should an unusual situation arise.

2. To evaluate the adequacy of the roof support system and to derive design criteria for future longwall mining efforts.

The program includes the following tasks:

1. Measuring convergence in gate roads and differential rock and floor strata movement to detect bed separation.

2. Measuring in situ pressure by installing stress meters of the vibrating-wire gage type in coal and floor of the longwall panel and chain pillars.

3. Tracing the loading profile in the roof support system by measuring hydraulic pressures with load indicators, gages and recorders (fig. 29).

4. Determining vertical extent of caving by boreholes from surface to coalbed either by Time Domain Reflectometry (TDR) or by Multiple Point Borehole Extensometers (MPBX) (34).

The effect of mining on the surface over the extraction area is monitored by surveys to measure vertical and horizontal surface movements, including tilting of monuments. Surface strains between selected monuments are measured by extensometers or strain gages. Automatic Data Acquisition Systems (ADAS) have not yet proven fieldworthy (35). The shape of the subsidence trough and the angle of draw are to be determined to indicate whether the mining will affect surface structures.

## CONCLUSION

Mine design with risk reduction and personnel safety in mind has encouraged the growth of longwall mining in the Illinois Coal Basin from one face to four in just 3 years. The low accident rate proves that the longwall system is safer than room-and-pillar mining. The use of shields has been a major factor contributing to adequate roof control. Short spans of exposed roof, quickly applied support, stability of structure, and full shelter for face crews are some of the significant advantages that led to the adoption of shields.

The ingredients to the success in reintroducing longwall mining in southern Illinois are the ingenuity and confidence of the Old Ben staff, the spirit of cooperation between labor and management, and the support of the Department of Energy as a partner in a cost-sharing endeavor.

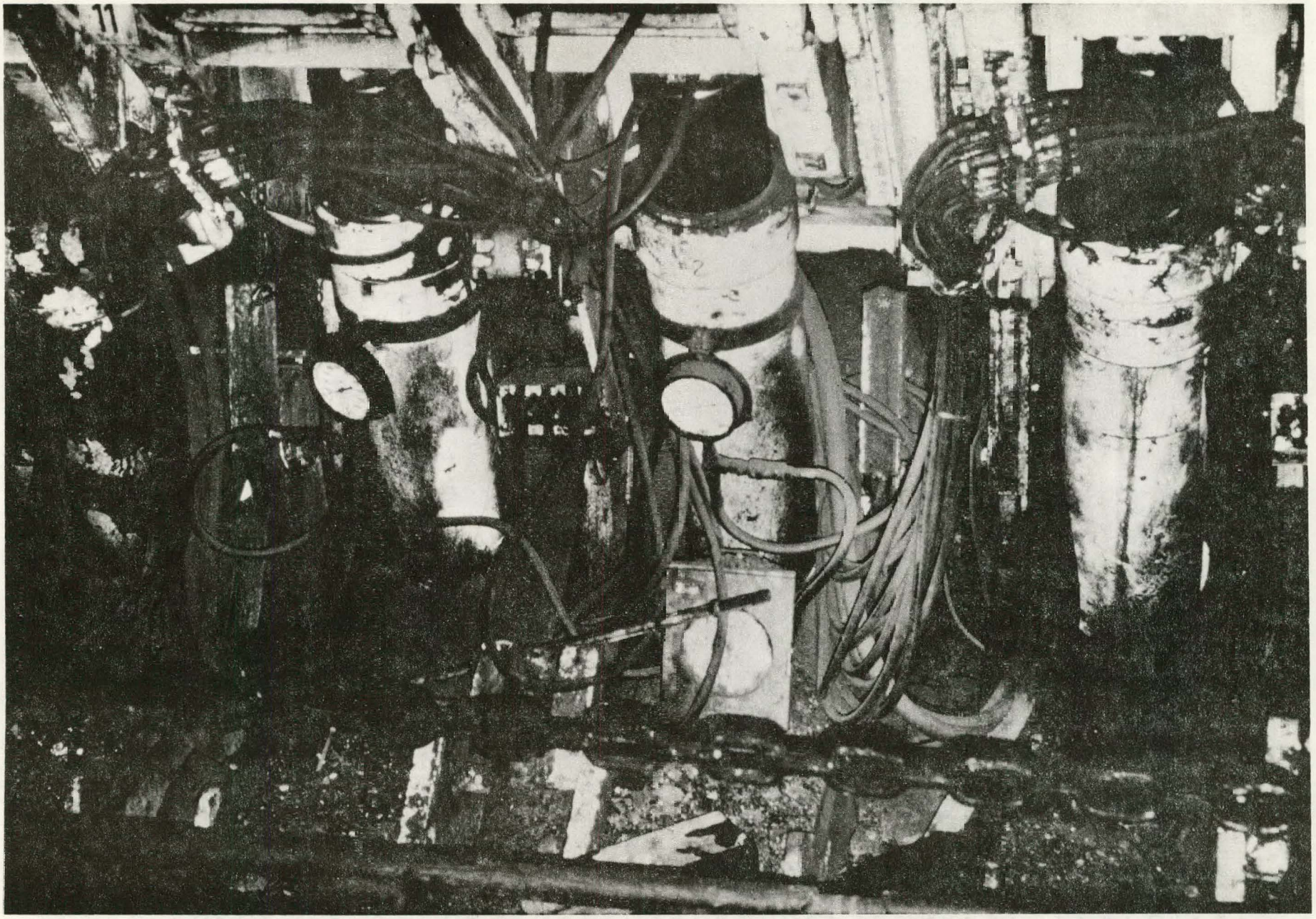


FIGURE 29 - Hydraulic pressure gage
and recorder on shield.

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# ROCK MECHANICS STUDIES AT OLD BEN COAL COMPANY MINE 24

Leroy P. Kettren
Kerry A. Johnston

Dames & Moore
Park Ridge, Illinois

## INTRODUCTION

Rock mechanics studies were performed by Dames & Moore in Old Ben Coal Company's Mine No. 24 as part of a longwall mining demonstration with the U.S. Department of Energy. The demonstration project was designed to introduce longwall techniques to the Illinois Basin where earlier attempts with longwall had failed. Six earlier attempts with longwall methods by Old Ben Coal Company had been unsuccessful due to roof control problems. These attempts were described by Moroni (1973). The present demonstration included a rock mechanics study designed to achieve the following purposes:

1) To observe unusual situations developing during mining so that corrective action could be taken.

2) To develop guidelines for future longwall efforts.

3) To evaluate surface subsidence, its character, rate and effects on surface features; and to develop criteria to assist in predicting subsidence.

## LONGWALL DEMONSTRATION AREA

Old Ben Coal Company's Mine No. 24 is located approximately 2 miles northwest of Benton in Franklin County, Illinois and is producing coal from the Herrin No. 6 Seam. The coal is approximately 8.5 feet thick. The floor consists of 1.0 to 1.5 feet of gray silty clay over 12 feet of hard, gray, very fine-grained nodular limestone. The immediate roof is comprised of two shale strata. The thin, gray, silty Energy Shale occurs as pods about 5 to 10 feet thick. The other is the hard, black, thinly laminated Anna Shale, about 4 feet thick. Above the shales is the hard, massive Brereton Limestone, generally 4 to 10 feet thick. The total thickness of the overburden above the mine is 620 feet and is comprised chiefly of siltstone and shale.

The Rend Lake Fault system is located approximately 2,000 feet west of the longwall panels with two spur faults occurring in Panel 2. Both faults had a north-south strike parallel to the panel centerline and dipped steeply to the west. Total vertical displacement on these faults was between 5 and 10 feet.

A more complete description of the geology in the demonstration area was given by DeMaris and Bauer (1977).

The longwall demonstration included three panels with Panels 1 and 2 being instrumented in the rock mechanics study. The panels were aligned in a north-south direction. Panel 1 measured 1,735 feet long by 450 feet wide. Panel 2 measured 1,713 feet long by 460 feet wide. Mining of Panel 1 began in September 1976 and was completed in May 1977. Panel 2 was mined beginning in August 1977 and was completed in December 1978.

## SURFACE INSTRUMENTATION

### General

The surface instrumentation was designed to monitor the development of roof cave and to observe the effects of subsidence at the surface. The instrumentation used to observe roof caving consisted of a 7/8-inch diameter coaxial cable grouted into each of two boreholes drilled from the surface. The cables were periodically monitored using Time Domain Reflectometry (TDR). Surficial subsidence was measured using a system of monuments placed in three rows over the panels. These monuments were installed and monitored by Old Ben Coal Company personnel.

### Time Domain Reflectometry

Time Domain Reflectometry (TDR) was used to monitor roof cave after mining. A 7/8-inch diameter coaxial cable was installed in each of two boreholes drilled from the surface through the No. 6 coal. Borehole 1 was on the centerline of Panel 1 approximately 230 feet from the start of the panel. Borehole 2 was on the centerline of Panel 2 approximately 800 feet from the start of the panel. The cables were monitored using a Tektronix Model 1502 TDR cable tester. The device transmits an electrical signal which is deflected by any cable damage. Calibration of the instrument allows location of the exact point of the cable fault. Mining of the coal and collapse of the strata damaged the cable and could be monitored with the TDR. Measurements were taken daily with the TDR beginning when the face advanced to within 70 feet of the cable and continued until no additional strata movement was observed. Figure 1 shows the results of the TDR measurements made on Panel 2.

Several differences were observed in the TDR records made over the two panels. Damage to the Panel 2 TDR cable occurred in advance of the face. This effect was not evident in Panel 1 due to the relative location of the two boreholes. The TDR cable in Panel 1 was located only 230 feet from the beginning of the panel, a distance slightly greater than the critical width. The subsidence trough was just beginning to form, and the draw angle had not sufficiently developed ahead of the face before the shearer mined through the cable. The Panel 2 TDR cable, however, was located approximately 800 feet from the beginning of the panel, a distance much greater than the critical width. As the face approached the cable, the subsidence trough was well developed and draw was well established ahead of the advancing face. In Panel 2, major cable damage occurred 300 feet above the coal as the face approached to within about 40 feet of the cable.

TDR records also revealed differences in the height of the roof break as shown by cable separation. The maximum height of roof break above Panel 1 was about 410 feet above the coal first recorded after

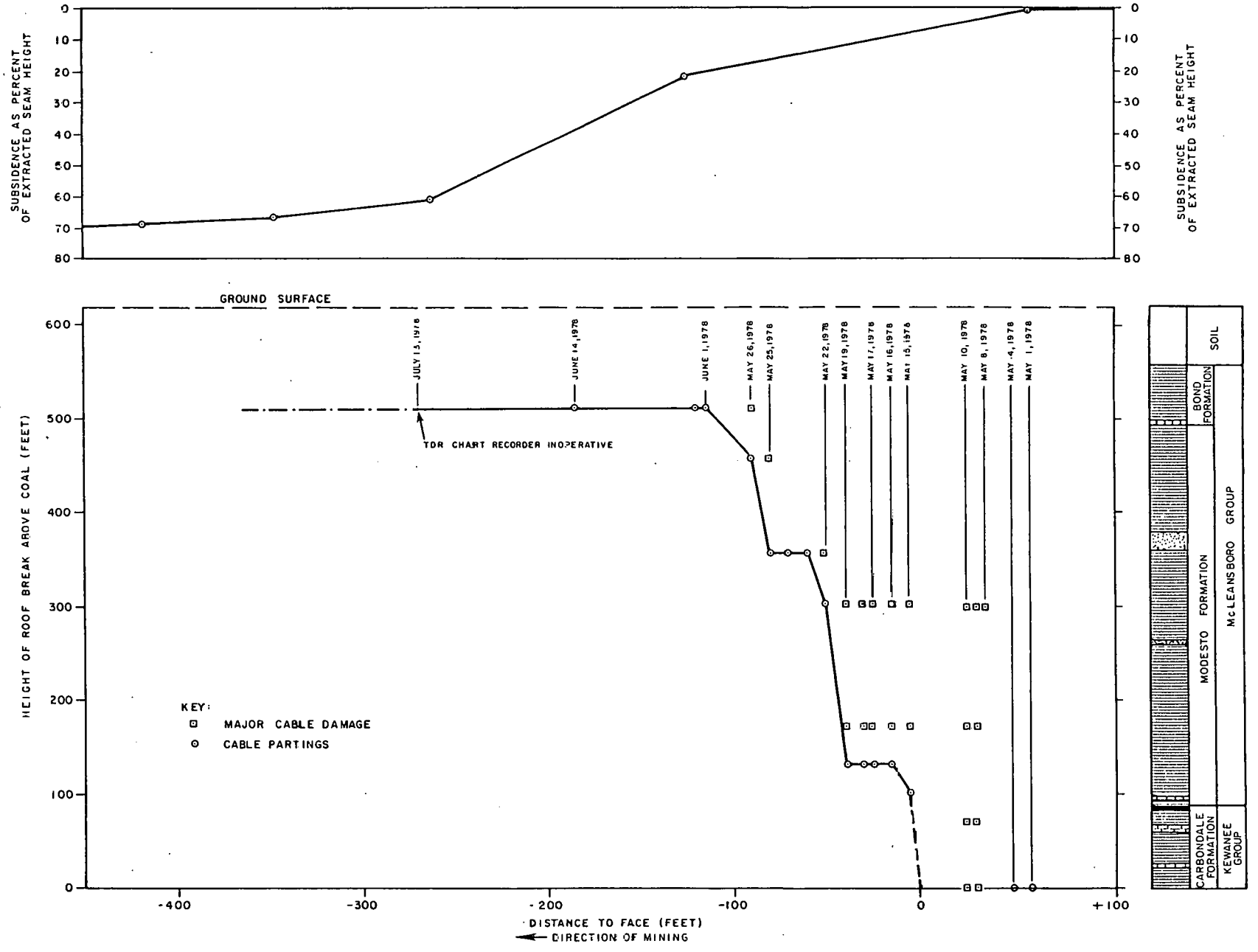



FIGURE 1 ROOF BREAK vs FACE ADVANCE, PANEL 2

the face advanced approximately 120 feet past the TDR cable. The maximum height of roof break above Panel 2 was about 510 feet above the coal. Similar to Panel 1, the maximum height was reached when the face was about 120 feet past the cable.

### Surficial Subsidence

The extent of subsidence was determined using a series of 196 monuments installed over both panels. One row was located over the centerline of Panel 1, a second row over the centerline of Panel 2, and a third row perpendicular to the other rows at the midpoint of the panels. All three rows were extended beyond the panels to better define the limits of subsidence. The monuments were installed by Old Ben Coal Company personnel who also performed monthly level circuits and supplied the reduced data to Dames & Moore. The maximum subsidence observed over Panel 1 was 4.71 feet on December 4, 1978. The maximum subsidence observed over Panel 2 was 5.21 feet, also on December 4, 1978. The maximum subsidence ratio for Panel 1 was 0.67, or 67 percent of the extracted seam height of 7 feet. The maximum subsidence ratio for Panel 2 was 0.74.

## SUBSURFACE INSTRUMENTATION

The subsurface instrumentation program was designed to measure pressure in the support hydraulic system; changes in stress in coal and floor; differential strata movement; and roof-floor convergence. The program was modified after completion of Panel 1. The work on Panel 2 included measurement of pressure in the support hydraulic system and changes in stress in the coal and floor.

### Support Loads

Support loads were calculated from records of hydraulic pressure in the shield legs corrected to canopy height. Each instrument installation consisted of two hydraulic pressure gauges and a continuous 7-day, two-pen recorder. One pressure gauge and one pen of the recorder measured pressure in each of the two shield legs, making the system capable of measuring differential loads on the shield. On Panel 1, instrument installations were placed on 10 shields spaced at regular intervals along the face. Eight shields were instrumented.

Hydraulic pressure records used to calculate shield support loads revealed that pressures in the shield legs were usually below the nominal setting of 4,350 psi. The yield pressure of 5,510 psi was reached or exceeded only in a few instances during mining. Generally, in both Panels 1 and 2, the shield pressure across the face did not appear to follow any consistent pattern.

Increased loads were encountered on Panel 1 as the shields moved into the recovery room. The canopy loads generally remained below nominal setting load until approximately 10 feet of coal remained to be mined. The shield hydraulic pressure records showed a rapid pressure increase during the last four passes of the shearer. The canopy loads continued to increase after completion of mining and, in several cases, exceeded the nominal yield capacity of the shields.

### Stress Measurement

Stress changes were measured in the coal seam and in the floor at various locations around both panels using vibrating wire stressmeters.

Stresses in the panels began to increase when the face was approximately 200 feet from the instrument. The rate of increase accelerated to a maximum rate 50 feet from the face. The maximum stresses occurred in the panel approximately 9 to 10 feet in front of the face.

The stressmeters placed in Panel 2 obtained somewhat different results from those placed in similar positions in Panel 1. Stressmeters placed in pillars at the bleeder entries of Panel 2 showed greater stress increases than those placed in Panel 1. In Panel 1, stressmeters recorded an average stress increase of 650 psi, while stressmeters in the Panel 2 bleeders recorded stress increases above 2,500 psi, as shown on Figure 2. The difference is believed to be due to the presence of coal balls or calcium carbonate petrifications of peat found in Panel 2. The coal balls are very numerous in the Panel 2 bleeders and will support a high load before failure.

Stressmeters in the floor beneath Panel 1 recorded gradual stress increases which reached a maximum approximately 50 feet ahead of the face. As the face passed, the instruments recorded stress below the original setting pressure followed by a gradual increase until the gauges recorded stresses approximately equal to the original overburden pressure. This pressure was reached when the face had advanced about 250 to 300 feet past the gauges. Figure 3 shows the results of stress measurements in the floor beneath Panel 2.

### Convergence

Convergence rates measured in entries out by the face showed increases when the face was approximately 50 feet away. Convergence commonly measured less than 1 inch, although some stations measured convergence greater than 2 inches. In all cases, the convergence is believed to represent floor heave resulting from underclay movement.

### Differential Movement

Differential movement measurements were made on both floor and roof strata around Panel 1. Multiple point wire extensometers were installed in 2-inch diameter holes drilled into the roof. Rod extensometers were used in the mine floor. Analyses indicated differential movement of roof strata to be very small or negligible. Measurements of floor strata movement indicate that the majority of floor heave results from squeeze of the underclay between the coal pillar and the lower limestone strata.

## CONCLUSIONS

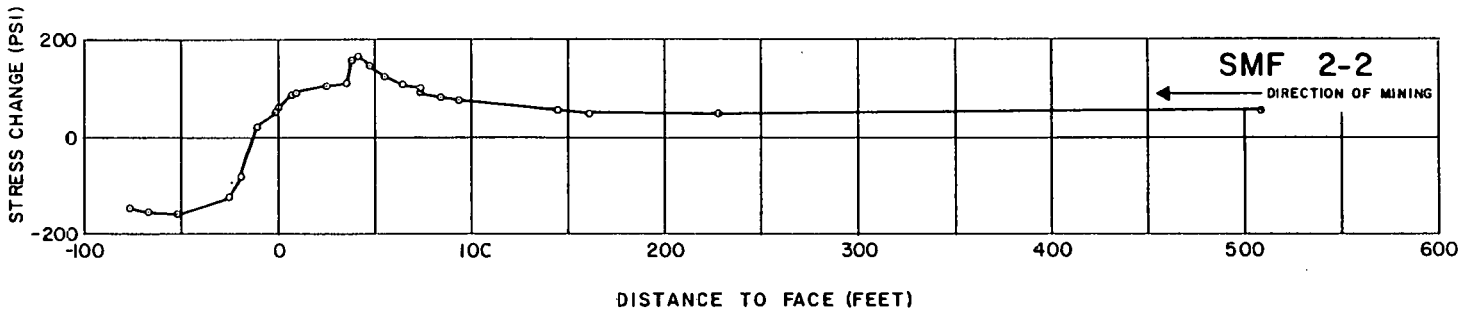
Significant differences, as well as similarities, were observed in the data from the two panels.

1) The subsidence observed over Panel 2 was greater than that observed over Panel 1. Panel 1 produced subsidence equal to 67 percent of the extracted seam height, while Panel 2 produced subsidence totaling 74 percent.

2) TDR measurements showed that once a critical opening width was reached, strata separation occurred in advance of the face. The TDR also showed that strata separation progressed closer to the surface on Panel 2 than on Panel 1.

These observations are reasonable. Panel 1 was developed between a recently completed pillar

FIGURE 2 STRESS CHANGE WITH FACE ADVANCE PANEL 2 BLEEDER ENTRIES



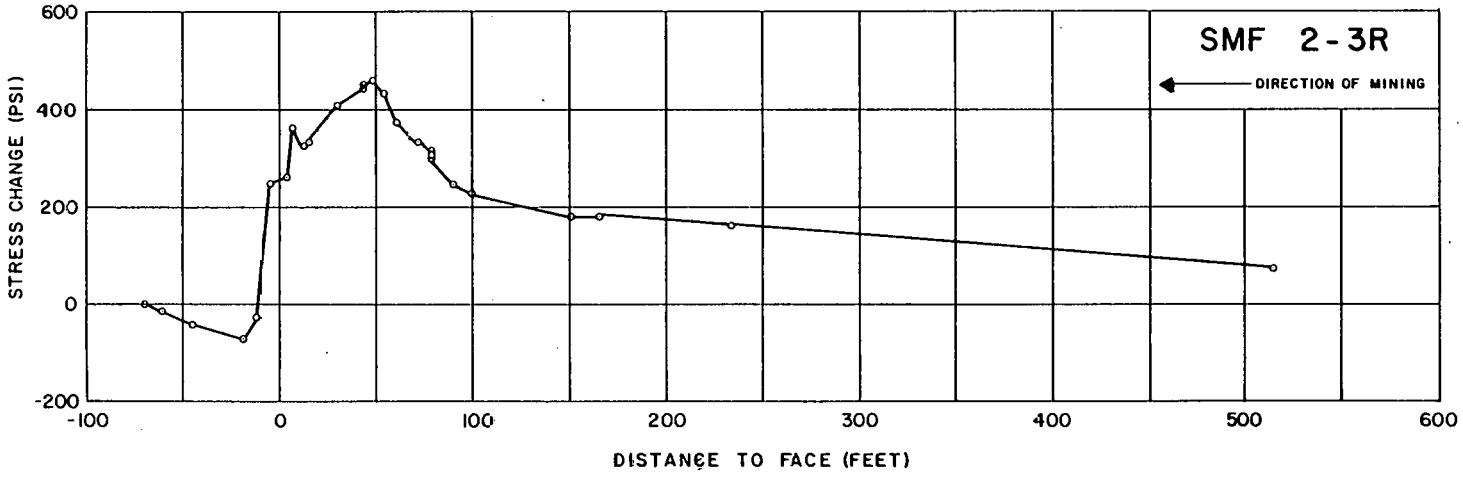




FIGURE 3 STRESS CHANGE WITH FACE ADVANCE, FLOOR BENEATH PANEL 2

extraction section and the unmined Panel 2. When Panel 2 was mined, the overburden reflected the reduced support provided by the broken strata over Panel 1.

3) Mining of Panel 1 produced an initial major roof fall when the face had advanced approximately 130 to 140 feet. The fall was accompanied by temporary stress relief on the bleeder pillars and by floor heave in the bleeders. On Panel 2, there was no observed initial major roof fall. The immediate roof broke early and easily. The more highly fractured strata above Panel 2 may be the result of the faulting encountered in the panel and the effect of the subsidence over Panel 1.

4) Stresses measured in Panel 2, in adjacent pillars and in the gob, commonly were greater than those measured on Panel 1. This observation is consistent with that reported by Carman (1965), who stated that abutment pressure increases in successive adjacent panels until a maximum is reached and the pressure is released.

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# FACTORS INFLUENCING THE LOAD ON LONGWALL SUPPORT

Tadeusz Ryncarz

Visiting Professor of Mining Engineering
Southern Illinois University at Carbondale
Carbondale, Illinois

## INTRODUCTION

Longwall mining is relatively new mining method in United States. Early trials were not successful, mainly because the face supports did not have sufficient capacity. At that time some cases were reported of supports being completely compressed by the heavy roof.

In the late 1960's the longwall mining was successfully reintroduced when the high capacity, self advancing powered supports were available. Since then the number of mines employing the longwall method has steadily increased. Contrary to U.S.A. practice longwall mining is the predominant method in European coal mines.

In the future longwall mining is certainly going to play a more substantial role in underground coal mining in the United States, however for the successful employment of longwall systems to underground extraction of coal seams certain conditions have to be fulfilled. To the most important conditions belong the proper assessment of the interaction between the powered supports and surrounding rocks and the appropriate evaluation of required support capacity.

Despite the large experience gained in European coal mining there still does not exist a uniform and commonly accepted method for estimating support requirements. It is known that a direct transfer of practical results from one mining region to another characterized by different geologic conditions may lead to unexpected failures. This precisely happened when the effort was made to introduce medium load capacity supports, which performance was already well proved in European deep coal mines, into shallow underground mines in United States. It has been found by trial and error method that in the U.S. coal mines the required load capacity is generally greater than the one applied in European deep coal mines.

Needless to say, that such statement must sound startling to the miners, who are accustomed to the simple idea that the gravitational rock pressure increases with the increase of depth and that the unit weight of rocks forming the coal measures is almost the same in Europe and in United States. No satisfactory answer has, as yet, been found to the question why at much lower depths a much greater support load capacity is needed and what factors are accounted for.

In this paper an attempt is being made to find certain explanation to the question posed above. It is based on a brief consideration of strata behavior above the longwall workings and on analysis of some methods predicting the load acting on longwall supports.

## STRATA BEHAVIOR ABOVE THE LONGWALL WORKINGS

Starting from a sufficient wide opening the longwall mining of a panel proceeds toward the solid coal and the width of the empty gob opening continues to widen. Having lost their natural support the rock layers in the roof start to sag and separate (Fig. 1). If the open roof area exceeds a critical size the roof layers begin to break and cave in the gob. The roof continues to cave as the longwall face advances and the support units move forward (1).

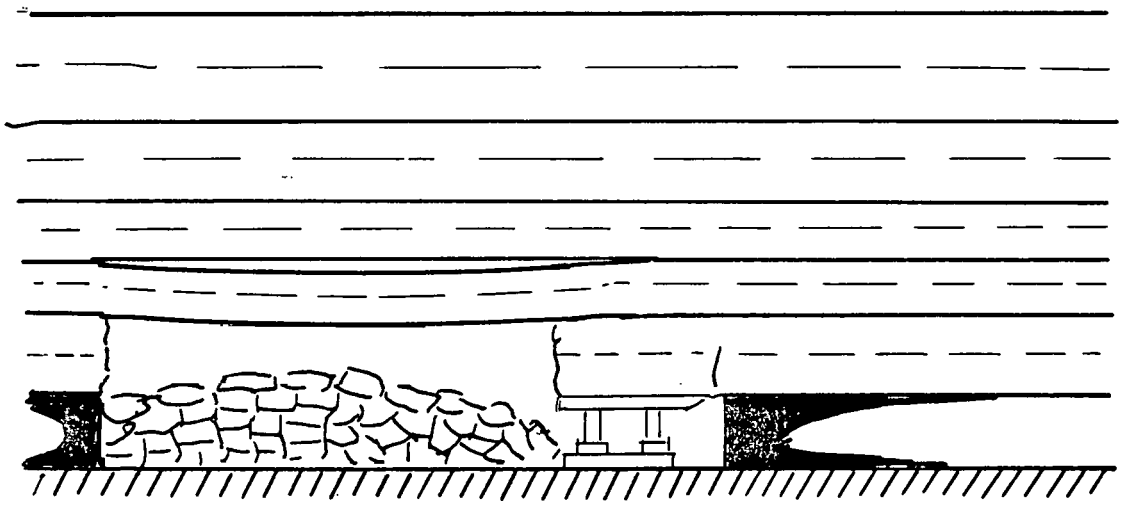

FIGURE 1.

Trying to asses the strata behavior above the longwall workings it is essential to know how far in the roof does caving stop. The answer to this question is very often based on the following assumption. Due to the breaking of solid rock the rock volume increased by a substantial part. Thus the broken rock fills up the void left by the extraction of the coal seam and provides support for the above lying, still unbroken roof layers which form the, so called, main roof. This simple assumption provides the possibility of expressing the height of the caving zone in terms of the coal seam thickness and the bulking factor of broken rock.

$$h_c = \frac{m}{k-1} \qquad (2.1)$$

Where

$h_c$ - height of the caving zone
$m$ - coal seam thickness
$k$ - bulking factor of broken rock

Assuming, for instance, 50% volume increase, and this corresponds to the value k equal 1.5, one gets the height of the caving zone equal twice the coal seam thickness.

According to this concept the strata above the coal seam is divided into two distinctive parts: the immediate roof and the main roof. The rock layers of the immediate roof cave in the gob area or rest on support units in the face area. The main roof, remaining almost in its original position, is bridged between the solid

coal in front of the face and at some distance in the gob where caved fragments are compacted enough to serve as its support.

However, recent investigations carried out in mines and in laboratories on phys ical models have not confirmed the existence of a stabilized main roof above the longwall workings. (2,3,4). It is well known, that the surface above the longwall workings subsides and the subsidence factor takes a value between 0.4 and 1.0 depending on many factors (overburden stratigraphy, geometry of mine workings, depth of the workings, etc). Thus, the subsidence of all rock particles being below the surface must be, at least, of the same magnitude. Measurements taken in the boreholes, drilled either from the surface or from an overlying roadway to the coal seam, showed a more or less continuous downward movement of the strata above the gob area (Fig. 2). The magnitude of this movement decreases with the distance from the coal seam. A similar pattern is shown by relative differential movement (strain). This latter movement accounts for loosening of strata structure and for apparent increase of rock volume.

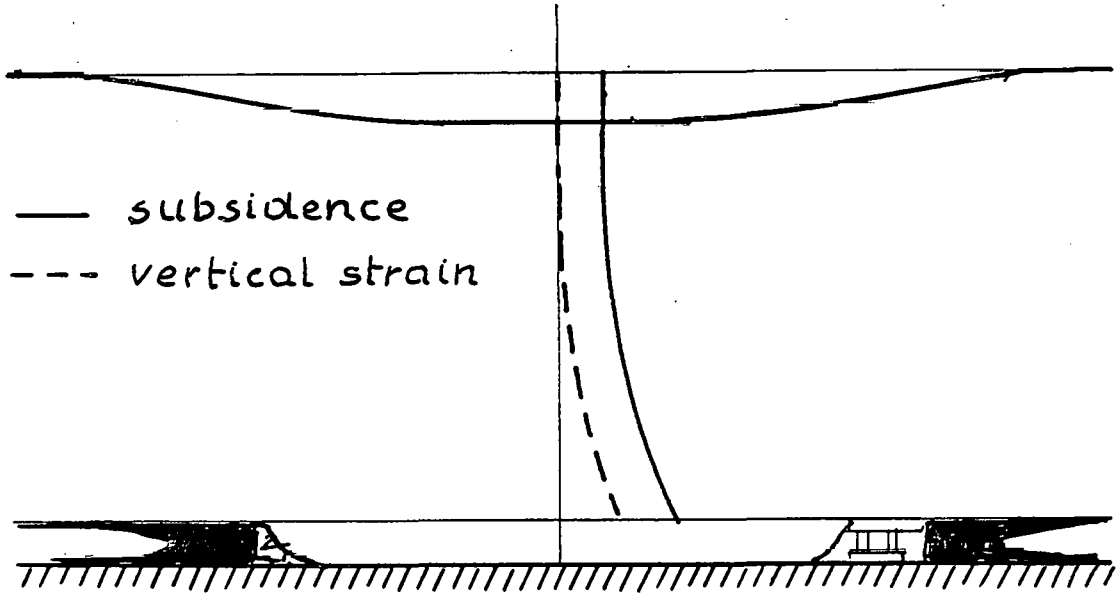


FIGURE 2.

The lateral extent of the fractured zone (clastic zone) above the longwall working usually is determined by the angle of break (Fig.3). The angle of break is defined as the angle between the veritcal and the line connecting the limit of excavation with the points on the surface of maximum tensile strain (points A). This line defines the plane of break inside the strata, although at large distances above the coal seam, near the surface, the strain may not be large enough to cause fracture. For any given rock type this line is assumed to be straight although there is some evidence to indicate that its slope increases with depth (5).

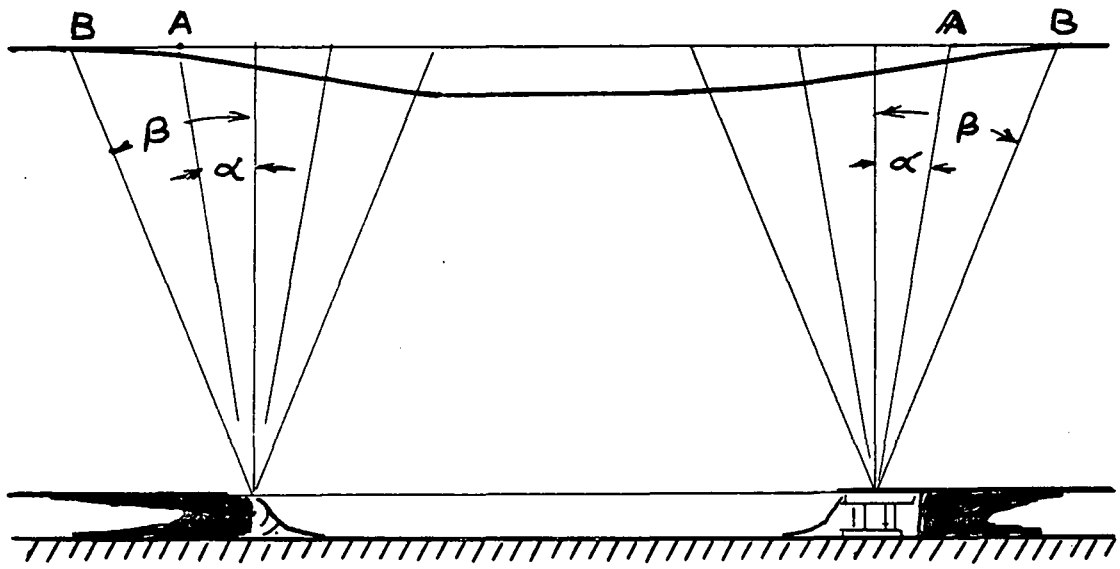


FIGURE 3.

It has been found by surveying measurements that the angle of break depends on local geology, but first of all, on the type of rocks which form the strata overlying the coal seam and on the location of major faults and fracture planes. The angle of break is strictly related to the angle of draw, which determines the lateral extent of strata displacement due to coal seam extraction (points B). In general the angle of break increases with the increase of the angle of draw. It is worth mentioning that the magnitude of the angle of draw may vary within a relative large interval from 0 to 45 degrees depending on the local conditions of particular coal fields. It has been found, for instance, that in the U.S. coal fields the angle of draw usually is smaller than in the European ones.

Summarizing the brief considerations on the strata behavior above the longwall workings, one has to point out the following main features of this behavior:
(a) Bedded rocks above the longwall workings behave like plates bend under the load of its own weight and the weight of overlying strata.
(b) Depending on supporting conditions the respective rock layers in the roof may break either right above the edge of the extracted coal seam or at some distance in front of the face above the solid coal.
(c) In the case of longwall workings there is no main roof at the near distance above the gob. It has been found experimentally that the caving process (clastic failure) propagates upwards to a horizon which is located over the coal seam at a distance equal approximately 35 to 50 times the mining height.
(d) The lateral extent of fractured zone above the longwall working is determined by the angle of break.

## LOAD ON LONGWALL SUPPORT

Many methods have already been proposed for estimating the support requirements in longwall workings. One of the best known and commonly used is the method proposed by A.H. Wilson (6,7). It has been based on the assumption that the support its are loaded only by the weight of rocks forming the immediate roof (Fig. 4).

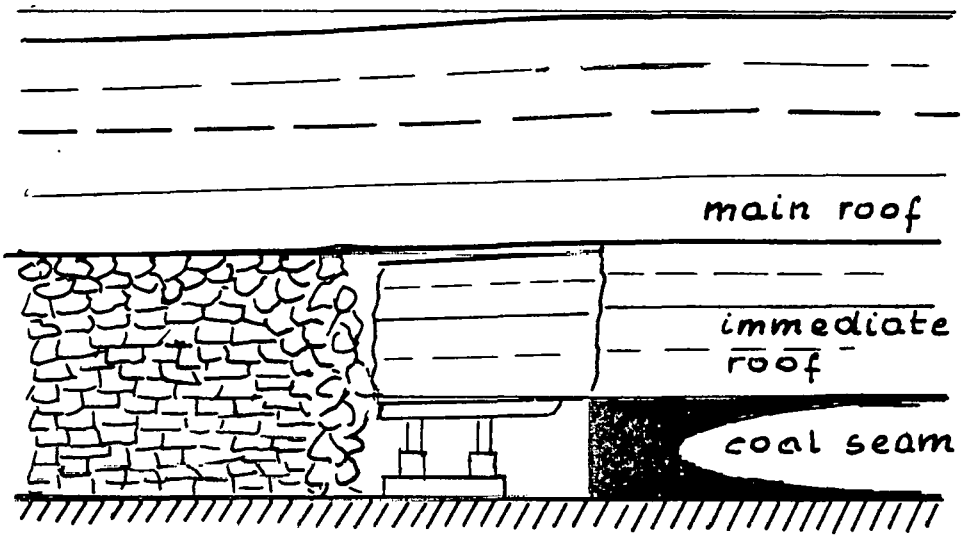


FIGURE 4.

The thickness of immediate roof is determined from Eq (2.1), thus it depends only on the assumed value of the bulking factor of broken rocks and on coal seam thickness. In that case the mean denisty of the support load may be expressed by following formula:

$$\gamma_z = \frac{\gamma m}{k-1} \tag{3.1}$$

where $\gamma_z$ - mean load density

m - coal seam thickness

k - bulking factor

$\gamma$ - weight density of rocks

Another method elaborated by U.S. Bureau of Mines is based on quite similar assumptions. Although simple and apparently successful both methods have the same unsatisfactory features. First of all, they relate the thickness of immediate roof, and thus the supporting load to the seam thickness or mining height. However, it is not self evident that in thinner seams the load on longwall should be smaller. Another criticism that can be made, is that the discussed methods do not take into consideration the stress condition in rock mass above the longwall working and neither the strength properties of the material involved.

Having in mind the objections presented above, I. Evans (8) made an effort to derive a formula for mean load density assuming the broken strata in the roof of a longwall working to be similar to granular material and in order to reach this goal he applied the arching theory developed in soil mechanics by K. Terzaghi. According to basic assumptions of this theory the local yield of the horizontal layer of rock can be produced by gradually lowering a strip-shaped section of the support. Before the strip starts to yield the vertical pressure on the horizontal support is equal to the depth of the rock layer times its unit weight. However, a lowering of the strip causes the rock located above the strip to follow. This movement is opposed by frictional resistance along the vertical boundaries (Fig. 5). As a consequence the total pressure on the yielding strip decreases by an amount equal to the component of the shearing resistance which acts on the boundaries.

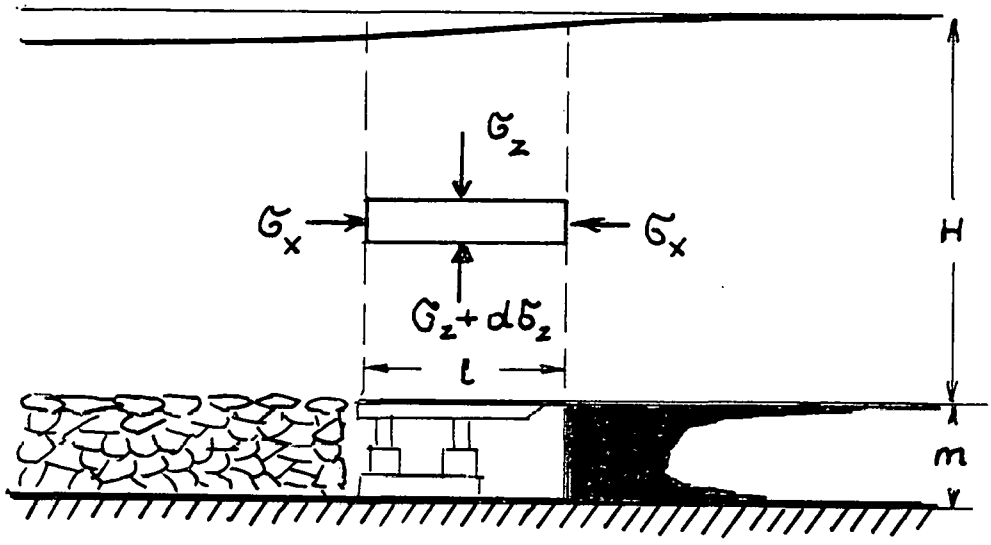


FIGURE 5.

Having applied the static equilibrium principle and Coulomb's Law of shearing resistance, Evans derived following formula for the mean load density on longwall supports

$$\sigma_z = \frac{l(\gamma - \frac{2c}{l})}{2K \tan \varphi}\left[1 - \exp\left(-\frac{2K \tan \varphi}{l} H\right)\right] \tag{3.2}$$

where

$\sigma_z$ - mean load density
l - breadth of the face
$\gamma$ - weight density of rock
$\varphi$ - angle of internal friction
c - cohesion of rock
H - depth of the seam
K - ratio of horizontal stress to vertical stress

For broken rock c may be taken to be zero, and for relatively great depth ($H \to \infty$) Eq (3.2) takes a very simple form

$$\sigma_z = \frac{l\gamma}{2K \tan \varphi} \tag{3.3}$$

Making use of Eq (3.3) for estimating the numerical value of required support capacity, Evans assumes the stress ratio K equal to unity. In the light of previous considerations on the bedded rock mass behavior above the longwall working this assumption seems to be, to great extent, unrealistic. In the roof strata above the face area prevail horizontal tensile stresses which cause the fracturing of rock layers. Under these circumstances the stress ratio K may assume values to be closer to zero than to unity. In that case Eq (3.3) would provide infinitively high values of mean density of the required support load. Thus, the determination of the real K value forms a crucial problem in Evans' method and this problem can ' ardly be solved empirically since it is very difficult to measure the stresses in ›of layers above the face area.

For these reasons an attempt has been made to generalize Evans' method of estimating the support load and to avoid mentioned difficulty. This generalization has

been based on the assumption, that the downward movement of the broken rock above the face area is opposed by frictional resistance along inclines boundaries between the moving and stationary mass of rock.

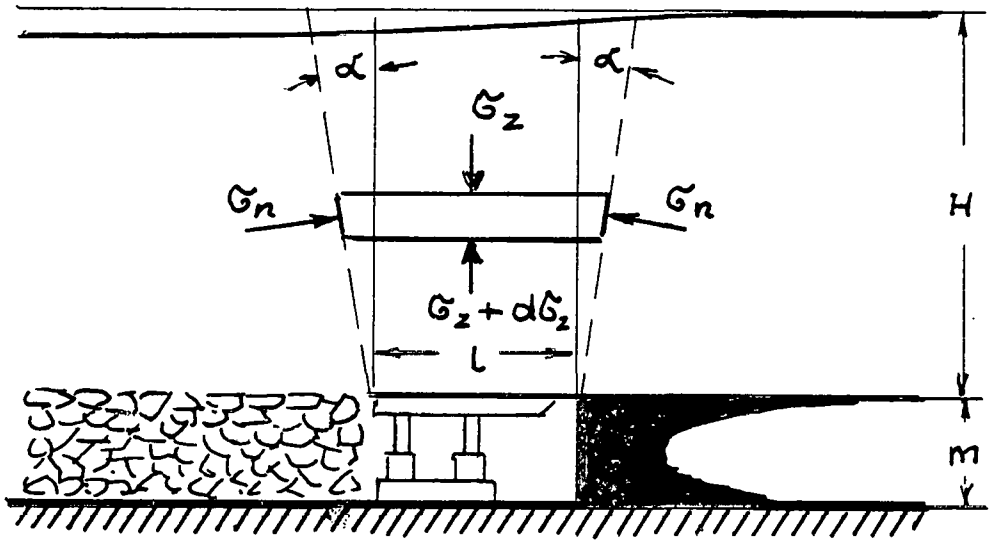


FIGURE 6.

These boundaries are formed by the planes of break and its position is determined by the angle of break (Fig 6). For these conditons the following formula determining the mean load density has been derived.

$$\sigma_z = \frac{l\,[\gamma l H + \gamma H^2 \tan\alpha - 2cH\cos\alpha]}{[l + 2H\tan\alpha]^2} \qquad (3.4)$$

Where

$\sigma_z$ - mean load density

$l$ - breadth of the face

$\gamma$ - weight density of rocks

$\alpha$ - angle of break

H - depth of the seam

c - cohesion of rocks

For broken rock c may be taken to be zero and then Eq (3.4) assumes a simpler form

$$\sigma_z = \frac{l\gamma H[l + H\tan\alpha]}{[l + 2H\tan\alpha]^2} \qquad (3.5)$$

According to Eq (3.5) the mean load density on face supports increases with the increase of the mining depth (Fig. 7) as well as with the increase of face breadth (Fig. 8) while it sharply decreases with the increase of the angle of break (Fig.9).

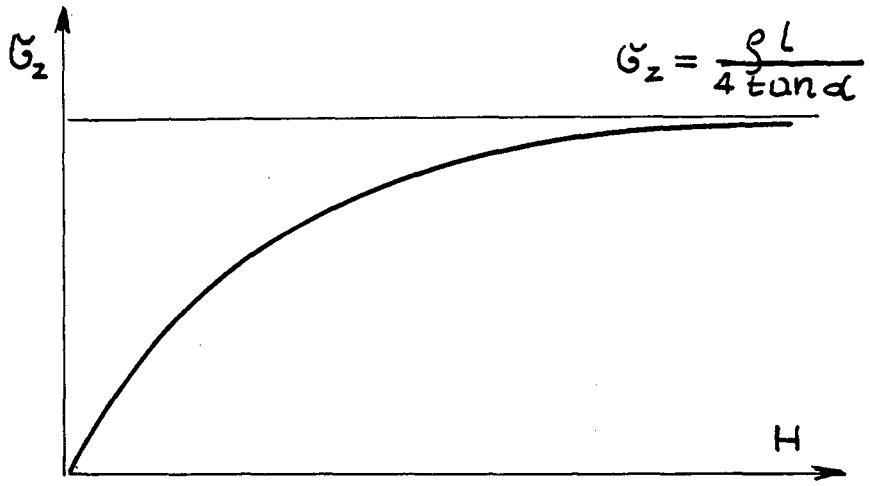


FIGURE 7.

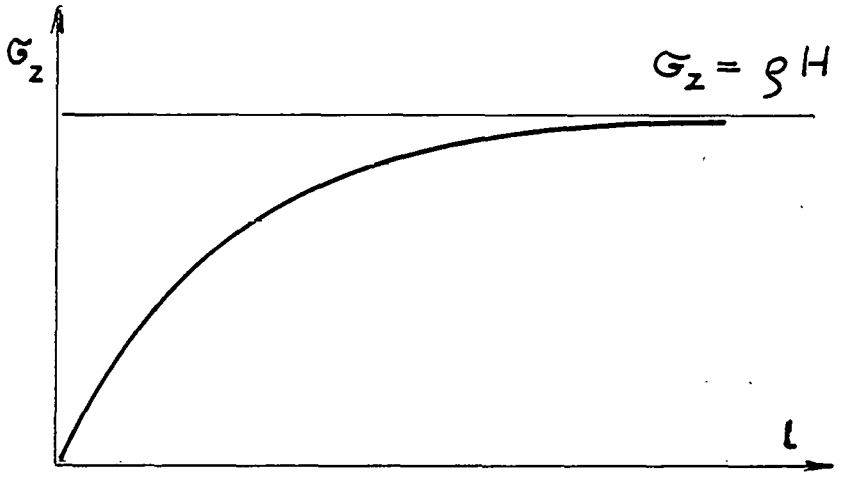


FIGURE 8.

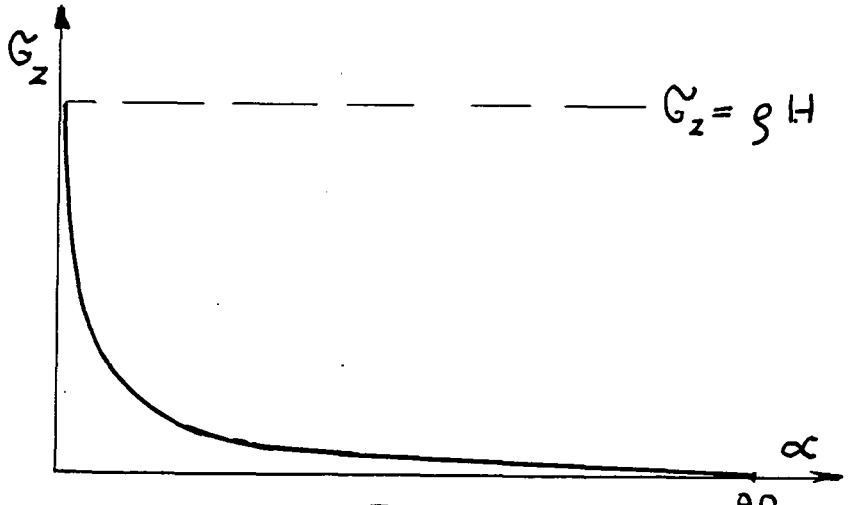


FIGURE 9.

It is worth mentioning that all quantities occuring at the right side of Eq (3.5) can easily be determined by appropriate measurements. The last formula may provide certain answer to the question posed at the begin of these considerations why in certain coal fields at lower depth a greater support capacity is needed. It is so because in certain fields the angle of break is small due to mechanical properties of roof rock layers or to specific geologic conditions of these mining fields. This conclusion can be best illustrated by numerical results of calculations which have been carried out using Eq (3.5) for two very distinctive coal fields in respect of mining depth and angle of draw, namely for Southern Illinois and Upper Silesian (Poland) coal fields. For the sake of comparison there have also been calculated the required support capacities according to Wilson's (Eq. 3.1) and Evans' (Eq.3.3) theories. For both coal fields the same values of face breadth l, mining height m, weight density of rocks $\varrho$, bulking factor k and angle of interval friction $\bar{\Phi}$ have been assumed. Thus only two factors differentiate the coal fields under considerations the mining depth H and the angle of draw $\beta$. For Southern Illinois coal field an average value of the angle of draw equal 10 deg have been assumed (9). In Upper Silesia the corresponding value has been found equal 28 deg. Making use of an approximate, empirical relationship between the angle of draw $\beta$ and angle of break $\alpha$ in the form

$$\tan\alpha = 0.4 \tan\beta \tag{3.6}$$

the corresponding values of the angle of break have been calculated. All the assumed and calculated values are presented in Table 1.

Table 1

| Coal Field | l ft. | $\varrho$ tons/cu.ft. | m ft. | H ft. | k | $\Phi$ deg | $\beta$ deg | $\alpha$ deg | $\sigma_z$ tons/sq.ft. | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Eq.3.5 | Eq.3.1 | Eq.3.3 |
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Southern Illinois USA | 13 | 0.071 | 6 | 490 | 1.5 | 40 | 10 | 4 | 3.17 | 0.85 | 0.55 |
| Upper Silesia Poland | 13 | 0.071 | 6 | 1900 | 1.5 | 40 | 28 | 12 | 1.08 | 0.85 | 0.55 |

In columns 10-12 of this table the required support capacitites (in tons per square foot of the roof to be supported) are shown which have been calculated from respective equations. It should be noticed that according to the new proposed Eq (3.5) the load capacity of the supports in Illinois coal mines has to be almost three times higher than this one in the deep Upper Silesian mines while Wilson's and Evans' formulas provide the same values for the both coal fields.

## CONCLUSION

The load of support units in the face area of longwall working depend mainly on the behavior of rocks in the vicinity of the working as well as in more remote area. This behavior is best expressed by subsidence pattern of the ground. The proper methods of evaluating the required support capacity have to take these factors into account. In this paper one of such method has been proposed in form of (Eq 3.5).

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# AUTOMATED LONGWALL GUIDANCE AND CONTROL SYSTEM

S. C. Rybak

Manager, Electronics and Control

Bendix Energy, Environment & Technology Office
Englewood, Colorado 80110

Abstract This paper presents the results of a study performed on the automation of longwall coal mining system. The automated longwall system consists of a Vertical Control System (VCS), a Yaw Alignment System and a Roll Control System which together constitute the Face Advance System (FAS), and a Master Control Station (MCS). The initial phase of the study is concerned with the modeling, analyses and simulation of candidate VCS and FAS required to satisfactorily automate the longwall system. It resulted in the specification of a highly reliable and economically advantageous longwall system configuration. The second phase of the study involved the preliminary electronic and mechanical design of the above specified system. It included the definition of major electronic blocks, electronic packaging, MCS panel layout, various sensor deployment mechanisms and integration of mechanical/electrical components on the shearer including cable routing.

## Introduction

Dwindling domestic and world oil and gas supplies with the inevitable rise in the price of these fuels have generated deep concern within government, industry, and the general public as to how the energy needs of the country can reasonably and economically be met in the future. The increased use of coal, our most abundant fossil fuel reserve, during the next quarter century will necessitate increasing the efficiency of underground coal extraction. Longwall mining techniques have the potential of greatly increasing the coal yield per acre and coal production per man per shift since it is essentially a continuous mining process. In addition, since longwall is a continuous mining process employing continuous haulage, it is extremely well suited for automation which is the subject of the present study. Automating longwall coal extraction will not only increase production but also minimize the amount of foreign material taken along with the coal thus reducing sorting time and cutter bit wear. Moreover, automating the longwall mining process will increase operator health and safety by removing the miner from the shearer and thus from the hazards encountered in the immediate cutting area.

The starting point of the reported results here is the work performed by Marshall Space Flight Center (MSFC)/NASA on the development of automated longwall shearer [1]. The objective of the present study is to specify the desired overall longwall system configuration by doing the analysis and simulation of candidate control systems required to satisfactorily automate the longwall system, and to perform preliminary design based on the recommended system configuration. The automated longwall system consists of a Vertical Control System (VCS), and a Face Advance System (FAS) which in turn consists of a Yaw Alignment System (YAS) and a Roll Control System (RCS).

The function of the VCS is to control the two cutting drums of the double-ended ranging arm shearer so that they remain within the coal seam. The VCS should be capable of taking all the coal on the roof and floor or, if desired, leave a given amount (i.e., bias) of head and/or floor coal while taking a minimum amount of rock. In addition, the distance between two successive cuts would have to remain within given limits in order that the roof supports advance satisfactorily.

In longwall mining after coal is cut along the face the conveyor is pushed forward, in preparation for the next cut, by the roof support system. A critical operational requirement is to maintain the face "straight", i.e., the coal panel normal to the head and tail gate entries. The function of the yaw alignment system is to automatically advance the conveyor and roof supports in such a manner that face "straightness" remains within acceptable limits. The function of the roll control system is to maintain the attitude of the longwall shearer about its longitudinal (roll) axis at a specific value regardless of the twists that may be in the conveyor track on which the shearer is riding. This mode of control is necessary if the shearer is to be maintained perpendicular to the longwall panel being mined and hence within the coal seam as repeated passes are made.

In the analytical portion of the study, a procedure is needed by which the performance of various control system configurations could be compared in order to determine which one is "best" for a particular application. In order to accomplish this end, "baseline" system configurations for all of the control subsystems required for longwall automation were defined. The baseline definitions used the work performed by MSFC as a starting point and were the "simplest" subsystem configurations that would satisfactorily perform a particular control function. It was against the performance achieved by the "baseline" systems that various other control system configurations /additions/modifications were compared.

Proposed modifications/additions to baseline sytem definition were evaluated by considering the improved performance realized versus the additional complexity and relative cost introduced by incorporation of the modification, and only those modifications/additions to the baseline systems that were cost effective were incorporated. Following this procedure, a highly reliable and economically advantageous longwall system design emerged.

## Vertical Control System

The function of the vertical control system (VCS) is to control the position of the shearing drums so that their cut follows the coal/shale interface (CSI) on the top and the bottom of the mine and leaves a prescribed amount of top and bottom coal. The VCS must also maintain the present cut within a prescribed amount of the last cut made. The shearing drums are positioned by raising and lowering the ranging arms on which they are mounted, using hydraulic actuators. The drums are controlled independently and are coupled only through the hydraulic system, where the pump flow is shared between the two ranging arm actuators.

Commands to the hydraulic actuators are generated from a complement of coal interface and drum position sensors. The coal interface detectors or CIDs are of two basic types: proportional sensors which measure coal depth, and discrete sensors which indicate the presence of coal or rock. The proportional sensors are the nucleonic sensor which measures depth by counting backscattered $\gamma$ radiation from a cesium source, and a natural radiation sensor which measures depth by counting natural background radiation transmitted through the coal. Both these sensors can be mounted only behind the shearing drum. The discrete sensors

are a sensitized pick and a combination impact penetrometer-reflectometer. The sensitized pick measures forces exerted on instrumented cutting picks to determine rock presence, while the penetrometer-reflectometer measures surface hardness and reflectance to sense rock. The pick is mounted on the shearing drum while the penetrometer-reflectometer is mounted behind the drum. The position sensors include a last cut follower (LCF) which indicates the shearing drum's position relative to the last cut made, and an actuator position sensor which allows the position of the ranging arms relative to the shearer body to be determined.

The sensors are combined in a hierarchical manner to develop actuator commands. The LCF has the highest priority and will never allow the present cut to differ by more than a prescribed amount from the last cut. The coal presence CID has the next priority and will move the shearing drum away from the interface if rock is detected. The proportional CID is the lowest priority and will provide the control for the cutting drum when it is within the coal seams and the commands are within the prescribed limits relative to the last cut.

Since the shearer operates in either direction, both drums must be capable of cutting either the top or the bottom. The leading drum cuts the top while the trailing drum cuts the bottom, and the individual drum control laws are therefore changed appropriately, depending on whether the drum is leading or trailing. The control laws also change, depending on the desired coal depth to be maintained. If it is desired to take all the coal, only the coal presence sensors are used for control.

The following are identified as the constraints within which the VCS analysis/design must be carried out:

(i) Two successive cuts cannot deviate from each other by more than $\pm 2$ in.

(ii) The amount of rock taken is to be minimum.

(iii) The CID that measures the depth of coal on the roof, is to be mounted about 4 ft. behind the cutting drum.

(iv) VCS should be capable of leaving from zero to 6 in. of head coal reliably.

(v) The nominal shearer face traverse velocity is 30 ft/min.

In addition to the above, it is determined that a proportional CID cannot be mounted to measure floor coal thickness due to the fact that loose coal, rock and water collect on the floor.

In order to analyze and design the VCS, a typical longwall shearer had to be chosen for computer simulation. The longwall shearer chosen was the Joy LW 300 and all control system studies involving the shearer were performed with this model. In addition, a reasonable representation of the coal/shale interface that would be encountered during in-mine operations had to be simulated. It became apparent that using artificial "worst case" profiles would not be very satisfactory from two viewpoints:

a. It is very difficult if not impossible to define a "worst case" coal shale interface without having access to the actual interface profiles in most or possibly all of the operational longwall faces and those being projected for longwall mining.

b. Even if a "worst case" profile could be defined, it would place extreme requirements on the longwall system resulting in an unrealistic and costly system design.

It was, therefore, decided to model typical coal/shale interface profiles that one would commonly encounter during longwall mining operations. To accomplish this end, use was made of a survey [2] in which the coal/shale interface profiles for six mines representing a variety of geologic conditions and seam heights were given. Statistical analyses were performed on the twelve profiles (i.e., top and bottom) given and it was determined that all of the mine profiles were statistically similar. This result certainly gives credence to the hypothesis that coal/shale interface profiles are more similar than different when going from mine to mine. Therefore, the York Canyon profiles were chosen from the given data set as representing typical (i.e., mean) coal/shale interface profiles that would be encountered during in-mine operations and most performance runs were made with these profiles. Performance evaluations made with other mine profiles had little effect on overall performance.

## VCS Performance Criteria

To evaluate VCS performance and compare various configurations with respect to each other, a measure of performance had to be defined. There are numerous performance measures that one can define for the VCS, none of which will be an adequate measure of performance for all operational conditions. The most commonly used criterion to characterize the performance of a control system is the RMS of the error between the desired and actual control achieved which, in our case, is the RMS of the difference between the desired and actual cuts achieved, or the cut error. Although this is generally an excellent measure of VCS performance, it does not totally specify all aspects of VCS performance. Other performance characteristics that are of interest are the amount of time spent cutting rock, the volume of rock taken, the volume of coal erroneously left, and the volume of coal erroneously taken. Therefore, in order to augment the RMS cut error, a number of other performance parameters were also determined which are: a) percent of time spent in rock; b) volume of rock taken; c) volume of excess coal taken; d) volume of coal erroneously left; and e) the average of the absolute value of the cut error. These performance parameters, in conjunction with the RMS cut error, adequately specify VCS performance over its designated range of operation.

## VCS Simulation Model

A block diagram of the VCS computer simulation is shown in Figure 1. The major features of the simulation are listed below:

a. Detailed modeling of the mine geometry including actual interface profiles and techniques for generating statistically similar coal/shale interface profiles from a given initial profile which is used to give realistic results when multiple passes are made.

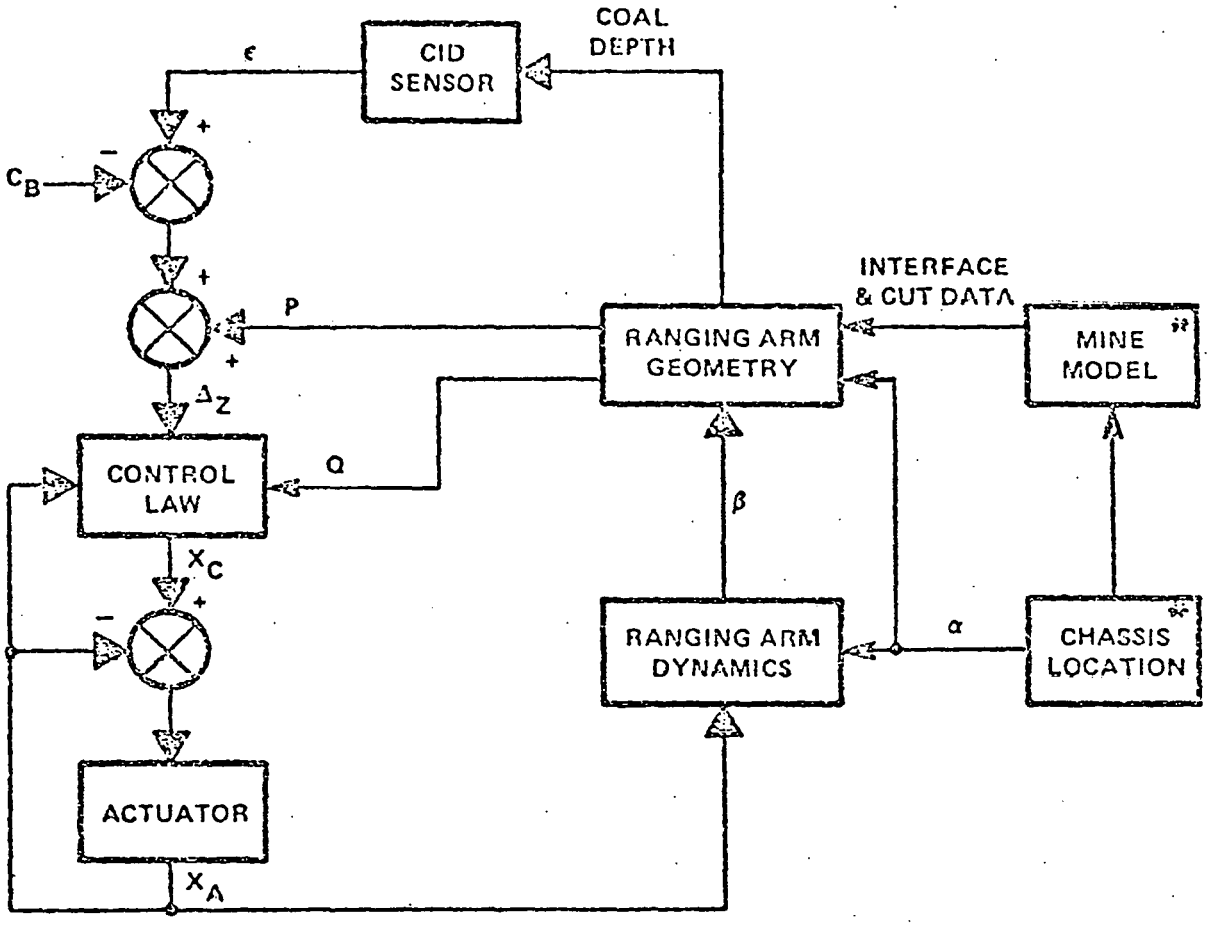



Figure 1. VCS Simulation Model Block Diagram

b. Detailed geometrical model of shearer including its location along, and orientation with respect to, the face.

c. Technique for determining the actual track that would result when laying five foot conveyor sections along a roughly cut bottom.

d. Detailed nonlinear representation of shearer actuator dynamics including the hydraulic coupling between the two ranging arms.

e. Detailed representation of the nucleonic and natural radiation coal interface detectors including the nonlinear calibration curves, Poisson distributed sensor noise, and air gaps.

f. Complete representation of coal/shale presence sensors including probabilistic representation of the sensors.

g. Complete representation of last and present cut followers.

h. Modular hierarchical control law algorithm that is able to accommodate a variety of sensor complements and control law implementations.

i. The effect of drum filtering on the cut obtained.

VCS Model Description: In Figure 1, the asterisks indicate those portions of the figure which are part of the simulation, but are not part of the actual shearer or control system.

The chassis location module represents those calculations necessary to determine the position and orientation of the shearer chassis relative to the mine. This includes keeping track of the chassis position along the face and the chassis pitch angle $\alpha$.

The ranging arm geometry calculations use the pitch angle $\alpha$ and the arm angular position relative to the chassis $\beta$ to calculate the shearing drum position in the mine. The quantities computed are the coal depth presently being cut, the drum height relative to the CID, P, and the drum height relative to this last cut, Q. The CID module processes the coal depth measurements, introducing the appropriate errors to simulate the various CID sensors. The coal depth output measurement is designated $\varepsilon$.

The baseline control compares the measured coal depth with the desired coal depth bias $C_B$. This difference is summed with the drum position relative to the CID sensor, resulting in a command $\Delta z$, the desired change in height of the cutting drum. This information, along with the last cut data and present actuator ram displacement $X_A$, is used in the control law computations. The result is $X_C$, the desired displacement of the hydraulic ram.

Actuator dynamics and ranging arm dynamics are modeled in some detail (Figure 2). The actuator model includes the control valve with input thresholds, check valves, and pump flow rates. Ranging arm dynamics are coupled with the actuator and develop the ranging arm angle $\beta$. The actuator is instrumented with a position sensor and used in a closed loop so it can be position commanded.

To develop a mine model, several one pass mine surveys were digitized and analyzed to provide an information base with which to generate subsequent simulated mine data. The statistical data obtained from these mines included seam slopes, coal shale interface (CSI) variances and seam heights. It can be seen from these data that, statistically, there is a great deal of similarity between the mines considered. The CSI variances and correlation data are quite similar. In addition, in general, there was a significant degree of correlation between the top and bottom which will be of some importance when drum slaving is considered. A procedure was also developed to generate random data with the same statistical and spatial correlations as the original surveyed data. Figure 3 shows a 3-D view of a 12.5 by 50 ft. section of simulated Robinson Run bottom.

Detailed description of each portion of the simulation model of Figure 1 can be found in [3].

Baseline VCS

The "baseline" system configuration for the VCS consisted of the following:

a. An active nucleonic coal interface detector (CID) for measuring coal thickness on the roof.

b. Sensitized picks incorporated into each of the two cutting drums in order to detect the presence of coal or rock.

c. A last cut follower (LCF) mechanism in order to measure the present cut relative to the last one made.

d. The bottom cutting drum is slaved to the top one if a given amount of floor coal is to be left. The reason for not using a CID for proportional floor cutting drum control is that it is not feasible to mount such a detector on the floor given all the debris and water found there.

e. Use presently obtained CID data in a measure and cut algorithm as opposed to optimal filtering/prediction algorithms which can be used to compensate for CID noise and that the measurement point is some 4 feet behind the actual drum cutting point.

f. Use of the nonlinear hydraulic actuation system commonly found on longwall shearers.

Baseline System Parameter Determination: For the baseline VCS defined, system parameters (i.e., gains, sensor locations, sampling and averaging times, etc.) had to be determined in order to obtain an optimum performance. One of the first parameters to be determined was the loop gain/compensation to be used for the position loop closed around the actuators which control cutting drum height. Various loop gains were investigated and it was determined that a gain of 170 yielded best overall performance even though the actuator did have limit cycle at this gain value. The only problem that can occur if the actuator is operated in the limit cycle region is that it may heat excessively. Should that be the case, a heat exchanger could easily be designed which will maintain temperatures within acceptable limits.

Another parameter to be determined was the mounting location of CID sensors. It was seen that system performance improves as the CID is moved closer to the cutting drum (and hence the cutting point) since this minimizes the distance between the point measured and the point being controlled. Therefore, the CID's were located 4 feet behind the cutting drum which was as close to the cutting drum as the CID could be mounted without interference.

Another critical parameter to be determined was the CID output interval or averaging time, T. It was found that in the absence of CID measurement noise, decreasing T improved the system performance. However, the effect of sensor noise was

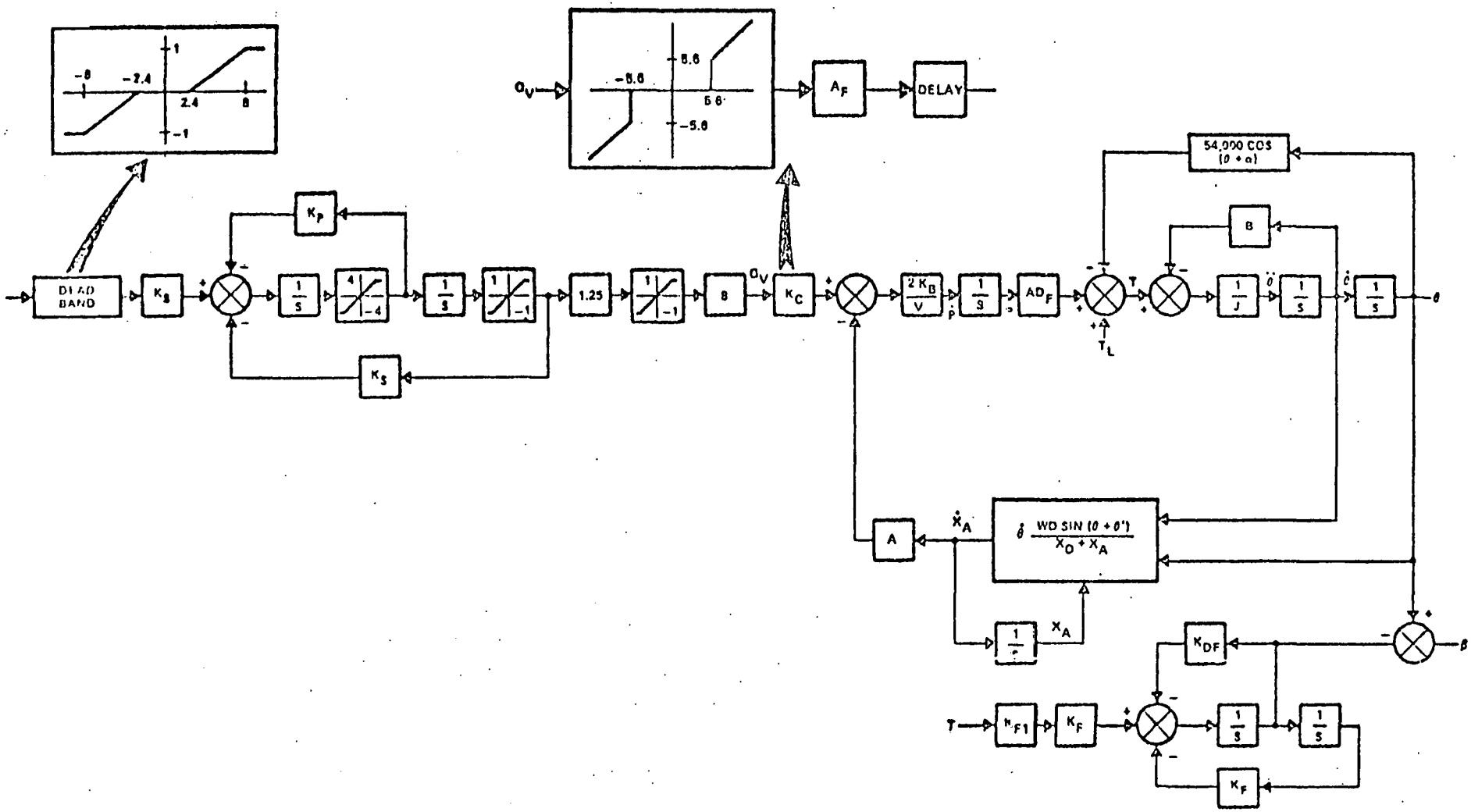



Figure 2.1 Actuator and Ranging Arm Simulation Model

ROBINSON RUN

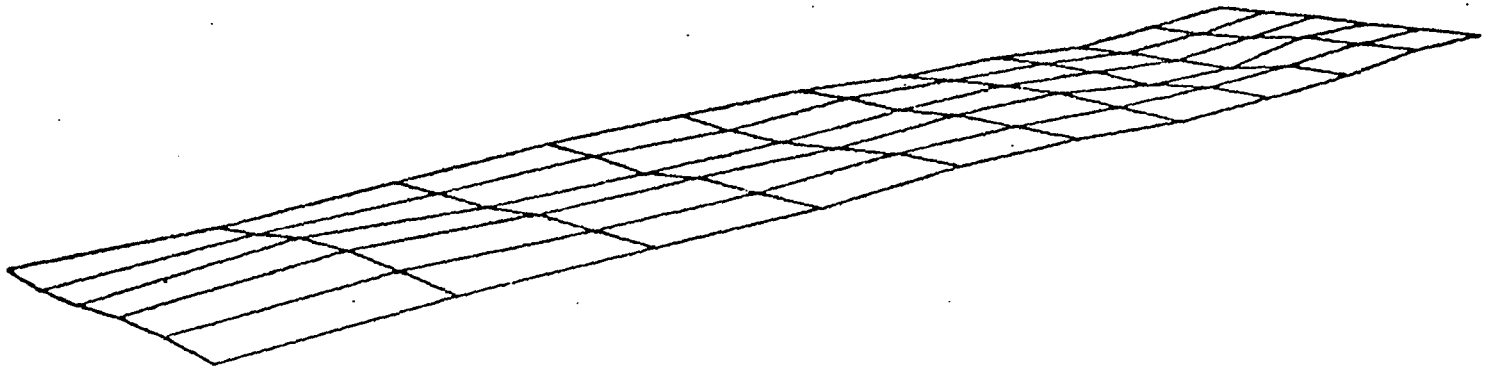


Figure 3. 3D View of the Generated Bottom for the Robinson Run Mine

more pronounced as the output interval was shortened. Thus, there exists an optimal value for the CID output interval and was found to be 0.25 second for the active nucleonic CID, and 1 and 2 seconds, respectively, for 5 and 3 inch detection crystals in the case of natural radiation CID.

Baseline VCS Performance Characteristics: The following observations are made as a result of the computer simulation of baseline VCS.

Effect of Nonlinear CID Calibration Curve - The CID calibration curves, which give the number of counts one expects for a given coal depth, are nonlinear and saturate as the thickness of coal increases. Therefore, for the same CID measurement noise, the error in the sensor measurement would increase as the desired thickness of coal to be left approaches the saturation characteristics of the sensor thus giving degraded system performance. Such an effect has been found and the simulation results indicate that reasonable performance is obtained when the desired coal to be left is 5 inches or less.

Effect of Adding Sensitized Pick and LCF to CID - The effect of adding a sensitized pick and using it in conjunction with the CID in a hierarchical manner is to improve the cutting performance. The technique that is used is to command the cutting drums on the basis of CID information and, when the pick indicates that rock is being mined, to override the CID command and depress the cutting head a given amount. The improvement realized by the addition of the sensitized pick is most pronounced when attempting to leave a small coal thickness (i.e., between zero and 2 inches), since it is during this mode of operation that using CID information only has a high probability of commanding the drum to cut the roof due to CID measurement errors. The addition of a last cut follower to the CID and sensitized pick and employing this sensor configuration in a hierarchical manner where the last cut follower is given highest priority with the CID having lowest priority further improves cutting performance, the reason being that LCF filters even further errors in the CID measurements since the present cut cannot deviate more than a prescribed amount (i.e., $\pm$2 inches) from the previous cut.

VCS Performance with Sensitized Pick Only -When attempting to take all of the coal the VCS will be controlled by the sensitized pick only, or possibly acting in conjunction with a last cut follower. In this mode of operation the system cutting performance degrades as the error in the pick information increases, the reason being that when the VCS is under pick control, errors in the pick information cause an immediate error in the cutting drum command. It is, therefore, concluded that the sensitized pick should be between 80 to 90 percent accurate to enable taking all of the coal, while maintaining the rock taken to a minimum.

Effects of Actuator Dynamics on VCS Performance - Inserting the nonlinear actuator dynamics into the VCS control system had little effect on system performance when a given amount of coal is to be left, and hence a CID is employed. However, if all of the coal is to be taken, and the VCS is under pick control, then the insertion of actuator dynamics has a marked effect on the performance achieved with the RMS cut error going from 0.68 inches without actuators to 2.06 inches when the effect of actuator dynamics is considered. Adding the actuator dynamics has the effect of introducing lag into the system and hence performance degrades. When the VCS is under CID control the errors introduced by CID noise and its being mounted behind the cutting drum overshadow the effect of the actuator dynamics thus minimizing its effect on overall system performance.

Effects of Varying VCS Traverse Speed Across the Face - The nominal traverse speed at which the baseline system was designed and its performance evaluated was 30 ft/min. Increasing this speed to 50 ft/min or decreasing it to 20 ft/min had no effect on system performance. The reason for this result is that a 0.25 second output interval is being used for the CID which is quite fast, and within the speed range of 20-50 ft/min the shearer does not travel an appreciable distance between CID information updates.

## Modified VCS

Modifications/additions to the defined VCS "baseline" considered are listed below.

1. Use of optimal filtering/predictive control algorithms for cutting drum control.
2. Use of an impact penetrometer/reflectometer for a coal/shale presence sensor.
3. Use of natural radiation CID.
4. A number of bottom drum slaving techniques.

VCS Performance with Optimum Filtering/Predictive Algorithms: Since the CID is mounted behind the cutting drum it is not measuring the coal/shale interface at the cutting point. Using the measured depth of the coal/shale interface at the CID location and saying that this is the depth at the cutting drum location obviously is in error since the coal/shale interface changes as one proceeds down the face. Therefore, one would suspect that using some form of filtering/predictive algorithm system performance could be improved, since a more accurate estimate of the coal/shale interface both at the point of measurement and at the cutting drum could be determined. At first glance one would be tempted to make use of all or many past CID measurements in order to obtain a "best estimate" of the coal/shale interface at the present measurement location and cutting point. Implied in such a scheme would be a method by which all of the CID measurements could be referenced with respect to each other through some common reference frame. However, no such frame exists in the longwall system and there doesn't appear to be any reasonable way by which such a frame could be obtained. Therefore, measurement changes due to floor undulations could not be differentiated from changes due to actual variations in the coal/shale interface height above the skid plane. Given this situation the filtering/predictive algorithms that will be considered for the longwall are those which employ CID measurements in the immediate vicinity of the present measurement and cutting points. These types of algorithms would require a priori knowledge of the coal/shale interface statistics in terms of correlation function, mean and variance as well as a description of the sensor noise in terms of its mean and variance. The reason for this a prior knowledge requirement is that the statistics of the coal/shale interface could not be derived from past CID measurements due to the lack of a common reference frame.

Two types of optimal filtering/predictive algorithms employing CID measurements in the immediate vicinity of the present measurement and cutting points were formulated. One algorithm (Scheme 2) uses the present CID measurements in conjunction with CID data in the immediate vicinity of the present measurement to estimate and predict the coal/shale interface at the measurement point and cutting drum respectively. The other technique (Scheme 1) uses the present CID measurement in conjunction with estimates of the coal/shale interface in the immediate vicinity of the present measurement point for estimation and prediction. These two types of algorithms were formulated in a one-dimensional sense using present cut CID data/estimates, and in a two-dimensional sense using present and past cut CID data/estimates yielding four different optimal filtering algorithms for possible use in the VCS.

Analyses performed on these algorithms indicated that both should give appreciable performance improvement if the floor didn't undulate, with Scheme 1 being better than Scheme 2. In addition, it was shown that the algorithm employing CID measurements only would be less sensitive to the unmodeled floor undulations than the scheme employing CID data in conjunction with coal/shale interface estimates. These results were verified by simulation for the two one-dimensional algorithms, and the data indicated that cutting performance degraded due to floor undulations to the point where it wasn't any better, and in most cases, worse than that obtained with the baseline measure and cut algorithm. The degradation in performance due to floor undulations would be even worse for the two-dimensional version of these algorithms and hence were not simulated. Therefore, the conclusion of this investigation was that there is no advantage to be gained by using optimal filtering and predictive algorithms for VCS control. Thus, the simple measure and cut control algorithm defined in the baseline system should be used for cutting drum control.

Use of Impact Penetrometer/Reflectometer for VCS Control: Investigations were conducted into the use of an impact penetrometer/reflectometer as the coal/shale presence sensor, i.e., to replace the sensitized pick. The prime difference between the two types of coal presence sensors from an overall systems viewpoint is the manner in which they are integrated into the VCS. The sensitized pick, which is mounted on the cutting drum measures whether the drum is presently cutting coal or rock, while the impact penetrometer/reflectometer is mounted at the approximate CID location, and hence can only measure whether the drum has cut coal or rock. The sensitized pick yielded excellent cutting performance (0.68 inch RMS) and VCS seam following capability when used by itself since it does measure what the drum is cutting virtually instantaneously. However, the impact penetrometer/reflectometer being mounted some 2.5 feet behind the cutting drum cannot measure whether the drum is cutting rock or coal until the shearer has traversed 2.5 feet to the present drum location, which at 30 ft/min shearer traverse speed takes 5 seconds. During this total time, if the impact penetrometer/reflectometer is measuring rock, it is continuously depressing the cutting drum or if it is measuring coal it is continuously commanding the cutting drum to rise, thus giving rise to severe system limit cycles and an increased RMS cut error. When the impact penetrometer/reflectometer is augmented by a last cut follower, system limit cycles are limited by the deviation allowed between the present and previous cuts. If, as in our case, this value is set at $\pm 2$ inches the resultant RMS cut error is essentially the RMS of a limit cycle ranging between $\pm 2$ inches. Therefore, the conclusion of these studies is that the impact penetrometer/reflectometer is not a very satisfactory coal presence sensor for the longwall application being considered, particularly when all the coal is to be taken, and hence is not recommended.

Use of Natural Radiation CID for VCS Control: The use of natural radiation CID instead of an active nucleonic one was investigated. The prime difference between these two sensors that affects system performance is the number of counts per unit time for a given coal thickness. The active nucleonic CID has many more counts per second (approximately a factor of 20) for a given coal thickness than a reasonably sized (i.e., 3 to 5 inch detection crystal) natural radiation CID. Therefore, the noise and hence the coal depth measurement uncertainty is worse with the natural radiation than with the active nucleonic CID, and degraded system performance when using the natural radiation CID should be expected. This indeed has been verified by simulation with increased RMS cutting error for the natural radiation sensor. However, there are drawbacks to the active nucleonic sensor ranging from sociological, (i.e., active radiation source) to technical (i.e., sensitivity to air gaps) to potentially severe integration problems (i.e., sophisticated suspension that must keep the sensor against roof with a minimum of air gaps, deployment and stowage mechanisms, etc.) that the natural radiation CID does not have. Thus with natural radiation CID, although as pointed out above, the system cutting performance degrades to a degree, the performance achieved is still quite good when compared to what is presently being obtained manually. Moreover, all of the benefits with respect to the health and safety that one gains by longwall automation would remain intact. Hence, the natural radiation CID is recommended.

Slaving the Bottom Drum: Since it is not feasible to mount a CID on the floor for controlling the drum cutting the bottom due to debris and water that will be there, an alternate technique of control for the bottom drum had to be devised for the condition where a given amount of coal is to be left on the bottom. (When all the coal is to be taken the bottom cutting drum can be controlled by a sensitized pick in the same manner that roof cutting drum control is achieved.) The technique used was to slave the bottom cutting drum to the top one by maintaining the instantaneous height of the cut constant. A number of ways of accomplishing drum slaving was investigated. The first of these was to maintain the height between the two cutting drums constant by using the readouts of the linear actuators controlling both drums. This implementation was quite sensitive to floor undulations and did not yield satisfactory system performance. Another implementation that was investigated was to use CID measurements to determine the height of the roof and command the bottom drum to maintain the instantaneous cut height constant. This implementation, although better than maintaining the instantaneous height between the cut-

ting drums constant, still yielded performance that wasn't any better than locking the drum for a flat floor, and degraded when the actual floor undulations were introduced. Surveyed data indicate that four of the six mines exhibited a high degree of correlation between the top and bottom coal/shale interface profiles. This phenomenon suggested that if the CID measurements were delayed until the bottom cutting drum was directly below the CID measurement point and then used to control the bottom drum, better slaving performance should be realized. This was indeed the case (for a flat floor) with improvement being quite apparent when the top and bottom interfaces were highly correlated such as in the Robinson Run Mine. It was also true to a lesser degree in a mine that did not have a high degree of correlation between top and bottom such as in York Canyon. However, when the floor undulations were introduced, significant performance degradation resulted and better system performance would be achieved by locking the bottom cutting drum.

One basic problem (besides performance) that the slaving implementations discussed above have in common is that they do not tend to maintain the difference between two successive bottom cuts within a prescribed value. To overcome this problem, the bottom drum was slaved to the last cut follower or to a present top cut following mechanism. Slaving the bottom drum to the last cut follower mechanism was tried and it was found that the system performance was sensitive to floor undulations. An attempt was made to salve the bottom drum to a last cut following mechanism located in the middle of the shearer in order to minimize the effect of floor undulations. This implementation did indeed improve slaving performance to acceptable levels; however, roof cutting performance degraded appreciably. Slaving the bottom drum to a separate present top cut following mechanism was tried and the performance obtained was the "best" of all of the implementations evaluated. Having a separate present top cut following mechanism above the floor cutting drum essentially eliminates the problems due to floor undulations and is presently the recommended technique for bottom drum slaving.

Recommended VCS System

As a result of the studies performed, the recommended VCS implementation is outlined below as a function of the mining capabilities desired.

a. Leaving a prescribed amount of coal on top and bottom.
1) Top Drum Control:
Natural Radiation CID
Two Sensitized Picks mounted 180 degrees apart on Cutting Drum
Last Cut Follower
2) Bottom Drum Control:
Present Top Cut Follower
Two Sensitized Picks Mounted in the Cutting Drum 180 Degrees Apart

b. Leaving a prescribed amount of coal on top and taking all coal on bottom.
1) Top Drum Control:
Natural Radiation CID
Two Sensitized Picks
Last Cut Follower
2) Bottom Drum Control:
Two Sensitized Picks

c. Taking all of the coal on top and leaving a prescribed amount of coal on bottom
1) Top Drum Control:
Two Sensitized Picks
Last Cut Follower
2) Bottom Drum Control:
Two Sensitized Picks
Present Top Cut Follower Mechanism

d. Taking all of the coal on top and bottom
1) Top Drum Control:
Two Sensitized Picks
Last Cut Follower
2) Bottom Drum Control:
Two Sensitized Picks

Performance of Recommended System: The average performance of the recommended VCS configuration is shown in Table 1. Examination of this table indicates that the recommended VCS configuration will outperform a human operator by at least a factor of two at a shearer speed as high as 50 ft/min. while under manual control the shearer can only move approximately 10 to 12 ft/min. These results indicate the increased productivity and hence economic potential of automating the longwall, in addition to the health and safety gains.

Yaw Alignment System

The function of the yaw alignment system is to automatically advance the conveyor and roof supports in such a manner that the face "straightness" remains within acceptable limits. The "baseline" system defined to accomplish this control function consists of an angle cart which is integrated with the shearer and measures the angles between respective conveyor sections as the shearer proceeds along the face. Once a set of measurements are taken the present conveyor profile is computed. A set of conveyor advance commands are then computed and sent to each roof support, such that if perfectly implemented, would result in a totally straight conveyor. This procedure is followed (i.e., angle cart measurements taken, conveyor profile computed, and appropriate conveyor advance commands determined) on every advance.

Various algorithms for computing the conveyor profile from the angle cart measurements were investigated. These algorithms were a basic angle cart measurement scheme and an optimal weighting scheme. In addition the effect of not being able to measure the angle between a number of conveyor sections on either end due to the inability of moving the shearer far enough into the headgate and tailgate was also investigated.

An alternate system implementation to the baseline defined was also investigated. This implementation consisted of a directional gyro mounted on the shearer which measures the orientation of each conveyor section with respect to a defined reference. These measurements are then used to compute the present conveyor profile and a set of conveyor advance commands are subsequently generated that if implemented perfectly would straighten the conveyor in much the same manner as in the angle cart implementation.

Two computer simulations were developed in order to evaluate the performance of the yaw alignment system. The first was a Monte-Carlo

Table 1. VCS Performance Summary

| | AVERAGE RMS ROOF CUT ERROR (IN) | AVERAGE RMS FLOOR CUT ERROR (IN) | AVERAGE RMS CUT ERROR (IN) | AVERAGE PERCENT TIME IN ROCK ROOF (%) | AVERAGE PERCENT TIME IN ROCK FLOOR (%) | AVERAGE ROCK TAKEN ROOF (%) | AVERAGE ROCK TAKEN FLOOR (%) |
|---|---|---|---|---|---|---|---|
| AUTOMATED VERTICAL CONTROL NUCLEONIC CID | 2.32 | 3.19 | 2.76 | 8.73 | 7.03 | 0 | 0.11 |
| AUTOMATED VERTICAL CONTROL NATURAL RADIATION CID 5 IN. CRYSTAL | 2.74 | 3.54 | 3.14 | 11.28 | 14.43 | 0 | 0.18 |
| * MANUALLY OPERATED | N/A | N/A | ** 5.50 | N/A | N/A | NOT KNOWN | NOT KNOWN |

* AVERAGE OF ALL MEASURED PERFORMANCE
12 FACES IN 10 DIFFERENT MINES (See 2 ).

**3.1 IN. MEAN ERROR

4 IN. BIAS ROBINSON RUN MINE

SHEARER VELOCITY 30 FT/MIN
RESULTS ESSENTIALLY UNCHANGED AT 50 FT/MIN

ACTIVE NUCLEONIC CID

AVERAGE TIME IN ROCK - 7.88%
AVERAGE ROCK TAKEN - 0.11%

PASSIVE RADIATION CID

AVERAGE TIME IN ROCK - 12.86%
AVERAGE ROCK TAKEN - 0.18%

simulation which evaluated the accuracy with which the conveyor profile could be computed in the presence of measurement error. This was accomplished by performing 500 measurement runs using particular values of measurement errors and computing the conveyor profile for each run. The RMS error between the computed and actual conveyor profiles was then computed and the average RMS error was determined for the 500 runs made. The average of the RMS error for the 500 runs made, called the "mode of RMS", was used as the performance measure of the measurement scheme being evaluated in the presence of measurement error. A block diagram of this simulation is shown in Figure 4.

The other simulation developed was the Yaw Advance Simulation. This simulation actually made repeated advances of the conveyor in the presence of system measurement, conveyor placement and roof support pull-up errors, and yaw advancement stability evaluated. In order to evaluate whether yaw advancement is proceeding stably, twenty advancements of the conveyor were made in the presence of a particular set of system errors. System instability was defined when a particular roof support was commanded to move the conveyor backwards, which is impossible, within the twenty advances made. If this condition resulted for a particular set of system errors, the system error vector was perturbated slightly and two more sets of twenty advances made. The average value of the error vector for the set of three twenty advances were then used to define the stability boundary. The yaw advancement simulation block diagram is shown in Figure 5.

## Angle Cart Performance Characteristics

The basic angle cart measurement system uses two sets of measurements made by the angle cart. One set of measurements is taken when the angle cart is totally in a single conveyor section. These measurements are used to estimate the bias or constant error in the resolvers, and are used in correcting the second set of measurements which are the angles between contiguous conveyor sections. The angle that the first conveyor section makes with the desired reference (i.e., line perpendicular to headgate and tailgates) is computed via the relative angle measurements between the conveyor sections in conjunction with the knowledge where the ends of the conveyor are with respect to each other. This is obtained by measuring the distance on both ends of the conveyor with respect to surveyed points within the headgate and tailgate.

The results with the basic measurement scheme indicate that the mode of RMS increases linearly with increased measurement errors. From a stability viewpoint the maximum allowable one sigma measurement error that can be tolerated in the presence of a 0.1ft one sigma roof support pullup error and a one sigma conveyor placement error of 0.02ft is $0.03^{\circ}$. These results remain unchanged even if the angle between one conveyor section on either end cannot be measured and hence assumed to be zero.

Conversations with mining personnel at Old Ben Mining Company where they are presently operating longwall faces indicated that the roof support pullback error does not exceed one inch. In addition, a ram placement error of 0.02ft (0.24 inch) represents approximately one part in 100 for a 30 inch advance which can easily be achieved with state-of-the-art instrumentation. Therefore. it is seen that the expected system errors are within those allowable for stable face advancement as determined via computer simulation, making the basic angle cart system a viable way of implementing yaw face alignment.

The weighting and bias estimation algorithm for determining the conveyor profile using the angle cart measurements requires that the angle between the conveyor sections on both ends be known with respect to a line perpendicular to the headgate and tailgate. Once these values are known the algorithm makes use of the fact that the conveyor profile could be computed with the same set of measurements proceeding from either end. Therefore, two coordinates could be determined for each joint of the conveyor which then could be combined to obtain a minimum variance estimate of the conveyor profile. This technique yields better results than the basic angle cart measurement scheme by allowing larger measurement errors, or, for the same measurement errors yielding a more accurate determination of the conveyor profile. However, in order to obtain any benefit from the bias and estimation algorithm the angle of the end conveyor sections with respect to the desired reference must be known to $0.2^{\circ}$ or better. This would require that specialized instrumentation be located in the headgate and tailgate to determine these angles to the required accuracy. Therefore, the basic angle cart measurement scheme is the technique recommended if the angle cart implementation is used for yaw alignment.

## Directional Gyro Performance Characteristics

In order to evaluate the performance of a directional gyro implementation for yaw alignment and to specify the required gyro for satisfactory system performance a directional gyro error model had to be formulated. In this model the errors considered are:

1) Low frequency random walk error (white noise through the integrator)
2) Constant drift error
3) High frequency error (white noise through first order filter)
4) Gyro reference error

Various types of algorithms for processing the directional gyro data in order to determine the conveyor profile were investigated. The results of these investigations indicated that the best algorithm for processing the directional gyro data is one that used a single measurement as soon as the gyro is aligned in a particular conveyor section. The measurements thus obtained are then put through an optimum filtering scheme such that a minimum variance estimate of the conveyor profile is obtained.

Investigations into the allowable system errors using the above algorithm with a shearer traverse speed of 30 ft/min, indicated that a spectral noise intensity of approximately $1 \times 10^{-7}$ $(\text{rad/sec})^2/(\text{rad/sec})$ could be tolerated in the presence of gyro reference error of 0.2 degrees and a roof support pullup error of 0.1ft (1.2 inch), for stable yaw advancement. This value of spectral noise intensity corresponds to a random drift error of approximately 0.6 deg/hr which is

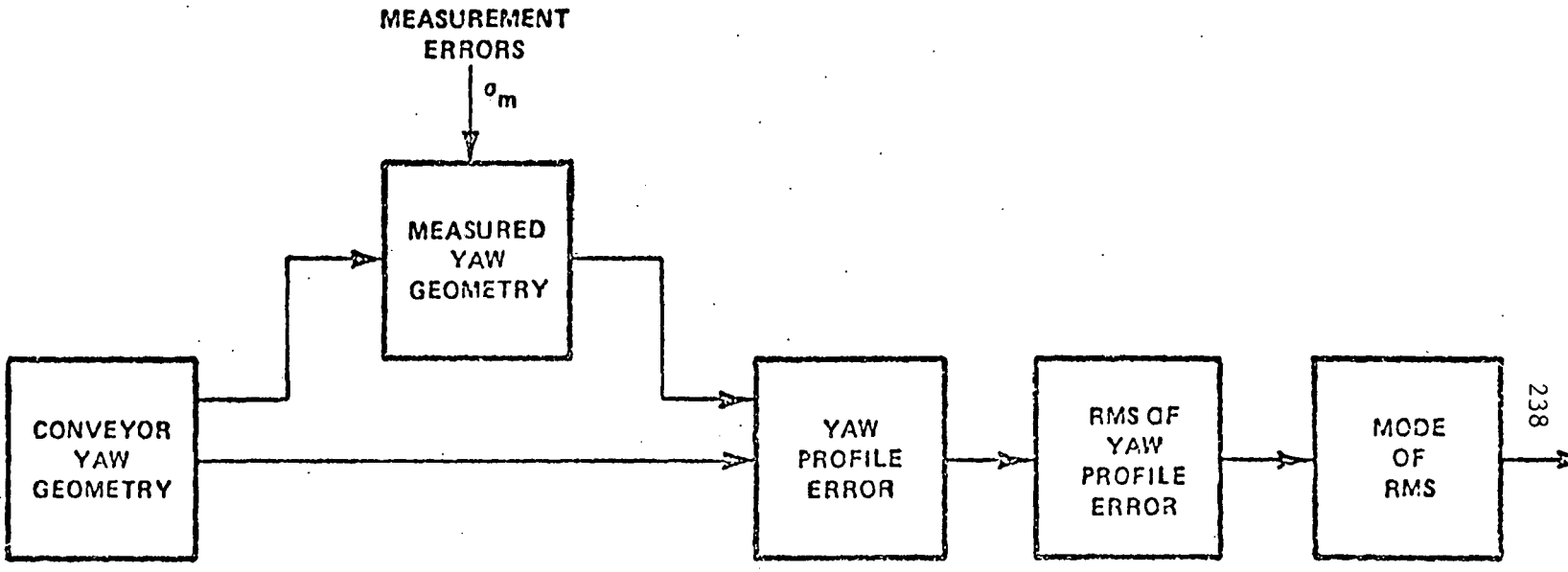



Figure 4. Monte Carlo Simulation

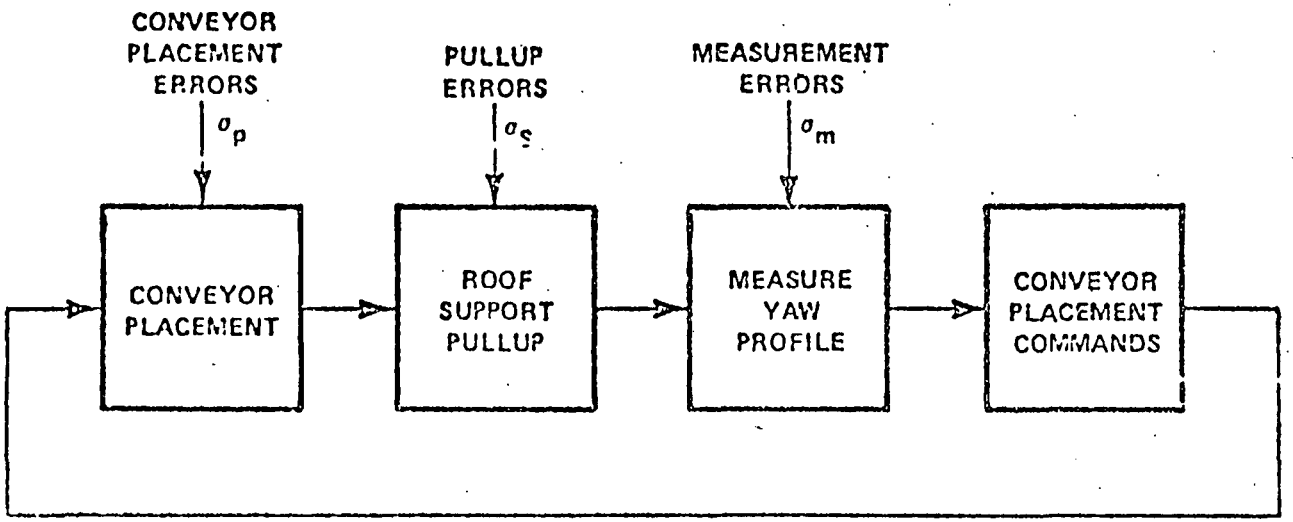



Figure 5. Yaw Advancement Simulation

considered to be a good and relatively expensive gyro. In order to increase the allowable random drift error that can be tolerated the gyro would have to be towed along the conveyor at a greater speed than 30ft/min. This implies that the gyro would have to be mounted on a separate cart which would either have its own motive power or possibly be towed by the conveyor. In either case, the money saved by allowing the use of a cheaper gyro would have to go into the cost of manufacturing a separate gyro cart. In addition, the use of a separate gyro cart would introduce operational problems with respect to integrating the measurement with standard longwall operations requiring additional time to make the required measurements, and would almost certainly preclude the measurement of the angles of the end conveyor sections due to interference with the shearer. Therefore, the recommended system implementation for yaw face alignment is the angle cart system employing the basic measurement scheme described above.

Recommended Yaw Alignment System Performance

The computer simulation results discussed earlier indicated a sensitivity to roof support pull back error. The results have shown that only 0.1ft. one sigma of roof support pull back error could be tolerated for stable yaw alignment system performance defined to mean indefinite roof support advance without requiring manual intervention for face straightening. This value of roof support pull back error was judged as being too restrictive and higher fidelity and more detailed mathematical and computer simulation models were generated to examine more precisely the allowable Yaw Alignment System errors. Various system configurations were studied which included:

a) Actively commanding every roof support
b)Actively commanding every second roof support
c) Actively commanding every third roof support

When every roof support is commanded the hydraulic rams attached to the conveyor are instructed to implement a computed straightening command based on angle cart measurements. When every second or third roof support is actively commanded, the hydraulic rams attached to the conveyor for those supports that are not commanded are unlocked. The rams for these supports are then allowed to float to whatever position they reach when those rams that are commanded execute their commands. For each of the system configurations described above three types of control algorithms were examined:

1) Full straightening
2) Periodic straightening
3) Partial straightening

Full straightening means that everytime the conveyor is advanced the control will be so as to place it in a straight line perpendicular to both headgate and tailgate. Periodic straightening is accomplished by advancing the conveyor without straightening for several advances and then straightening it. This can be done in several ways. One way is to straighten it when it gets crooked by more than a specified amount. Partial straightening advances the conveyor so that it is straight - but not fully straight.

The results of the analyses and simulation studies conducted indicate that the one sigma ram placement error could not exceed 0.004ft. (0.048 inch) for stable yaw alignment system performance. This is an extremely small error and great difficulty would be encountered in designing a conveyor placement system of such accuracy. When placing every second or third conveyor section ram placement error of 0.1ft one sigma has little effect on system performance. In addition the roof support pullback error that could be tolerated is as much as 0.20ft one sigma which can be met by present longwall conveyor and roof support systems. In general, the results indicated that placing every second or third roof support yielded similar allowable system errors (i.e. measurement, placement, and pull up errors) for stable yaw alignment system performance.

The investigation into the various control algorithms listed above indicated that full straightening yielded the poorest system efficiency where efficiency is defined as follows:

$$\text{Efficiency} = \frac{\text{Actual Volume of Coal Cut}}{\text{Volume of Coal Cut with Full Advance and No Errors}}$$

For a one sigma roof support pull-up error of 0.2ft. the system efficiency for full straightening on every pass was approximately 76 percent, 85 per cent for partial straightening on every pass and 85.5 per cent for periodic straightening.

It can be seen that combining the periodic and partial straightening control algorithms, i.e., partially straighten the conveyor periodically when it gets sufficiently out of alignment, increased yaw alignment system efficiency could be achieved. A simulation combining these two control algorithms was performed and system efficiency increased to approximately 88 percent for a maximum sigma roof support pull up error of 0.2ft.

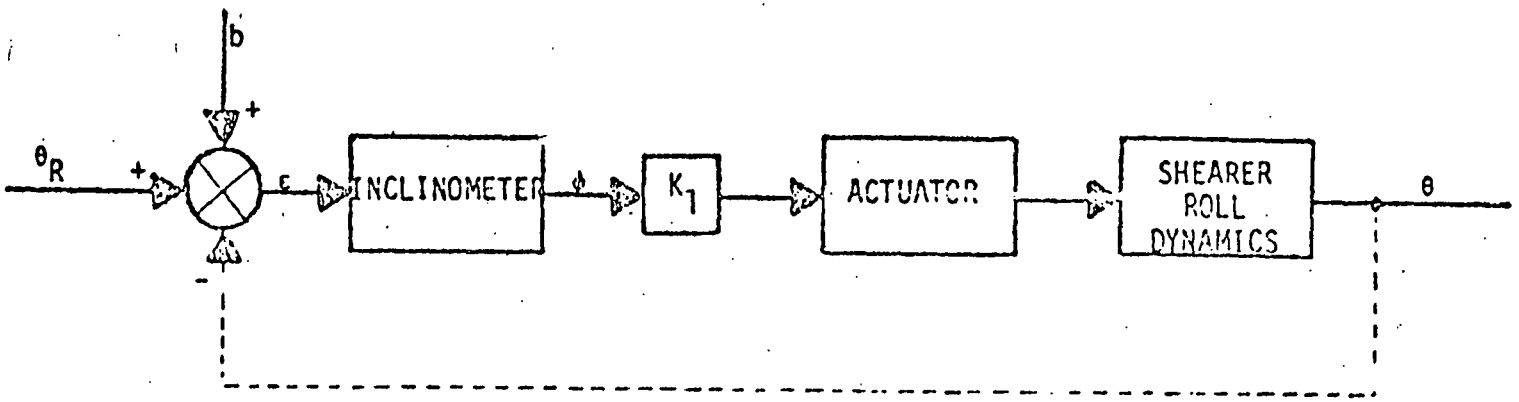
Therefore, the recommended yaw alignment system configuration and control algorithm is to actively place every third roof support with periodic partial straightening.

Roll Control System

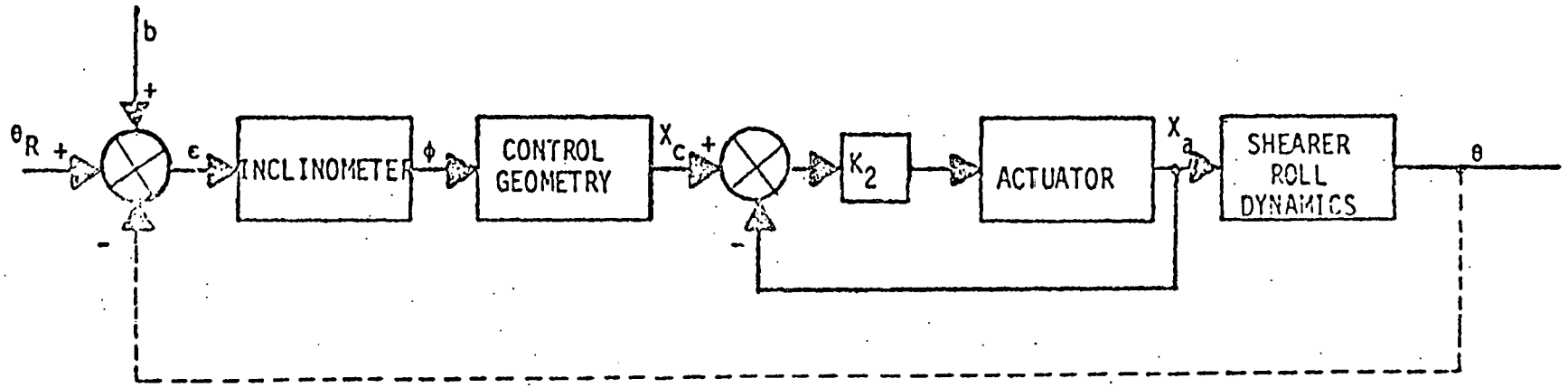
The function of the roll control system (RCS) is to provide an additional degree of control for the shearer. The shearer is equipped with hydraulic actuators which allow it to be rotated about its longitudinal axis, thus tilting both of the shearing drums relative to the coal seam. The purpose of roll control is to provide the capability to correct for twists and undulations in conveyor. These twists can occur when coal or other debris becomes lodged under the conveyor.

The roll sensor used in the RCS is an inclinometer. Since the system is intended to be active during shearing operations, it will be subject to vibrational disturbances which will affect its performance. The inclinometer relies on gravitational acceleration to detect roll angles and, therefore, any vibrational accelerations along the sensitive axis of this device are interpreted as roll signals and thus represent noise (referred to as the cross-axis acceleration) in the system.

Two control loop implementations were investigated. One has the control loop closed around the inclinometer, i.e., the inclinometer is within the closed loop. (Figure 6a). The advantage of this




(a) OPEN ACTUATOR LOOP




(b) CLOSED ACTUATOR LOOP

Figure 6. Active Roll System Control Loop Implementations

implementation is that there is no need to instrument the hydraulic ram. However, since the inclinometer is within the closed loop any filtering that might be added to eliminate the effects of cross-axis acceleration will affect overall control loop stability and hence is an added constraint on the filter design. The other implementation closes the control loop around the hydraulic actuator with the inclinometer being external to this loop and furnishing what essentially can be looked at as an update signal. (Figure 6b). This implementation has the advantage of eliminating the dependence of control loop stability on the filter charactertistics that may be used to eliminate the effects of cross-axis accelerations, thus enabling the filter design to proceed from these considerations only. However, this implementation requires that the linear hydraulic actuator be instrumented.

A computer simulation for the roll control system was developed and consisted of the following:

a)Nonlinear model of the hydraulic actuation system.

b) Second order inclinometer dynamics.

c) Cross-axis accelerations due to the cutting forces and torques as the shearer proceeds along the face.

This simulation was used to evaluate the performance characteristics of the two roll control system implementations described.

## Roll Control Loop Bandwidth Determination

In order to choose the parameters for both control loop implementations the desired loop bandwidth had to be determined. This was done by estimating the maximum input frequency that the roll control loop would have to respond to. Assuming that two adjacent 5 foot conveyor sections are twisted in opposite directions, an input frequency of 0.05 Hz results for a shearer traverse speed of 30 ft/min. Hence the desired roll control loop bandwidth was set at 0.05 Hz.

To choose system parameters so that the desired 0.05 Hz control loop bandwidth would be realized, system frequency responses were taken at various amplitude inputs ranging from the maximum commanded roll angle of $\pm 5$ degrees to one quarter this maximum. The reason for taking frequency responses at different input amplitudes is due to system nonlinearities which make system response characteristics a function of input amplitude. The gains that were finally chosen for both control loop implementations were such that a control loop bandwidth of 0.05 Hz would be realized at a 2.5 degree input, or one-half the maximum possible command. This was judged to be adequate since it is highly unlikely that the roll control loop would be required to command a larger value than 2.5 degrees. In addition, the loop bandwidth also falls off due to flow constraints and attempting to achieve the desired loop bandwidth at 5 degree inputs would probably require a hydraulic modification which isn't desirable.

## Roll Control Loop Performance Characteristics

Both control loop implementations perform well (i.e., RMS errors between 0.05 to $0.1^{\circ}$) in the absence of cross-axis accelerations, with the implementation that closes a control loop around the actuator being slightly better. However, system performance for both implementations degrade significantly even when relatively small values of cross-axis accelerations are introduced. The reason for this sensitivity becomes clear when one examines the value of RMS g represented by a 10 percent error in tracking a 1.77 degree (i.e., $\pm 2.5^{\circ}$ peak) input signal. This value is approximately $3 \times 10^{-3}$ g RMS which means that if the sensed accelerations are in error by approximately 3 milli g's a ten percent tracking error should be expected. Or alternately, if the cross-axis acceleration sensed by the inclinometer mounted on the shearer has an RMS level to approximately 3 milli g's within the control loop bandwidth at least a ten percent tracking error will result. In addition, it should be noted that if the cross axis RMS g levels within the control loop bandwidth are in the order of three milli g's or greater, filtering of the inclinometer output will not yield appreciably better performance if the control loop bandwidth is to be maintained at 0.05 Hz. This result was verified by simulation when the inclinometer bandwidth was reduced from 2.4 to 0.24 Hz to simulate the effect of filtering.

A number of alternate roll control system implementations were considered in addition to the base line concept of a shearer mounted inclinometer controlling in real time as the shearer traverses the longwall face. These alternate roll control systems implementations are listed below:

a) Inclinometers mounted on a seperate cart

b) Inclinometers mounted on the conveyor proper

c) Inclinometers mounted on the roof supports

d) Inclinometers mounted on the shearer and controlling during a clean up pass

e) Inclinometers mounted on shearer and measure without cutting on clean up pass

f) Inclinometers mounted on shearer, stop at various points and measure roll

Simulation studies, however, indicated that in spite of improved performance in some cases, the above alternate roll control concepts are unsatisfactory because they resulted in unstable systems or required complicated system implementation or operation.

The conclusion of this portion of the study is that since both of the control loop implementations originally considered yield almost identical performance for all operational conditions, the open-loop actuator implementation (i.e., the inclinometer within the feedback loop) is recommended since it eliminates the requirement for instrumenting the hydraulic ram. However, due to the sensitivity of roll control system performance to cross-axis accelerations, it is recommended that measurements be taken of the cross-axis acceleration levels on a longwall shearer while mining coal, particularly within and around the expected roll control loop bandwidth. It is possible that when these measurements become available that it might be more economical to remove the inclinometer from the feedback loop and implement a position loop around the actuator in order to better filter the inclinometer output.

## Design Outline for Automated Longwall System

The following summarizes the prototype prelim-

inary design of the Vertical Control System, the Face Advance System and the Master Control Station. (See [4] for details).

## VCS Electronic Design

The heart of the electronics for the automated longwall system and hence the VCS is the Electronic Control Module (ECM). The ECM accepts signals from all of the shearer mounted sensors, operates on them and drives the appropriate control elements (actuators) to achieve satisfactory automated system operation. The major electronic blocks comprising the ECM are signal conditioning, communication subsystem, Input/Output (I/O), and Central Processing Unit (CPU). An overall block diagram of the major blocks comprising the ECM and the interfaces between them and the rest of the automated longwall system elements is shown in Figure 7.

There are three major electronic design considerations: (i) Analog vs Digital System implementation, (ii) Multiplexing vs Direct routing of sensor signals, and (iii) Microprocessor or Minicomputer implementation.

Due to the large electrical motors on the shearer (i.e., drum cutter motors, shearer haulage motor) face conveyor, stage loader, and panel conveyor the potential for electromagnetic interference is high. Therefore digital electronics and sensors were used wherever possible in order to minimize the effects of E&M interference. Next it was necessary to establish whether the digital sensor signals should be brought directly to the ECM via cabling, thus maximizing the number of lines in the cables employed, or to use serial or parallel digital multiplexing techniques that would reduce the number of lines going to the ECM. The result of the trade study performed indicated that the electronic complexity and cost introduced by multiplexing and the loss of flexibility in the manner data could be transmitted to the ECM outweighed the savings that could be realized by eliminating some lines in cables. Therefore, the digital sensor data is brought directly to the ECM without employing any multiplexing techniques. An estimate of the storage capacity and computational power required for the implementation of control laws, resulted in a system size that could be handled in a very flexible and cost effective manner by present day microprocessors. Therefore, a microprocessor based system was decided upon for the electronic design of the ECM. Moreover, CMOS technology was selected wherever possible to further reduce the power requirements and to obtain high noise immunity. The above design decisions has resulted in an intrinsically safe ECM.

System safety was a prime consideration in the electronic design of the VCS. The status of the various sensors mounted on the shearer are continually monitored. Should there be an indication that current/voltage levels are exceeding nominal limits a warning is flashed to the MCS operator. If the voltage/current limits exceed maximum allowable limits, power is removed from the affected sensor and system operation is interrupted. In addition the manual controls on the shearer is safely interlocked with VCS electronics allowing the automatic/remote modes of operation only if the manual controls are in their proper (i.e., off) positions.

Algorithms are also incorporated to determine whether a particular measurement obtained from a sensor is reasonable. If the sensor measurement is considered unreasonable then the last value of the measurement is employed. Should a particular signal fail to pass the reasonable test criteria a warning is flashed to the MCS operator.

The electronic design of the VCS assumed that an active nucleonic CID was employed for shearer drum control although this is not the recommended CID. The reason for this approach was the desire to make the electronics accommodate both the passive radiation and active nucleonic CIDs. The active nucleonic CID has a more complex electronic implementation, particularly with regard to the requirement for radiation hazard monitoring, and the additional encoding needed for the two independent suspensions. Therefore the electronic design that resulted can readily accommodate the passive radiation CID by eliminating those functions peculiar to the active nucleonic sensor.

## Face Advance System Electronic Design

The Yaw Alignment System consists of the angle cart, electronics in the Electronic Control Module (ECM) and electronics mounted on each of the roof supports along the face. The angle cart measurements, made as the shearer traverses the face are sent to the ECM. On the basis of these measurements the face (i.e., conveyor) alignment is computed and control signals generated for the actively controlled roof supports. These roof support advance commands are sent from the ECM via the communication link to the MCS and then to the individual roof supports utilizing a time synchronous multiplexing technique. Once the commands are received by the roof supports they are stored and implemented at the properly prescribed time. The electronics control box on the individual roof supports control the roof support advance, monitor the sequencing and send a signal back to the main processor in the ECM once the roof support advance commands have been properly completed. The electronic packages on all of the roof supports are active, including those supports whose placement rams are not actively commanded, since each of the roof supports must advance and hence its sequencing must be monitored and controlled. The power for the roof support mounted electronics is furnished by the MCS power supply mounted on the stageloader.

The electronics needed to affect roll control is located in the ECM. The output of the inclinometer is fed to the ECM where the central computer processes the data and issues the approximate roll actuator commands to accomplish roll control.

## Master Control Station Electronic Design

The Master Control Station (MCS) located and mounted on the stageloader is the primary man/machine interface and the central monitoring and command station of the Automated Longwall System. Dedicated displays are furnished by which the console operator can monitor longwall sytem performance during the alternating modes of operation. Various malfunction indicators are present that alert the console operator to minimal and potentially dangerous conditions. In addition the status of various other elements in the longwall min-

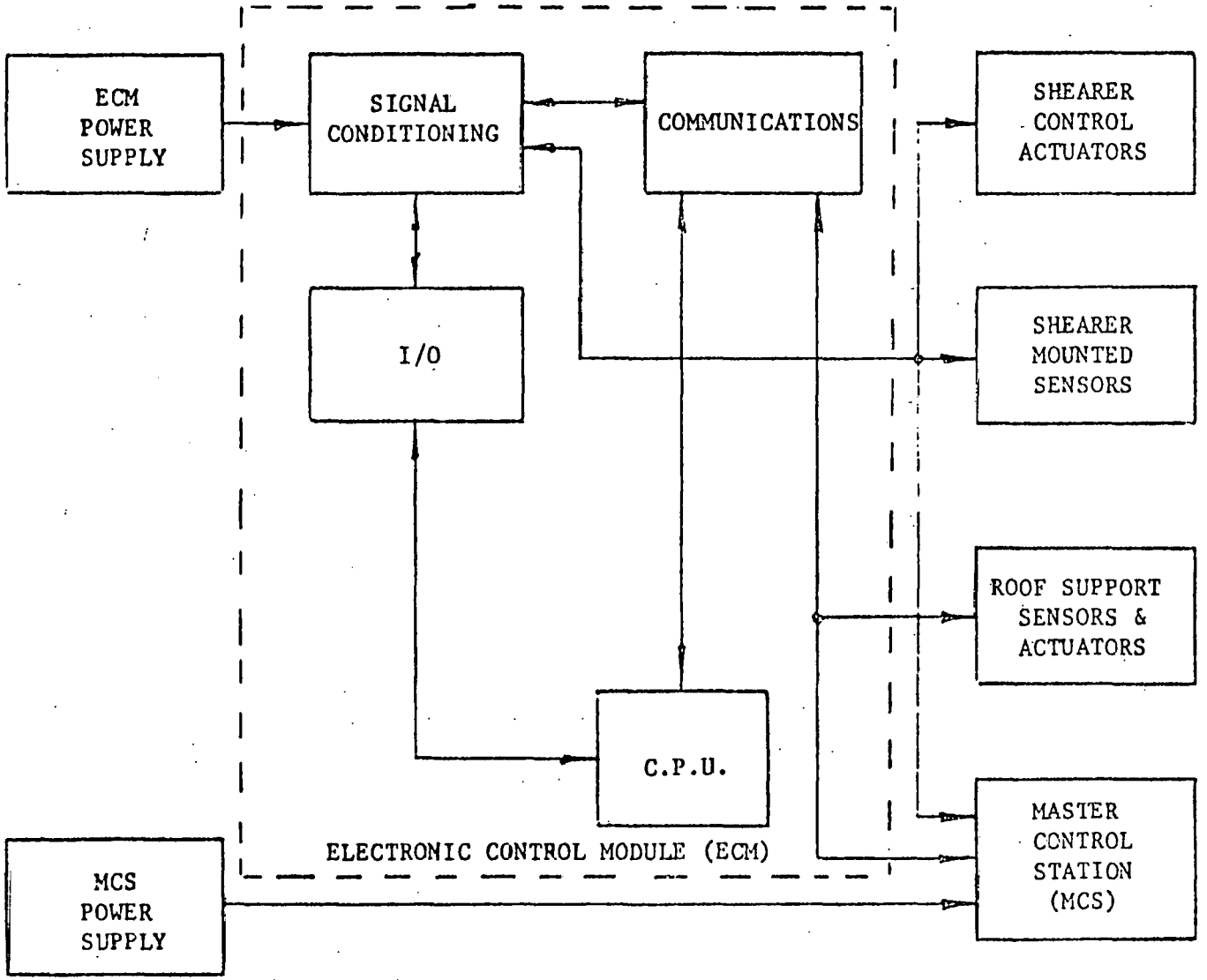



Figure 7. Electronic Design of Automated Longwall System

ing system other than the automation equipment are also monitered. These included currents in the panel conveyor, two stage loader, and two face conveyor motors the status of which are critical if coal spillage is to be avoided if one of these motors/conveyors are overloaded or fail.

Appropriate controls and displays are furnished on the MCS to enable an operator to operate the Automated Longwall System remotely.

The MCS has a digital address system (DAS) through which virtually any sensed, computed, and control variable can be called up and displayed on its alphanumeric display. The DAS will be used extensively during system checkout, and malfunction/fault isolation to enable a fast and economical means of performing these functions. The DAS is also used to override numerous system holds that are included in the design to protect the longwall from inadvertent damage. However there are system holds that cannot be overrided by DAS command, specifically those that would result in hazardous situations. In addition provision is made for the connection of a portable recorder or printer to the MCS and upon the proper DAS command the programs contained in the ECM and MCS microprocessors will be printed out in order to facilitate system debugging. It should be noted that a special key is required to be inserted in the MCS panel to enable remote system operation or to activate the DAS. This is done as a safety precaution to prevent inexperienced console operators from operating the system remotely or commanding the system through the DAS.

Another prime function of the MCS is to provide the central microprocessor in the ECM with required parametric data needed for satisfactory control system operation. Since non-volatile memory is expensive, and using battery power on the face to float the memory when power goes down is undesirable from permissibility considerations, a means must be found to input parametric data needed for control that changes periodically as a function of mining conditions. To this end thumbwheel switches are provided on the MCS which are set to the appropriate values for the parameters they represent. The microprocessor routinely reads those switches. Power interruption will not cause any problems since once power is restored the needed parameters will be read by the microprocessor on the next sampling cycle.

The method by which CID calibration and data are presented to the microprocessor is to use a polynominal curve fit. Examination of the curve relating count rate to coal thickness indicates that a second and certainly a fourth order polynomial is sufficient to characterize it. Assuming that a forth order polynomial would be employed only five coefficients would vary as a function of CID calibration and would have to be presented to the microprocessor. This can easily be furnished by thumbwheel switches on the MCS which is the way it is configured. In addition the MCS microprocessor will have the capability of computing the coefficients required for the polynomical fit. This will be accomplished by inserting measured counts vs. known coal depth via the DAS and the MCS microprocessor would have the capability of computing the required coefficients. These coefficients will then be read on the DAS alphanumeric display and the thumbwheel switches on the MCS set appropriately.

## Mechanical Design

The mechanical design of the VCS that has evolved suspends the CID and present and last cut followers from a parallelogram type mechanism which follows the ranging arm through most of its travel (i.e., within 5 degrees from horizontal). The CID is mounted approximately four feet behind the cutting drum for a 54" drum diameter and maintains this distance constant throughout the design travel of the parallelogram mechanism.

The present and last cut following mechanisms are combined into one cut follower, and depending on shearer direction of motion, will index to follow the last or present cut. The measurement of cut height in either case is with respect to the drum centerline i.e., the height of the cut directly above the drum center of rotation. The cut following arm is in such a position that when an obstruction or void is encountered the follower will be knocked out of the way without being damaged. Hydraulic damping is provided so that the CID will not spring back too quickly once the obstruction is passed. The passive radiation CID is also loaded against the roof primarily to achieve self cleaning (if coal accumulates on the measuring surface of the passive radiation CID a measurement error results).

The mechanical design for the VCS is compatible with all the drum sizes manufactured for the Joy 1LS1-300 shearer which range from 42 to 62 inches regardless of CID being employed. The design will operate in seam heights of 62.8 inches and above for both CIDs being considered.

Another design feature of the mechanical system is that all mechanisms will be locked in place should a hydraulic failure occur. This will prevent the collapse of CID and cut follower suspensions, and the mounting platform, which could possibly result in instrument and mechanism damage. The CID's, cut followers, deployment/stowage mechanisms can be stowed manually by loosening the proper hydraulic fittings.

## Acknowledgment

The longwall system analyses and design described in this paper has been the result of the combined effort and dedication of numerous people. The author would like to extend particular thanks to V. R. Karanam of BEETO for his help in editing of the paper and the valuable contributions he made in the area of optimal control of the VCS and Yaw Control Loop; W. H. Gelbach of The Bendix Engineering Development Center (BEDC) who made significant contribution to the VCS and Roll Control loop designs and performed computer simulations of both; G. A. Cornell of BEDC who made significant contributions to the design of the Yaw Control loop, specification of the alternate roll control loop configurations and the computer simulation of both; T. Smith and H. E. Wells of BEETO for their contribution to the electronic design of the Automated Longwall System.

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# SESSION V

# SUBSIDENCE

Session Co-Chairpersons:

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Research Center
Southern Illinois University at Carbondale

Dr. H. Damberger, Coal Section, Illinois State Geological Survey
Urbana, Illinois

# METHODS OF CHARACTERIZATION OF SUBSIDENCE DUE TO LONGWALL MINING IN THE ILLINOIS COAL BASIN

William P. Santy
Mathematical Statistician

William F. Eichfeld
Civil Engineer

U. S. Department of Energy
Carbondale, Illinois

Robert J. McKelvey
Mining Engineer

Old Ben Coal Company
Benton, Illinois

## INTRODUCTION

In April of 1975, the Old Ben Coal Company and the Bureau of Mines agreed to perform a Demonstration of Longwall Mining in the Old Ben No. 24 Mine. The government's contract adminstration for the demonstration was assigned to the Department of Energy in 1977. The mining of three panels was called for in the agreement and two have been completely mined at this time. Panel #1 was started in September of 1976 and completed in May of 1977. Panel #2 was started in August of 1977 and completed in January of 1979. Panel #2 was started in April of 1979, and is still being mined at this time.

The subsidence monitoring program began in May of 1976 with the first complete survey of the original network being finished the next month. Monitoring of this network is continuing at this time. A second monitoring system was constructed at the site in ay of 1978 and operated five months. .. description of each monitoring system and a comparison of their results is the subject of the paper.

## BACKGROUND

The subsidence monitoring program was conducted in conjunction with the Rock Mechanics Study which was required for the demonstration. The rock mechanics work was undertaken in order to aid in evaluating the mining system and to provide a warning if overburden behavior indicates an unusual condition. The subsidence work benefited not only the Rock Mechanics Study, by increasing knowledge of the overburden behavior, but the field of subsidence research as well.

Two subsidence monitoring systems were employed for monitoring subsidence over Panel #2. The reasons for using two systems were (1) to show the feasibility of using a concentrated system for characterizing subsidence and (2) to compare the results obtained from more traditional monitoring techniques.

One of the main objectives of subsidence research is minimizing the costs for mine operators in dealing with subsidence-related environmental

constraints. The development of improved subsidence monitoring techniques is one method by which this objective can be achieved so the work done as a part of the Old Ben Project is a direct contribution to the field.

## SITE DESCRIPTION

The Old Ben #24 Mine is located approximately three miles northwest of Benton, Franklin County, Illinois. The surface above the demonstration panels is a topographically level, low-lying area at an average elevation of about 383 feet above Mean Sea Level. The difference in elevation over the study area was approximately 5 feet and several areas of standing water existed before mining began. Coal is being mined from the Illinois (Herrin) #6 coal seam which is under about 620 feet of overburden at the demonstration site. The coal seam is from 8.5 to 9 feet thick in the mine with a mining height of 7.2 to 8.25 feet[1/] in the demonstration panels.

The coal seam is essentially flat in the mine but dips northward regionally. The immediate roof in the demonstration panels is shale and limestone with an overburden composed mainly of flat lying shale beds (80 percent). The bulk of the remaining 20 percent of the overburden consists of limestone, siltstone, and claystone beds. The surface soil is predominently clay for a depth of 50 feet.

The demonstration panels are being mined by retreat longwall methods with an Eickhoff EDW 2x 100 L double ranging drum shearer cutting the coal, 95 Thyssen RHS 18/20 L shields providing support and an Eickhoff EDF-3 single-chain conveyor moving coal from the face. Panel #1 was 460 feet wide and 1740 feet long while Panel #2 was 1720 feet long with the same width. Panel #2 is also 460 feet wide, but it will not be as long as the other panels.

## SUBSIDENCE MONITORING

The network of subsidence monuments that was designed for this monitoring program consists of two lines of monuments over the centerlines of panels #1 and #2 (68 monuments over Panel #1 and 64 monuments of Panel #2) and a third line of 64 monuments perpendicular to the centerlines at the panel mid points (Figure 1). A monument spacing of 30 feet (approximately, depth of seam/20)was chosen for the portions of the lines over the edges of the panels while a greater spacing of 60 feet was allowed for less critical areas such as the panel centers and areas outside the limits of mining. A spacing of 120 feet was used for additional monuments at the ends of each line in order to assure that no movement went unobserved. A spacing of 15 feet was specified for the monuments which were placed over the chain pillars between panels #1 and #2 in order to monitor strains in this area. A 15 foot spacing was also used for those monuments which were instrumented as a part of the ADAS. An additional 26 monuments were added over Panel #3 at a later time to augment the existing network. The monuments used in this program were designed to minimize the potential for their disturbance due to surface phenomena, mainly frost heave, by isolating the point used for observation from the surrounding soil.

The monuments consisted of 10 feet of 1-inch pipe driven 7 feet into the ground in two 5-foot sections. This inner pipe was shielded from the surface soil by a 5-foot section of 2-inch pipe driven 3 feet into the ground. The space between the two pipes was filled with closed-cell foam pipe insulation coated with silicone grease. The 1-inch pipe was exposed for 8 to 10 inches above the 2-inch pipe for measuring purposes. The use of an oversized bell-reducer and vinyl caulking where the 1-inch pipe entered t 2-inch pipe allowed free vertic movement between the pipes while sealing the space between them from the

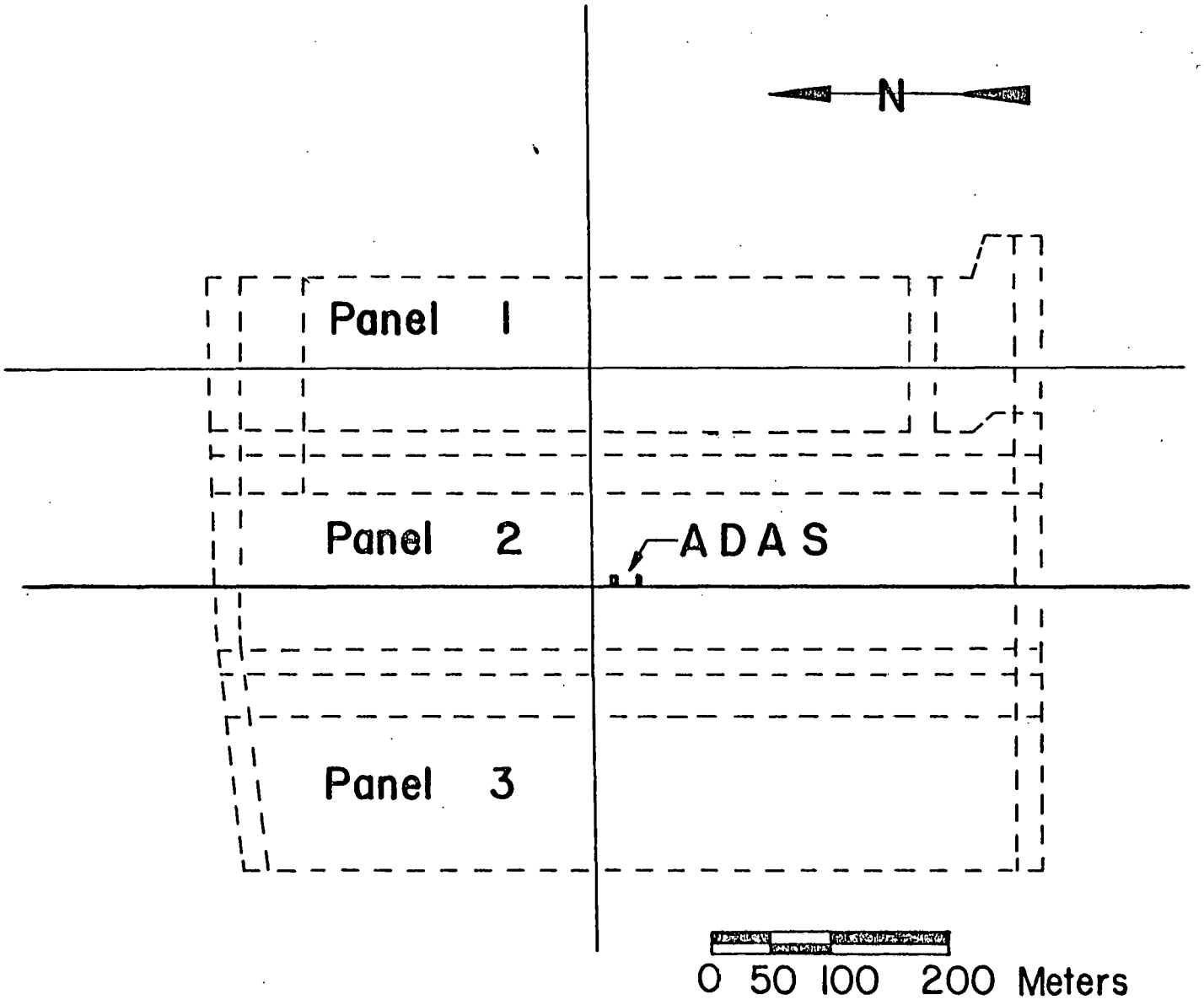



Figure 1

environment. These monuments functioned quite well during the monitoring program.

The bulk of the monitoring program was carried out with conventional surveying methods. Measurements of distance, direction, and elevation were made to all monuments periodically, although only elevations were observed for each monument during each survey. Horizontal surveys were made to Third Order, Class II accuracy standards and vertical surveys were made to Second Order, Class II accuracy standards.

Monthly monitoring consisted of observing the elevations of all monuments and the distances between the strain observation monuments over the chain pillars between panels #1 and #2. In addition, slope distances were measured between each pair of monuments within 200 feet of any monument showing vertical movement so that strains over the active part of the subsidence trough could be determined.

Complete surveys were performed before mining, when half of Panel #1 had been mined, when Panel #1 had been completed, when half of Panel #2 had been mined, and when mining of Panel #2 had been completed. During these surveys all the measurements required for monthly surveys were made and the horizontal positions of all monuments were determined.

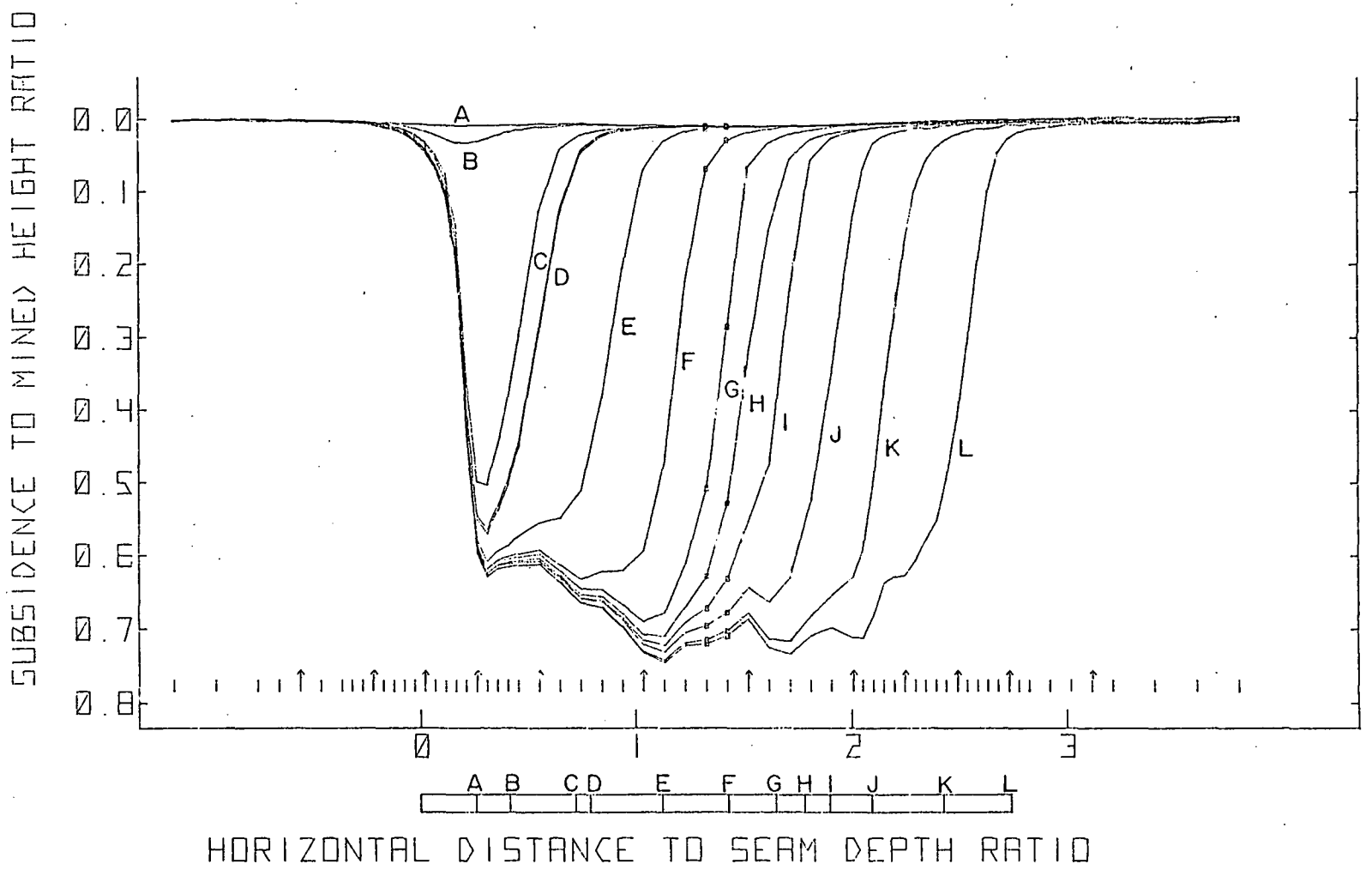
The second system has been named a concentrated monitoring system. It consists of five surveying monuments, located 15 ft. apart, and monitored frequently for a shorter time interval. Part of the concentrated system is an Automated Data Acquisition System (ADAS) which monitors tilt and strain every two hours via an automated data logger. A description of the system and the results were reported by Schmechel etal.[2] In addition to the ADAS data, rod and level surveys and slope distance measurements were made as with the other monitoring system. These measurements were made twice weekly for five months in an effort to characterize the traveling wave.

Thus, there are two systems: a typical monitoring system; and a concentrated monitoring system. Each system measured subsidence with rod and level and each measured strain with a steel tape. By not including the ADAS measurements, we restrict the comparison of the data to measurements made with the same "yardstick," if you will, and do not confound (statistical) differences in systems with differences in measuring devices.

The difference between the two systems, then, is a difference in sampling. The typical system samples a large number of points, but each point only a few times. The concentrated system samples a few points but each one a large number of times. This is the basis of the comparison.

The series of monthly profiles shown in Figure 2 portrays the data collected from the typical monitoring system. The vertical scale is change in elevation divided by mined height. For the purposes of this paper, a constant mined height of 7.00 feet was assumed. The actual mined height ranged from 7.20 to 8.25 feet and as all of this information becomes available, the proper corrections will be made. The horizontal scale represents distance in terms of seam depth. The 0 indicates the beginning of the face, 1 indicates a horizontal distance equal to one seam depth (620 ft.) away. The linear distribution of the monuments which were used for obtaining these profiles is illustrated in the tic marks directly above the horizontal scale. A bar graph directly below the horizontal scale presents the monthly longwall face position in terms of the seam depth.

Because of the relationship between subsidence and face position, this same data can be used to represent subsidence as a function of face position (Figure 3). Figure 4 shows the calculation of face distance to seam depth ratio. The subsidence data from the concentrated monitoring systems is shown in Figure 5. Plotting both data sets on the same graph in Figure 6, the


SUBSIDENCE TO MINED HEIGHT RATIO
0.0
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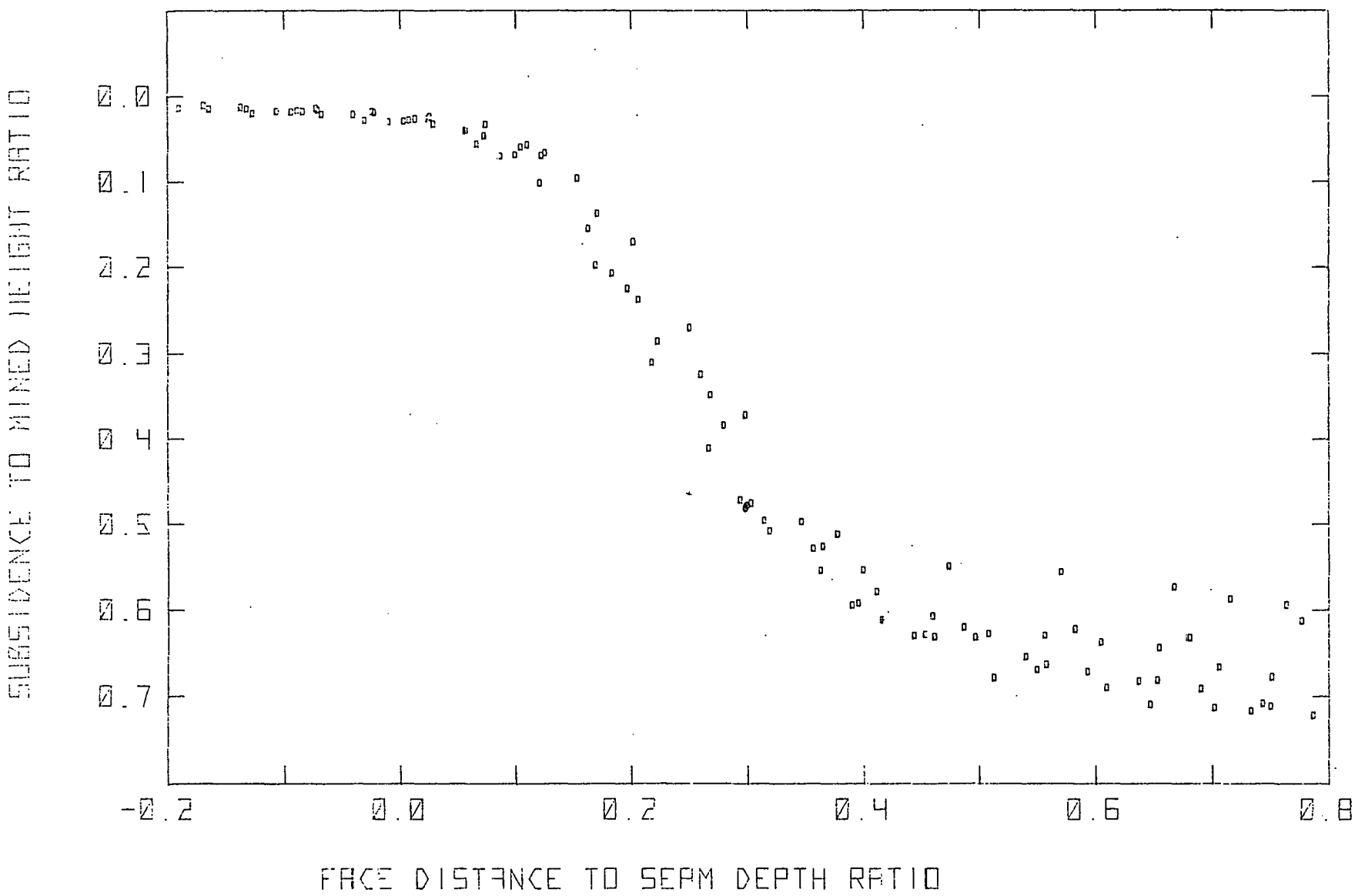
HORIZONTAL DISTANCE TO SEAM DEPTH RATIO

Figure 2.


SUBSIDENCE TO MINED HEIGHT RATIO
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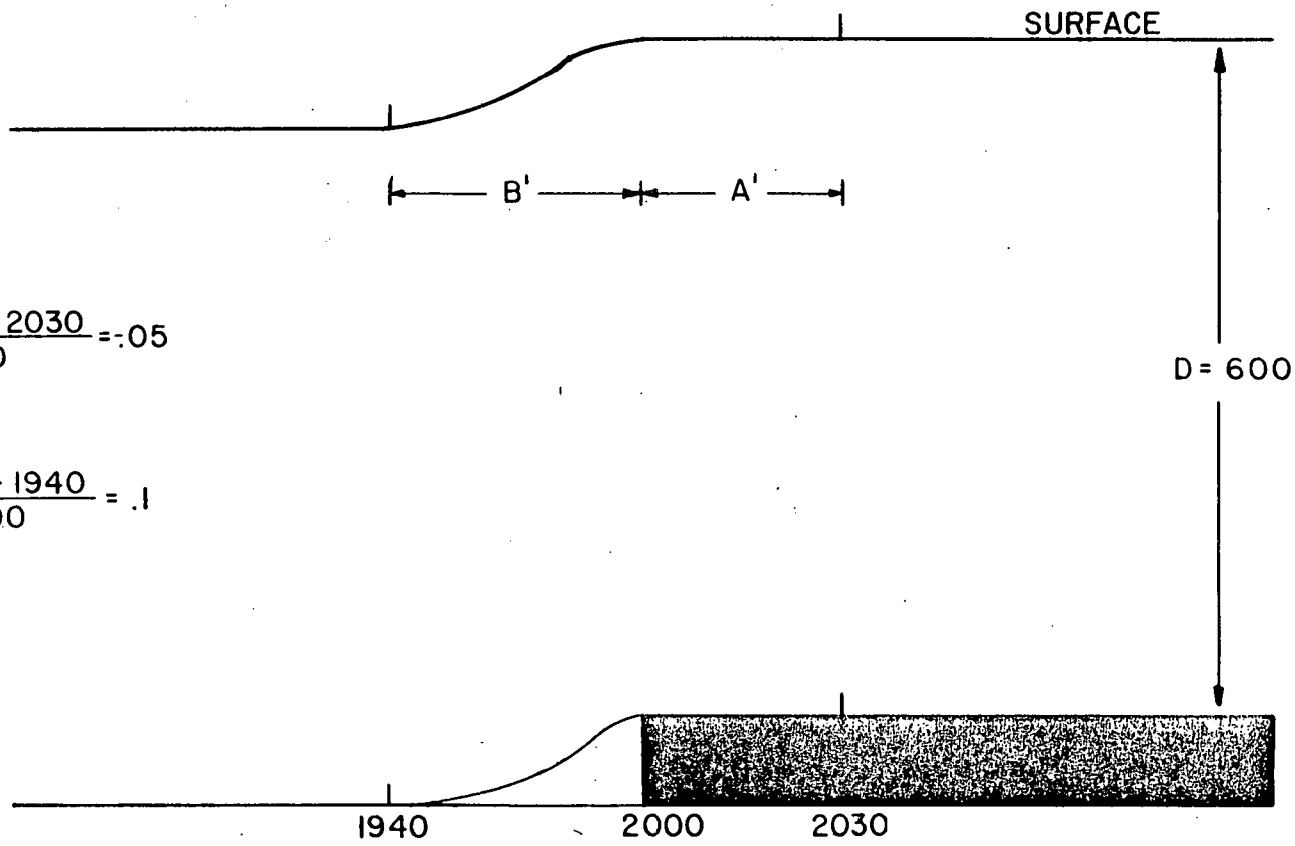
FACE DISTANCE TO SEAM DEPTH RATIO

Figure 3

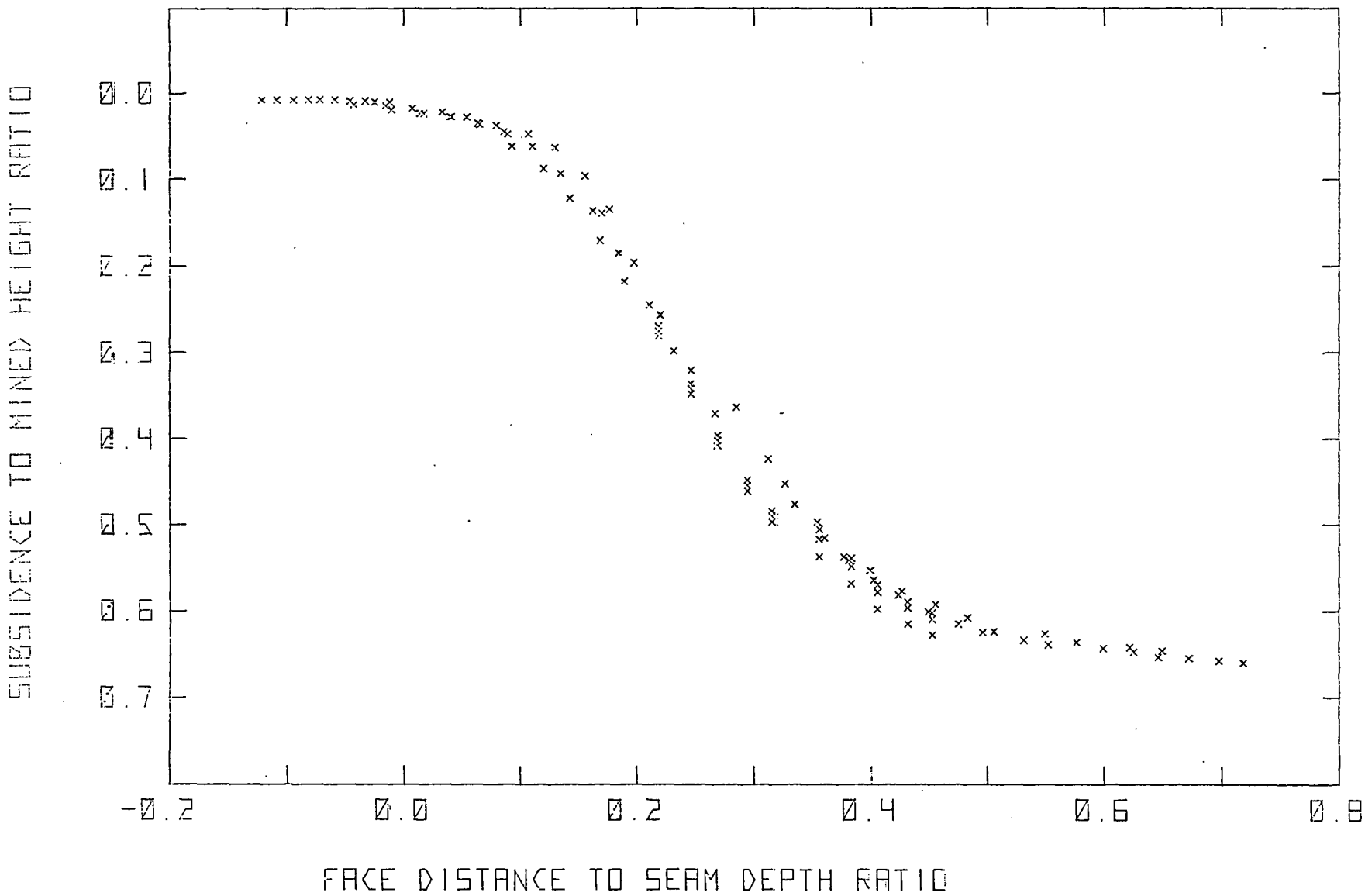
## CALCULATION OF FACE DISTANCE TO SEAM DEPTH RATIO




for $A = \frac{A'}{D} = \frac{2000 - 2030}{600} = .05$

for $B = \frac{B'}{D} = \frac{2000 - 1940}{600} = .1$

Figure 4


SUBSIDENCE TO MINED HEIGHT RATIO
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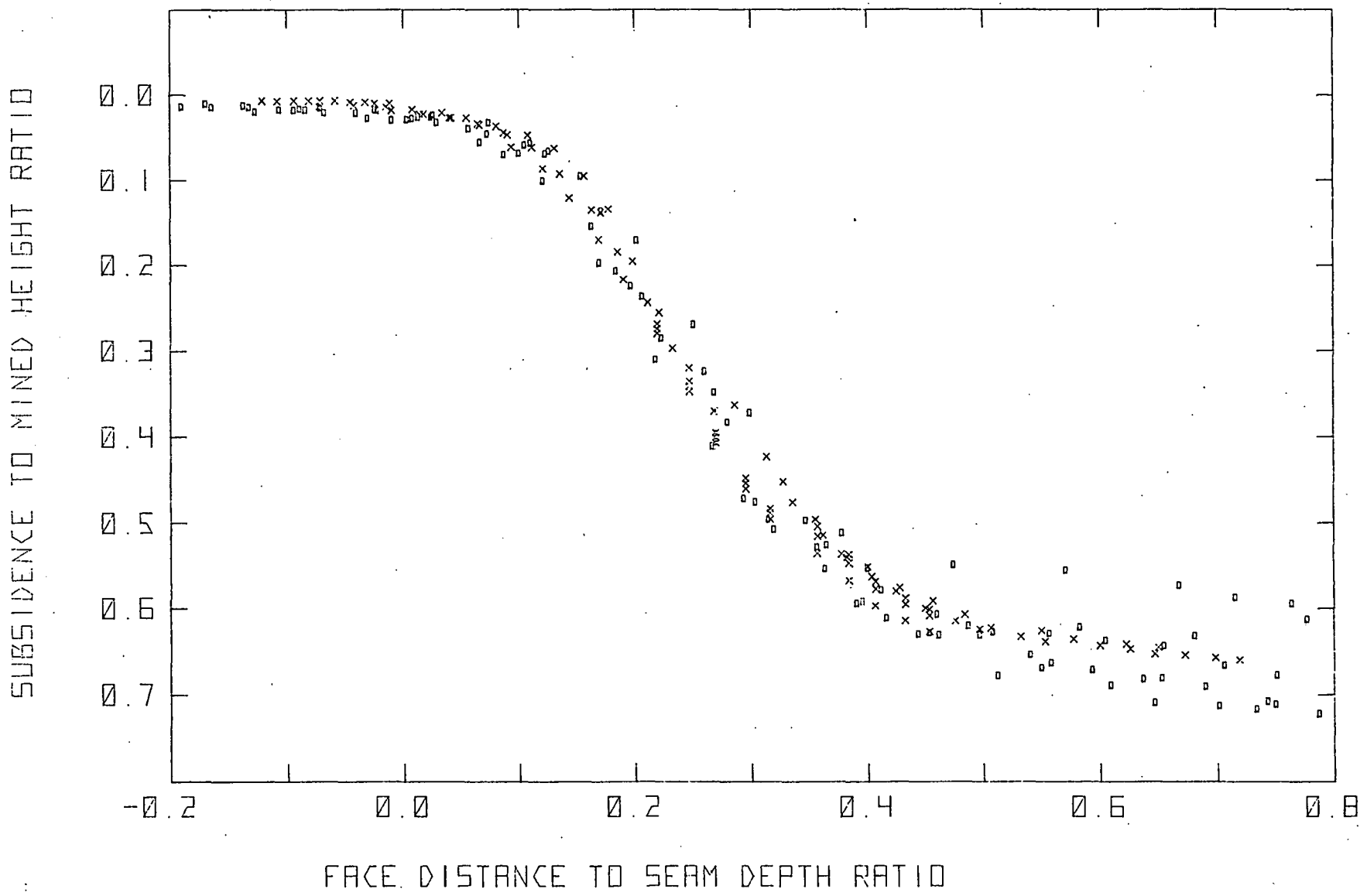
FACE DISTANCE TO SEAM DEPTH RATIO

Figure 5


SUBSIDENCE TO MINED HEIGHT RATIO
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0.4
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FACE DISTANCE TO SEAM DEPTH RATIO

Figure 6

x's from the concentrated and □'s from the typical system, we find close agreement. The biggest difference between the two systems is the scatter in the higher values of the horizontal scale. Plotting the data in this manner shows the typical response of the surface down the center of the panel as the face passes underneath, i.e., the traveling wave.

The strain measurements are presented in Figures 7, 8, and 9, where the □'s represent the typical system and the x's represent the concentrated system. Again we find close agreement between the two monitoring systems. The typical system has more scatter than the concentrated system which is consistent with the differences in sampling techniques.

## CONCLUSION

Only a portion of the data taken during the subsidence monitoring program at the Old Ben No. 24 mine is presented in this paper. The agreement of the two systems in the characterization of subsidence, both vertical displacement and horizontal strain is very encouraging. To recommend one system over the other would be premature--based solely upon a prototype system. This project is just the first of a series of such project directed toward the determination o optimum subsidence characterization systems and cost-effective monitoring techniques.

## ACKNOWLEDGMENTS

The results presented in this paper were obtained from a portion of the data collected on the surface above Old Ben Coal Company's No. 24 Mine. The program was initiated in 1976 by the Bureau of Mines and later transferred to the Department of Energy under contract No. ET-75-7-01-9023. The authors wish to express their sincere gratitude to all those personnel in each of the above-mentioned organizations who made contributions to the success of the project.

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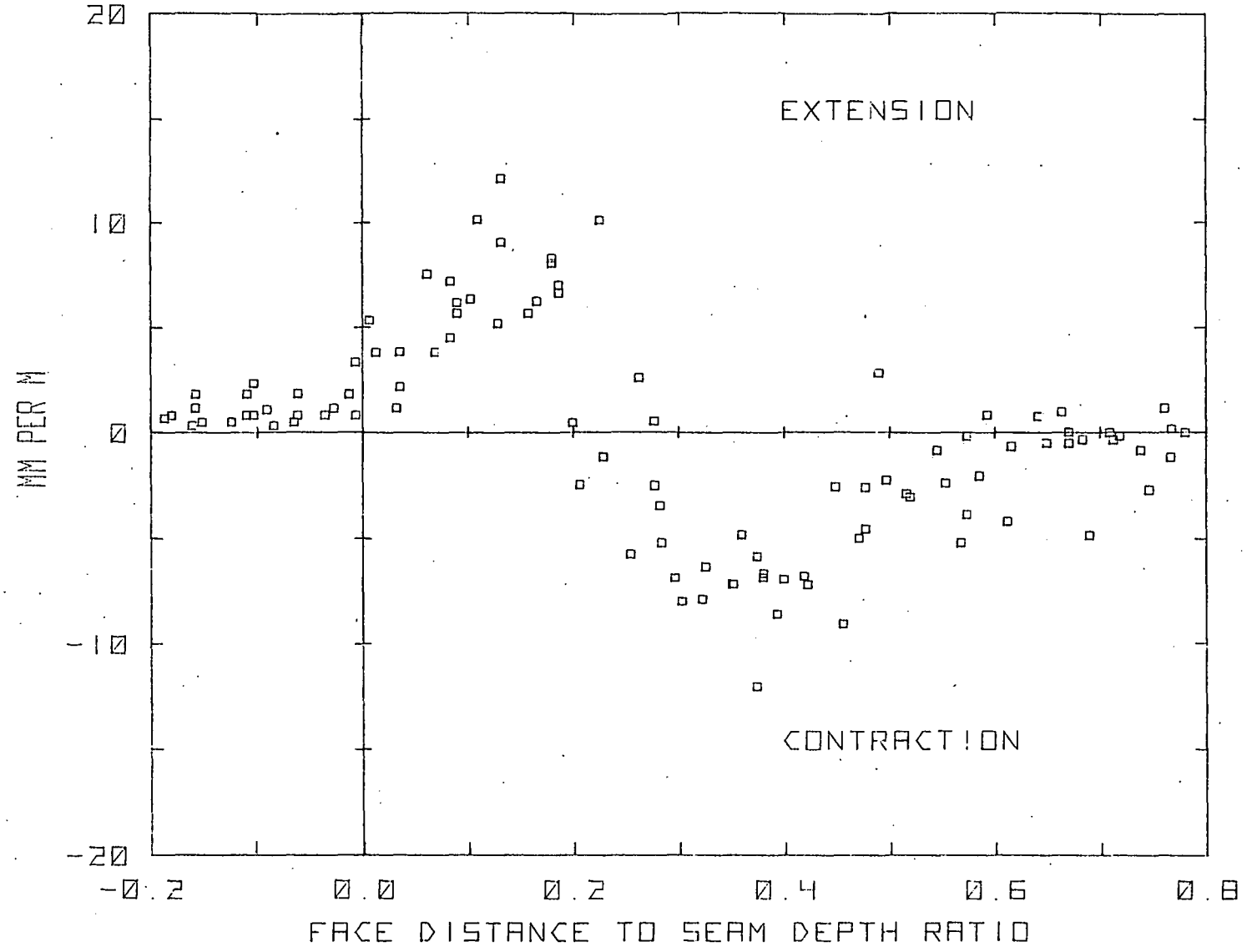
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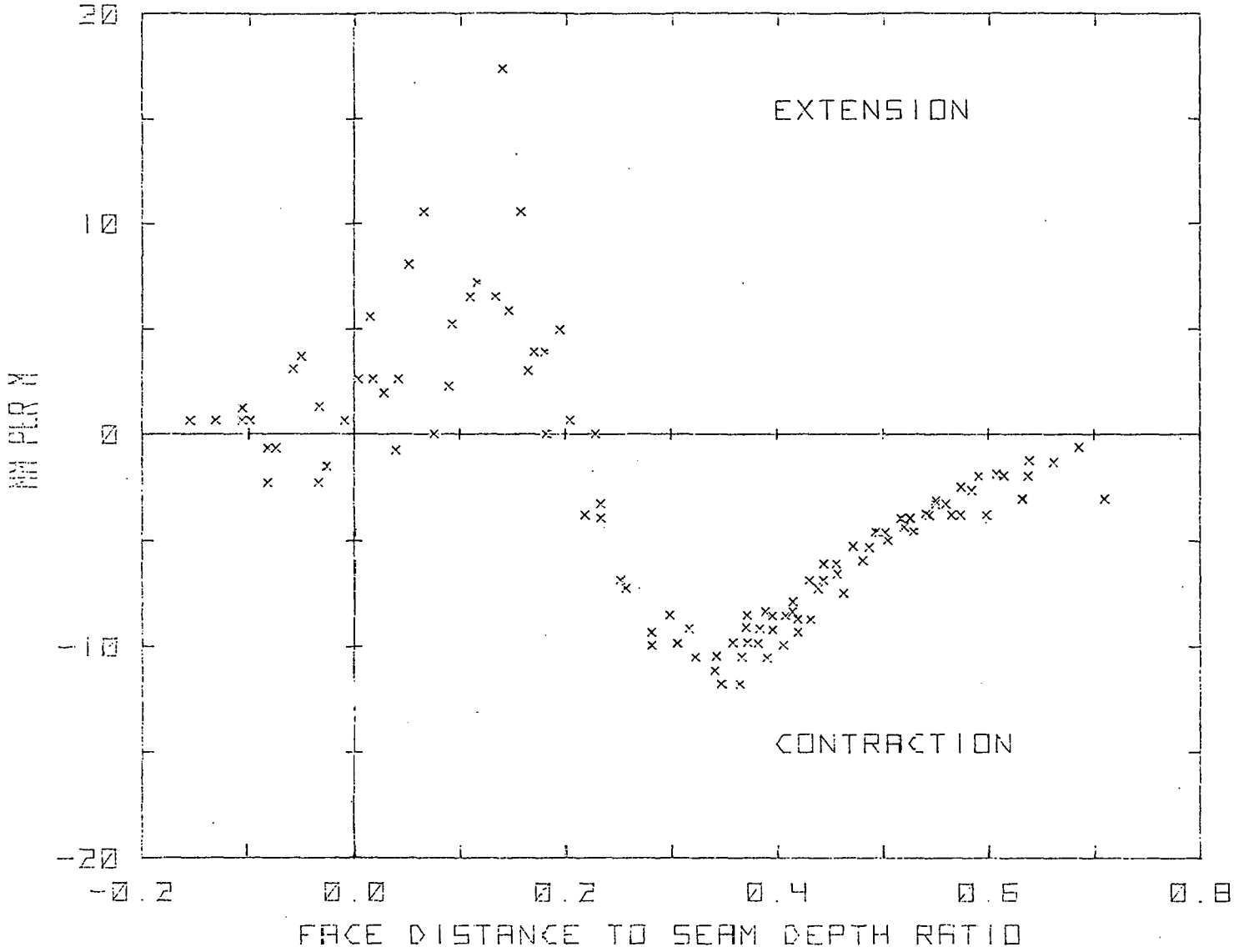
FACE DISTANCE TO SEAM DEPTH RATIO

Figure 7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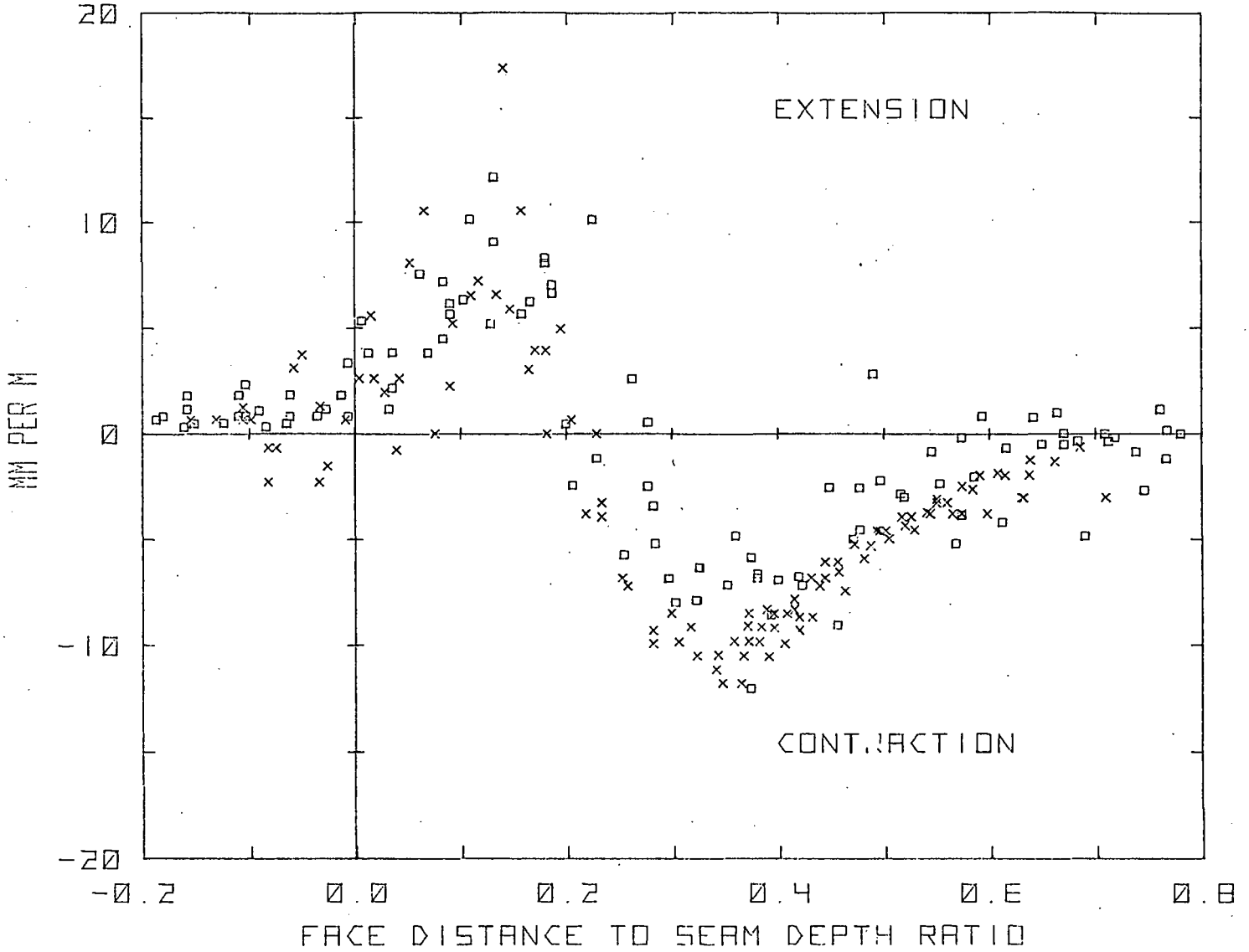
FACE DISTANCE TO SEAM DEPTH RATIO

Figure 8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FACE DISTANCE TO SEAM DEPTH RATIO

Figure 9

# THE TIME FACTOR ON SUBSIDENCE

by

N. B. Aughenbaugh, Chairman
Department Mining, Petroleum, Geological Engineering
University of Missouri-Rolla

## INTRODUCTION

In recent years, subsidence due to underground coal mining has been receiving much attention mostly because residential and commercial developments placed over old, abandoned mines have sustained damage due to differential settlement. With renewed activity in coal mining and the continued development of the surface to urban-type use, conflicts continue to arise between development of coal reserves and surface land use. Some of the questions being asked are: will abandoned mines continue to subside in the future? will modern mines cause subsidence? when will subsidence occur? and what surface form will it take?

The surface expression of subsidence (size, shape and depth) has been investigated and numerous articles have been published on this aspect. In contrast, very little information is available about the time factor on subsidence. The information and data that has been published on the time factor relates only to longwall mining of coal. The authors of the Subsidence Handbook published by the British Coal Board specifically state "None of the foregoing remarks applies to pillar and stall working.", in the section discussing the time aspects of subsidence due to longwall mining (1).

This paper will discuss only the time factor on room and pillar mining of coal.

Numerous beliefs and opinions are held by the general public and professionals concerning subsidence over coal mines. These beliefs vary from place to place but tend to have in common the idea that no subsidence or further settlement of the surface will occur if certain conditions are met. Many people firmly believe subsidence cannot occur if the percent of extraction is less than 45 or some other arbitrary upper limit. The author has been told that once subsidence has occurred at a site, it will not take place again within an area of a quarter or half mile surrounding the sink. Other opinions held are subsidence will not occur if the mine is below a certain depth, or if the strata overlying the mine contains a five feet thick or more limestone bed, or if the mine remains dry. It also has been stated by a professional that subsidence cannot take place if the cavity found by drilling is above the original mine level. Ironically after many of these statements have been made, subsidence has taken place. From the above discussion, it is evident that the time aspect of sub-

sidence is not understood.

## FAILURE OF MINE CAVITIES

Any cavity beneath the earth's surface represents a condition of non-equilibrium and therefore will tend to close with time. Densification of earth materials of which cavities due to mining must be included is a continuing natural process. The question then is not if an underground coal mine will undergo degradation resulting in closing of the cavity and subsidence at the surface, but when and how these events will occur.

The time dependent stability of underground mines is anologous to the structural stability of buildings. If both are misdesigned and constructed, immediate collapse can occur. However, with conservative designs both structures will remain stable as long as maintenance and repair are made on local points of failure. If a building is abandoned and repairs are not made, deterioration of the structure will occur in a snowball-like effect as the unrepaired failures destroy its integrity. Examples are to be seen in city and rural areas alike. The point being stressed is local failures gradually destroy the continuity of a mine which will precipitate a general failure of the entire mine with time.

### Types of Mine Failures

Failures of cavities in coal mines can be classified into four catagories:

1. Collapse of the roof
2. Failure of the floor
3. Failure of the pillars
4. Elastic rebound of the rock

One mode of failure may dominate but generally combinations of the four will occur. Also, initial instability may be one type which will evolve into or be joined by the other modes with time.

The role of elastic rebound is not well defined because very little data has been collected on it. When loaded, a material will deform. The deformation can be divided into two forms: a permanent set which is unrecoverable and an elastic strain that will recover once the loading is removed. The percentage of permanent and elastic deformation is dependent upon many factors such as magnitude of the load, type of materials and length of time of the loading. In general, shallow underground coal mines are thought to be little affected by elastic rebound of the rocks into the cavity.

Failure of the roof is the most obvious cavity stability problem in most operating mines. Upon exposure the roof begins to sag because of lack of support. The bridging capabilities of the roof rock are dependent upon the material strength, geologic discontinuities and reaction of the exposed rock to humidity in the mine air. Failure of the roof can vary in magnitude from gradual flaking off of the surface rock to massive roof falls tens of feet high and hundreds of feet long.

Some active mines are plagued with unstable floors. The mechanics of floor instability can take the form of general heave or pillar punching. The thicker and weaker the immediate floor material, the more prone an active mine will be to floor problems. Abandoned mines will sustain floor heave because of overstressing of the floor material beneath the pillars and seep water softening the underclay.

Pillar failure will take the form of spalling of the exterior as the rock crushes and expands laterally. The process is referred to as "pancaking" because the end result is a shorter and wider pillar.

### How the Failed Cavity Reflects To The Surface

As the cavity collapses and closes by the above processes the failure will reflect towards the surface.

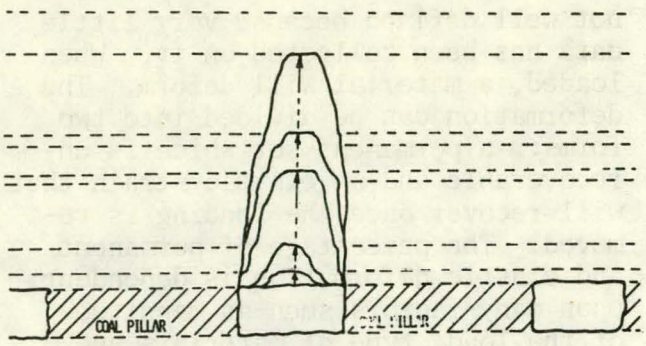



FIGURE 1 Stoping of a Roof Fall

Failure of the roof causes the cavity to migrate upward in an action called stoping. Figure 1, illustrates the process. If stoping reaches the surface, a sinkhole-type of subsidence will take place.

Failure of the floor or pillars will result in a downwarping of the overlying beds as illustrated in Figure 2. The surface expression of

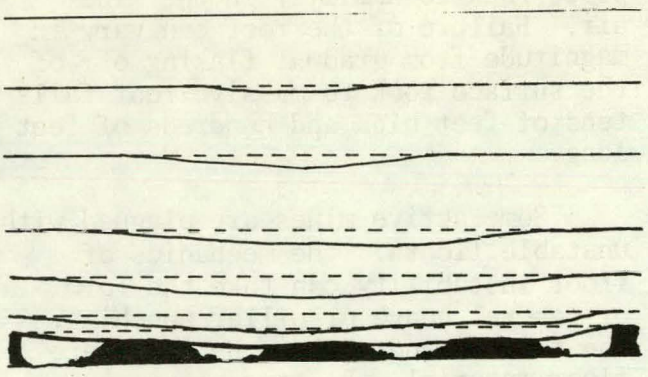


FIGURE 2 Downwarping of Strata Over Crushed Pillars

this process is a basin-type of subsidence.

Fracturing of the overlying strata can occur either with roof failure and stoping or with floor and pillar failure and the downwarping of the overburden. Figure 3, is a picture of a highwall of a strip mine over an abandoned room and pillar coal mine. The arrows point to fractures of the overlying rock due to collapse of the old workings. The fractures will vary in size and position.

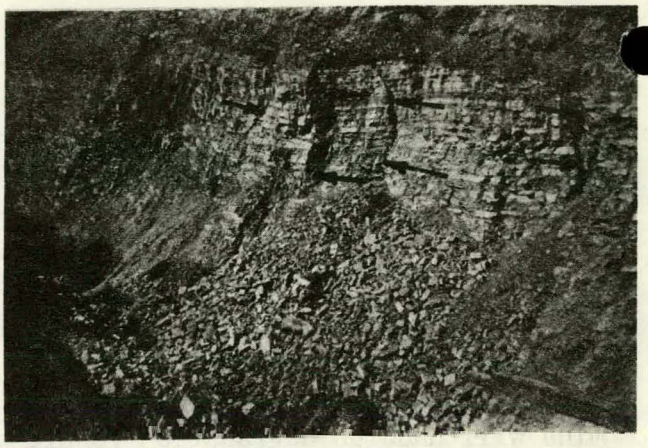


FIGURE 3 Fractures Due To Mine Collapse

Some may extend from the surface to the mine, whereas, others will be relatively short and not intersect the surface. The belief that fractures due to subsidence will occur only at the land surface and extend to a depth of less than ten feet because the surface is "brittle" and the underlying material is wet and "plastic" is not valid.

Because a mine will undergo a combination of different failures the reflection of these failures also will be a combination of stoping, downwarping and fracturing.

Influence of Overburden

The type of material overlying the mine will dictate how and when subsidence will occur. The main reason subsidence problems vary from one coal mining basin to another is because the geologic conditions are different. Rules of thumbs developed from data gathered in one geographic area generally can not be applied to another area.

With few exceptions, soils are weaker than rocks and will have less capacity to maintain a bridge across a cavity than rock. Therefore, the ratio of soil thickness to the entire overburden is a factor of significance in subsidence prediction. When stoping, downwarping or fracturing reaches the soil mass, the time delay in

subsidence manifesting itself at the surface becomes short. The type of soils present also will influence the reaction time. Granular materials such as sands have less bridging capacity than fine-grained soils and therefore will reflect subsidence almost immediately.

Rocks must be catagorized and evaluated two ways. Shales and shale-like rocks are weak and incompetent relative to sandstones and limestones. They are unable to support their own weight plus that of the overlying strata for long periods of time when the support beneath them is removed. In contrast, limestone and sandstone beds are strong enough to bridge over areas where the support has been removed. This especially is true for small areas of stoping due to roof falls as illustrated in Figure 4.

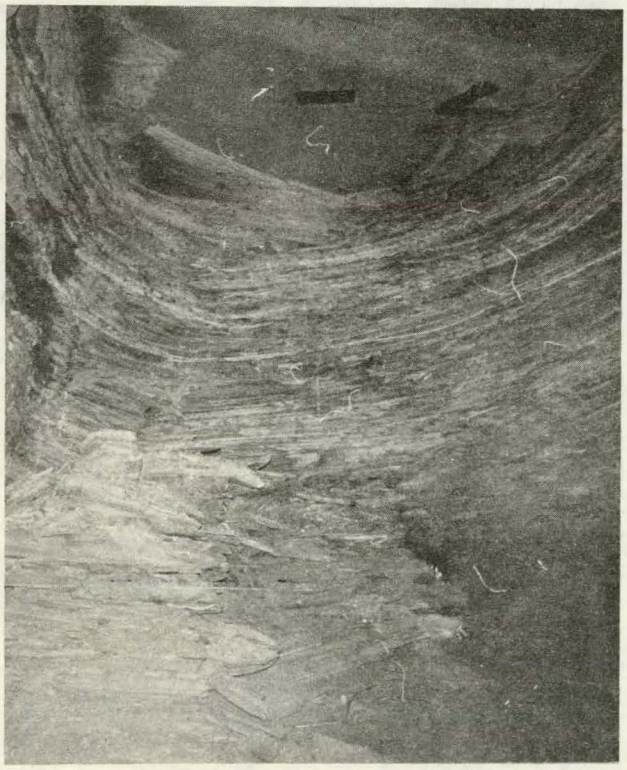


FIGURE 4 Roof Fall Terminating on Limestone Bed

Where larger areas are effected by pillar and floor failure, the span length will exceed the capacity of these competent beds to maintain a bridge over the unsupported mass for any length of time. They too will downwarp and fracture allowing closure of the mine to migrate upward to the surface.

Besides the relative competency of rock types, the thickness of the individual beds also will dictate the ability of the overlying rocks to maintain a bridging effect when the support is removed. Thicker and more massive strata are able to span unsupported areas much longer than thin bedded deposits. Even small lamanae or partings in a massive bed can appreciably lower a rock unit's bridging capacity.

Lateral disruptions of the beds throughout a mine are another geologic factor that will influence how the mine retains its stability and what the subsidence potential will be at the surface with time. Facies changes, lensing and pinchouts can occur to the roof and floor material which will greatly affect the relative stability of the mine. "Bad ground" in mining usually indicates there has been a change from a competent to a more incompetent material in the roof or floor. Lateral variations in the geologic conditions of the overburden will determine what surface areas are first afflicted with subsidence over a mine. The very nature of coal forming environments is one of changing conditions both laterally and vertically with time. Therefore, it is not prudent to assume a thick limestone member found by drilling in one section of the mine will be continuous throughout the mine. It can pinchout, become shaley, or be absent because of a channel cut and fill, or other changesin the depositional environment.

In addition to changes in rock types, geologic structures can adversely affect mine stability and the ability of the overlying strata to bridge areas of failure in a mine. Included under geologic structures are joints, faults, slick-

ensided beds and concretions. All these conditions will increase the possibility of subsidence occurring in a shorter period of time. Folds and "rolls" in strata also are a negative effect.

Other Influencing Factors

To further add to the question of when and how much subsidence will occur over a given site is the problem of evaluating what are the actual conditions throughout the mine. When the mine is active, many things take place that must be known and considered if any kind of subsidence prediction is attempted.

During mining, any roof falls that occur in operating passageways will be removed to nearby crosscuts. The net affect is the place where the roof fall occured is an area where the mine cavity is higher than the coal seam, whereas, the gobbed site has a reduced cavity height. As a result, deeper subsidence basins can occur over these roof fall areas, whereas, little settlement will take place over the gob areas.

The type of roof control will influence when that particular area of the mine will collapse. Bolted and propped passages will tend to fail sooner than those with cribs, steel supports and other types of bracing.

Pillar sizes, shapes and spacing are not uniform throughout a mine. The mains usually are protected by large rib pillars, whereas, in the panels the pillars are smaller in order to increase the extraction ratio. Large barrier pillars usually are left between panels. Some mining plans include pulling pillars in retreating from a panel. The percent of pillars pulled varies widely. In old, abandoned mines much pillar robbing took place which was not recorded on the mine maps. To add to the problem of trying to evaluate conditions in old mines, maps of these mines, if they exist, usually are not accurate. In some cases, the mining company did not employ a qualified surveyor, whereas, in other instances, the maps were deliberately falsified.

Both in old and present mines large areas may go unmined because of channel fills and cutouts or because of hazardous mining conditions. These unmined areas usually are very irregular shapes. Also large blocks of coal will be left in order to protect existing surface structures and transportation right-of-ways from subsidence. In old mines many of these protective pillars are too small and will tend to accentuate differential settlement at the surface.

Inflowing ground water most frequently causes more rapid deterioration of a mine. Exploration drilling has demonstrated many abandoned mines are partially or completely filled with water. In some cases, only portions of the mine contain water. Some people feel pumping of water from abandoned mines will cause subsidence because the water supports or holds up the roof. A cavity filled with water will not support the roof above it. The only way the author can visualize pumping of water from an abandoned mine will cause roof failure and subsequent subsidence is the action will allow more inflow of ground water from the roof. The seepage pressures from this flow could cause the weak roof materials to collapse.

## THE MECHANICS OF TRANSMITTING STRAIN TO THE SURFACE

In the proceeding sections the author has discussed broadly how failure in a coal mine can migrate to the surface by stoping, fracturing and downwarping of the overlying beds. Also discussed were the many factors that will influence the mode and time aspects of mine deterioration. As portions of a mine fail, the strain does not necessarily migrate upward continuously to the surface and settle out to completion. The larger

the area that fails the more likely complete subsidence will occur at the surface in one phase. Subsidence due to longwall mining bears out this supposition. Because of the large size of longwall panels and total extraction is achieved, subsidence occurs rapidly and will be complete. However, in room and pillar mining, except when large portions of a mine undergo a sequeeze, the failures will be more localized. These failures will reflect upward but are unlikely to continue all the way to the surface as one complete action.

As observed in the active mines, roof falls will stope upward and temporarily terminate on some distinct bedding plane or competent strata. If the fall is left unreinforced, it may remain relative stable for a period of time and then go through another period of active stoping or lateral migration. Then once again the fall will stabilize.

Large falls can destress the immediate area by transferring the load through an action often referred to as arching to adjacent areas of a mine. The concept of destressing an area by drilling or tunneling holes parallel to the main working face has been used successfully in many mines. Figure 5, is a sketch of an actual large roof fall and the adjacent rooms and pillars in an Illinois coal mine which illustrates how the immediate area surrounding the fall exhibited no signs of distress, whereas, further from the fall the roof, floor and pillars all were experiencing an unusual amount of instability. This illustrates how localized instability will undergo a period of activity and stabilize which in turn will cause additional loading by arching to nearby sections of mine. If the increased loading exceeds the strength of the roof, floor, or pillars; the mine will begin to collapse in these areas. The small localized failures can trigger larger areas of failure.

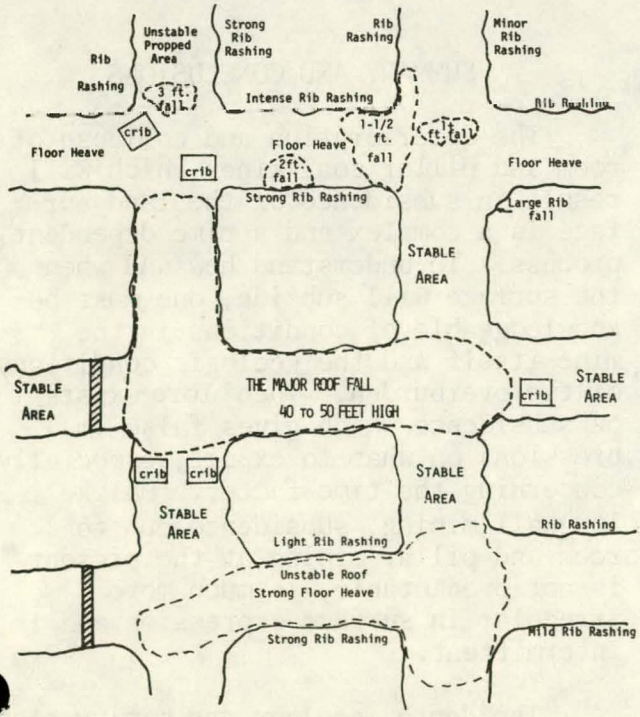



FIGURE 5 Sketch of Panel With Large Roof Fall And Conditions of Adjacent Areas

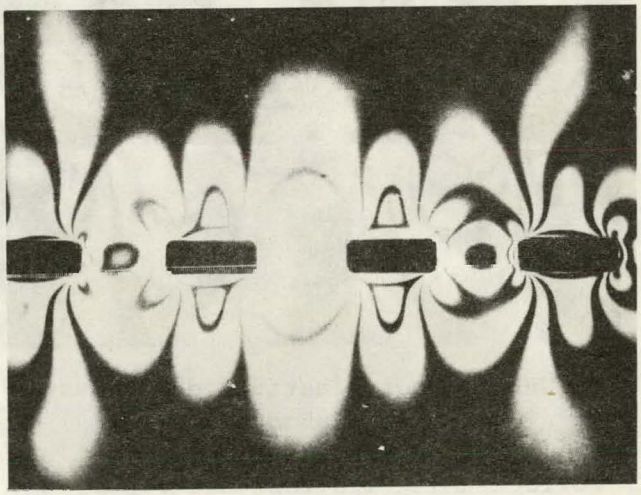


FIGURE 6 Photoelastic Model of Rooms and Pillars With No Failures

Figures 6, 7, and 8 are pictures of photoelastic models from a pilot study (2) which illustrates the principles discussed above. Figure 6, illustrates rooms (rectangular black areas) and pillars in which no failures have occurred. The shades of light and dark are stress distribution patterns. Without attempting to define photoelastic stress analysis, the relative symmetry of the patterns

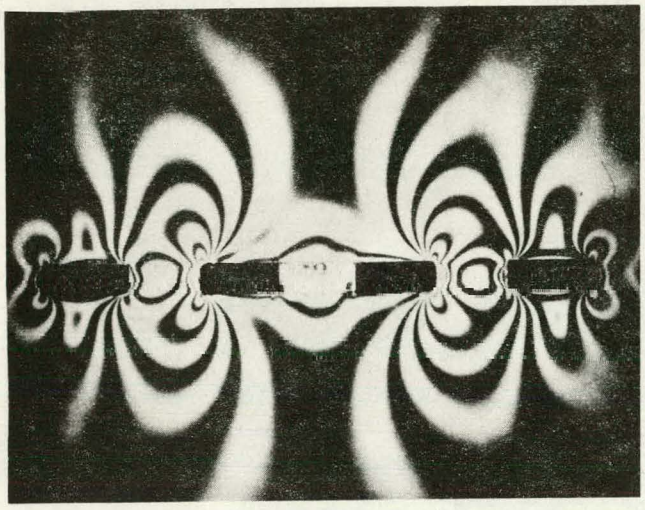


FIGURE 7 Photoelastic Model Illustration a Partially Failed Pillar

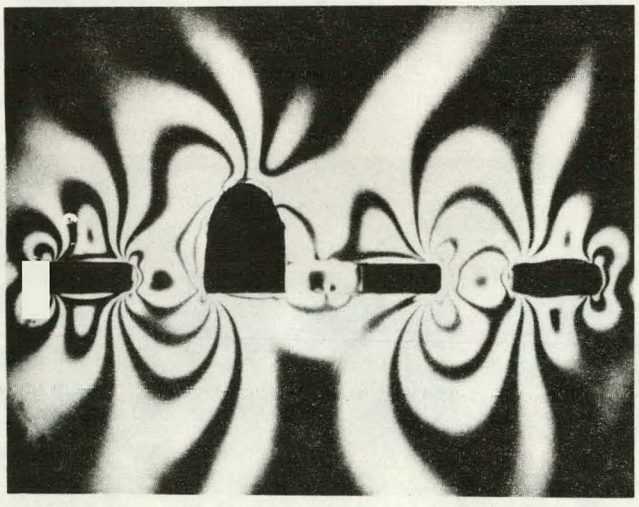


FIGURE 8 Photoelastic Model Illustrating Roof Fall

reflect uniform stress distribution across the roof and floor and through the pillars. Figure 7 is a model in which the center pillar has partially failed. Note the redistribution of stress to the two adjoining pillars. Essentially the span length has increased to include two rooms and a pillar. The extra loading on the adjoining pillars, floor and roof can trigger instability of these units. The case of a roof fall and stoping is illustrated in Figure 8. Once again there has been a redistribution of stress as reflected by the contorted patterns. These models illustrate how failure of the mine opening or pillar will cause a redistribution of the loading which in turn affects other parts of the mine.

Because of the nature and cyclic action of coal mine deterioration the surface above a mine may undergo more than one period of subsidence. People are lured into a feeling of false security that once subsidence has occurred at a site, it will not take place again. Until all the cavities have collapsed and closed in the mine that affect the site by the angle of draw, future subsidence can and will occur. Another unknown but important factor is the contribution to stability or instability due to bulking of the materials as they break.

Multiple subsidence of the surface has been documented. Interviews with property owners have revealed structures have been damaged by subsidence more than once with periods of no settlement in between. In Johnson City, Illinois, a school site has experienced at least two periods of subsidence. Exploratory drilling has revealed the mine below the property still has open voids.

## SUMMARY AND CONCLUSIONS

The deterioration and collapse of room and pillar coal mines which will result in subsidence of the land surface is a complex and a time dependent process. To understand how and when the surface will subside, one must be knowledgeable of conditions in the mine itself and the geologic conditions of the overburden. Much lore exists on subsidence which gives false impressions on what to expect, especially concerning the time factor. Unlike longwall mining, subsidence due to room and pillar mining at the present is not predictable, is much more irregular in surface expression and is intermittent.

The depth, geology and mining plan will influence when and how subsidence will occur in room and pillar mining. A given site may undergo more than one

period of subsidence. In abandoned mine areas, subsidence prediction is especially difficult because of lack of information on the mines. Land use above old mines should be carefully considered. Any development should be designed to cope with subsidence in the future.

## REFERENCES

1. National Coal Board, Subsidence Engineers Handbook, Section 5, London, Great Britian, p. 40., 1975.

2. Skudrzyk, F., Unpublished data, 1979.

# THE EFFECT OF GEOTECHNICAL FACTORS ON THE SUBSIDENCE RESPONSE

John N. Edl, Jr.

Mining Engineer

U.S. Department of Energy
Carbondale, Illinois

## INTRODUCTION

Subsidence has in the past, and continues to be, a source of serious problems in Illinois. The concern for subsidence problems in Illinois is demonstrated by the fact that Illinois is one of the few coal mining states where the legislature has passed legislation dealing directly with subsidence problems. This concern is also demonstrated by the subsidence investigations that have been conducted by state and federal agencies in Illinois since at least 1916 (1).

Many subsidence problems that occur could be alleviated or avoided if the subsidence phenomenon were more thoroughly and generally understood, and appropriate techniques for predicting subsidence ground movements were available. For example, a more suitable prediction technique would allow more informed planning of both; mining operations in areas where surface uses are significantly affected by subsidence, and land use planning and surface construction in areas likely to be affected by mining. Subsidence prediction techniques have been developed in most European countries where coal mining operations have been conducted. For example, an empirical subsidence prediction technique developed by the National Coal Board (NCB) has been successfully used as a basis for a wide range of subsidence engineering activities in the coal mining regions of Great Britain. Based, in part, on the pioneering efforts of European subsidence engineers, work is currently proceeding under USBM and DOE auspices to investigate the subsidence phenomenon and to develop subsidence prediction techniques applicable to the coal mining operations in this country. The study of subsidence and the development of suitable subsidence prediction techniques can be aided by an appreciation of how various geotechnical factors can affect the subsidence response.

The purpose of this paper is to discuss the effects of selected geotechnical factors on subsidence. The information presented necessarily relies heavily on subsidence experience from European countries as subsidence research is still in the early stages in the U.S.

## SUBSIDENCE RESPONSE

To describe how the various geotechnical factors affect the subsidence phenomenon it is first useful to discuss the various parameters that

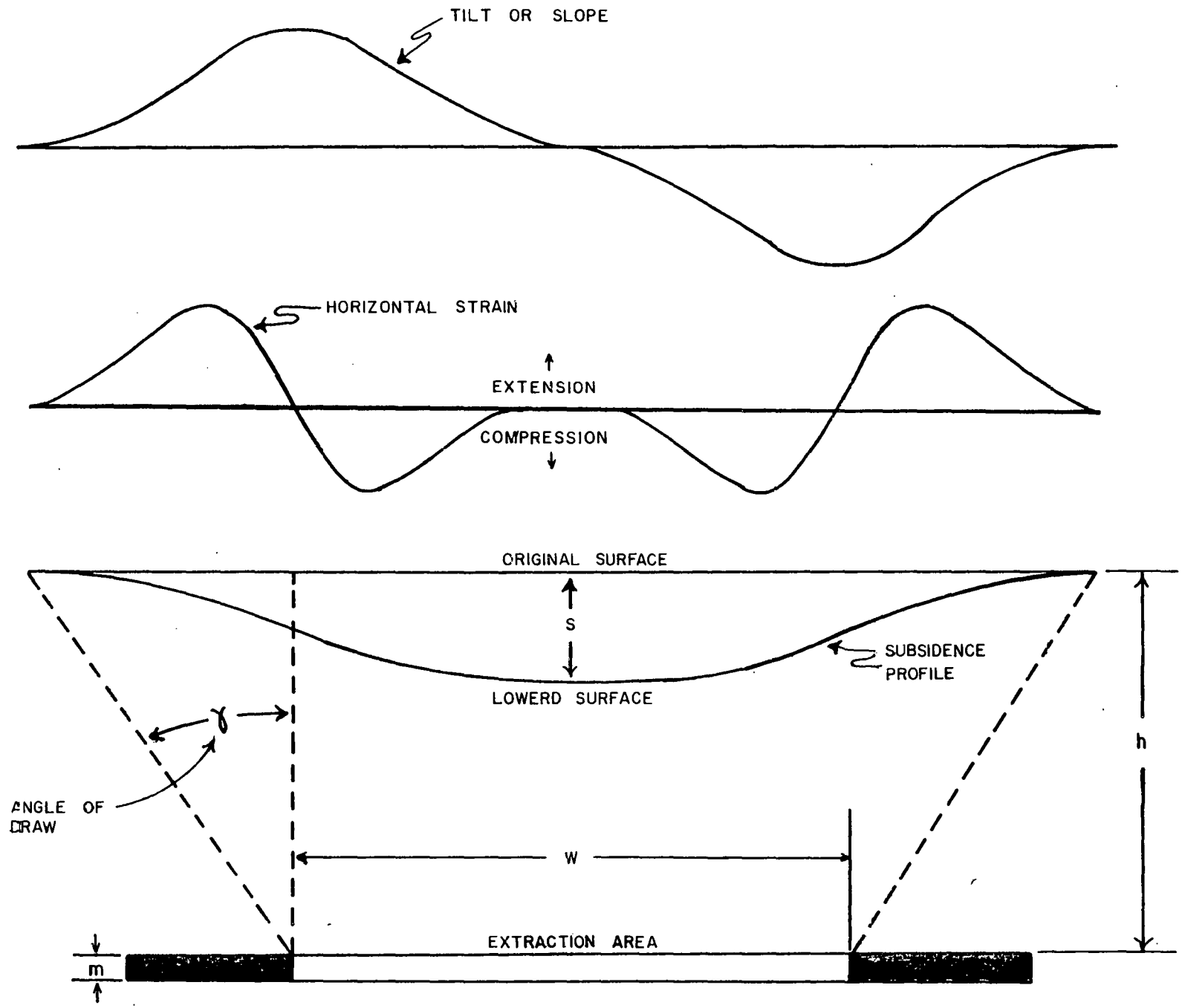
are used to describe subsidence ground ovements and present some ideas on how hese parameters can affect surface structures and surface land uses. The subsidence parameters that are most useful for describing the impact of subsidence are: vertical displacements or subsidence, slope or tilt, and horizontal strain. Typically the vertical displacements, that occur as the result of the extraction of coal within an area in the coal seam, form a trough on the surface somewhat larger in areal extent than the extraction area (Fig. 1). The trough structure results in differential vertical displacements which produce tilts and horizontal strains. Uniform vertical displacements will generally not have significant impact, or even be noticed, except in areas where water occurs on or near the surface. In these areas vertical displacements can result in flooding and disruption in the normal flow of water. Tilt can result in damage to selected structures, for example, tall structures such as smokestacks could be severely damaged if subjected to significant tilts. Tilt movements can also locally alter the gradients of such things as sewer lines or waterways which can clearly have severe consequences. The subsidence parameter that usually has the most severe impact on most man-made structures is horizontal strain. Since both geologic materials and most man-made structures are much weaker in tension than in compression, the zones where tensile horizontal strains are developed are the areas where the most severe structural damage and noticeable affects of subsidence usually occur.

## GEOMETRICAL GEOTECHNICAL FACTORS

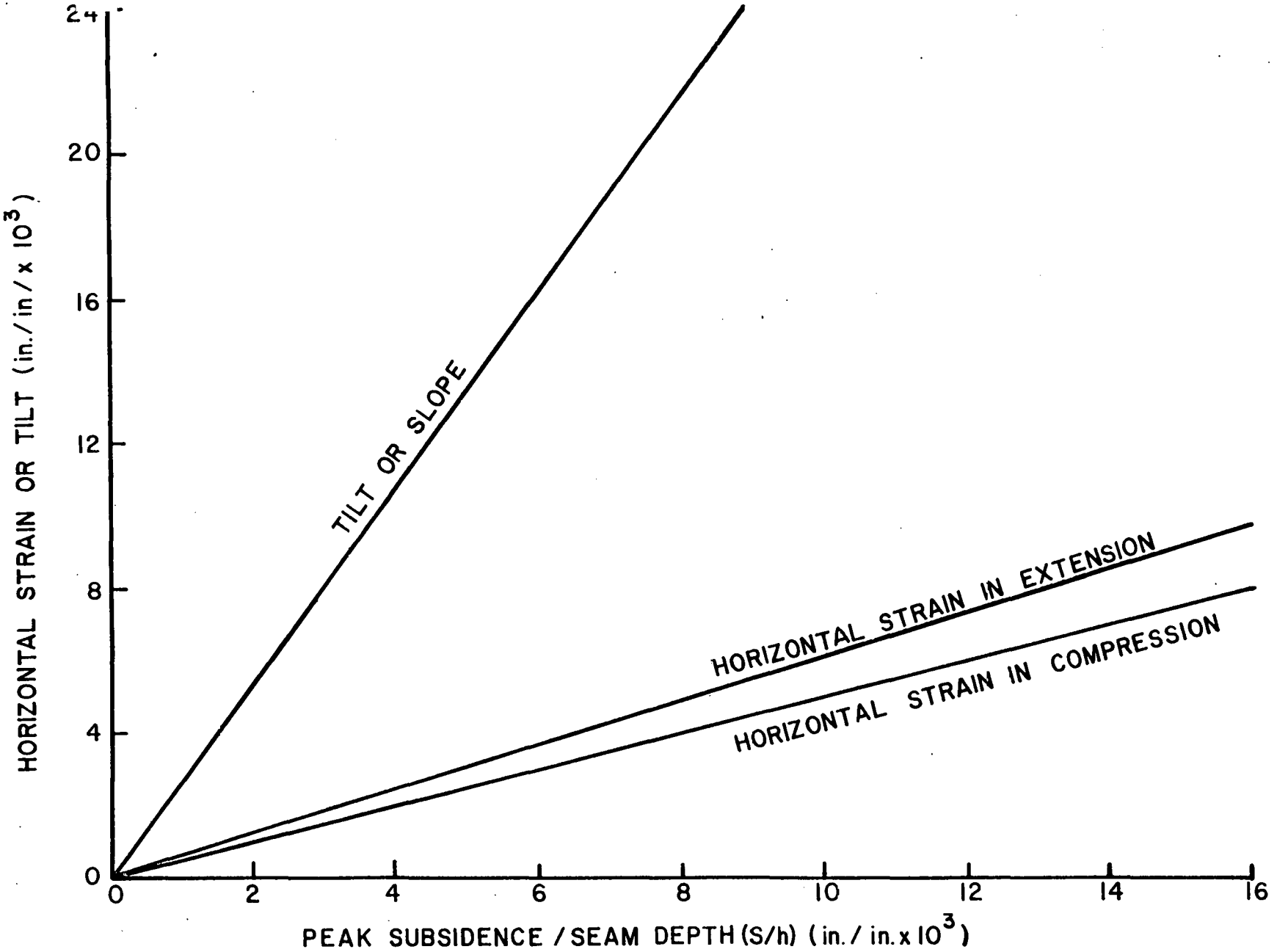
The subsidence effects of the most general and significant geotechnical factors have been determined from previous investigations and are incorporated into the subsidence prediction techniques developed in other countries. The typical approach used was to relate these geotechnical factors to the subsidence parameters used to describe the response (2). Two of the more important subsidence parameters from a structural damage point of view are tilt and horizontal strain. For both of these parameters the S/h (peak subsidence/seam depth) ratio is a very important geotechnical factor. European experience indicates that, for a given set of circumstances (constant width to depth ratio), both the horizontal strain and tilt are directly proportional to the S/h ratio (Fig. 2). Clearly, both the peak subsidence (S) and the seam depth (h) are important geotechnical factors when considering the subsidence response. The effective thickness of the extracted seam (m) is also an important geotechnical factor. The magnitude of the peak subsidence is directly proportional to the effective excavation thickness, which in turn is determined by both the mining thickness and the extraction ratio. Another geotechnical factor that directly affects the magnitude of the peak subsidence, hence the whole subsidence response, is the w/h (excavation width/seam depth) ratio. From Fig. 3 (which is a widely published curve developed by R. J. Orchard for the National Coal Board) (3) it is seen that the excavation width gradually increases from zero to something like 1.5 of the seam depth, the magnitude of the peak subsidence (hence the tilt and horizontal strain) increase from zero to the maximum value. These geometrical geotechnical factors of m, S/h, and w/h have the most significant impact on the subsidence response and are incorporated into the empirical subsidence prediction technique developed by the NCB (2). While the impact of the geometrical factors on the subsidence response is well understood and has been extensively examined in the literature, it is useful to include a discussion of these factors to assist in understanding the manner that the other geotechnical factors influence the subsidence response.

## SURFACE GEOLOGY

An important geotechnical factor

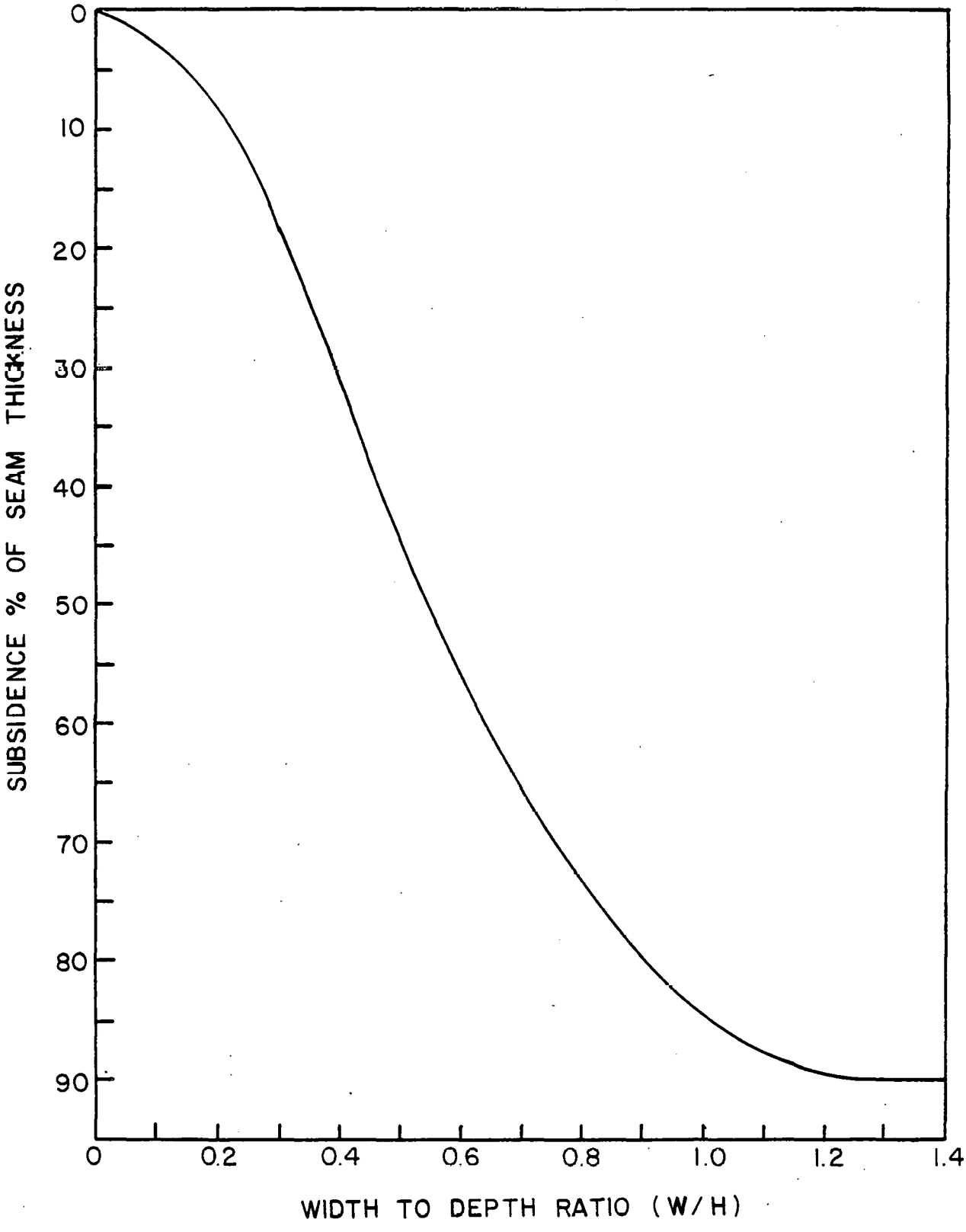



ig. I CROSS-PANEL SUBSIDENCE, HORIZONTAL STRAIN, AND TILT PROFILES




THE RELATIONSHIP BETEEN THE S/h RATIO AND HORIZONTAL STRAIN OR TILT FOR W/h-1.4

Fig. 2




RELATIONSHIP OF SUBSIDENCE TO THE WIDTH/DEPTH OF WORKING

Fig. 3

that is usually not explicitly accounted for in most subsidence prediction techniques is the geologic nature of the upper zone of the overburden, i.e. the overburden material near the surface. As a starting point or reference condition, the subsidence response when the overburden is made up exclusively of coal measure rock will be used as a basis for comparison purposes. The subsidence response is particularly well behaved when the upper zone of the overburden consists of coals, mudstones, shales and etc. which make up the coal measure rock strata and which tend to deform plastically and accommodate the ground movements that result from mining. For this case the subsidence response closely approximates the textbook description of the subsidence phenomenon in that the subsidence parameters (particularly the horizontal strain) vary in a continuous manner across the excavated areas (3). In contrast, when brittle layers of sandstone and/or limestone make up the upper regions of the overburden, the subsidence response is not so well behaved. In these cases fissuring and crack formation on the surface is much more likely, particularly if the mine is not very deep. This brittle layer tends to fracture locally, breaking the initially continuous layer into a series of essentially independent blocks. Cracks or fissures then form on the surface at points where adjacent blocks come together. As a result, the horizontal strain profile across any excavation made under an area where thick brittle layers are located near the surface may be very discontinuous (Fig. 4) (3) (4) (5) and any surface structure spanning these fissures is likely to be severely damaged.

On the other extreme, when a significant portion of the upper zone of the overburden is made up of unconsolidated material, for example in an alluvial valley or in an area where a thick layer of glacial till has been deposited, the subsidence response again differs from the base case of an exclusively coal measure overburden. In this case the magnitude of the maximum vertical displacements (subsidence) may be significantly less than for the base case and the angle of draw may be much greater (Fig. 5) (5) (6). In other words, the surface area affected may be much greater for unconsolidated overburden than for coal measure strata. A direct consequence of this behavior is that, in addition to shallower subsidence, both the slope and horizontal strain may be significantly less for this case than when the overburden consists exclusively of coal measure strata.

The next item to be discussed will be the effects of shallow mining (depths less than 300 ft.). As a general rule the subsidence problems that result from shallow mining are of a rather severe nature. The severe nature of the subsidence problems is a natural consequence of the severe distortions in the overburden that are necessary to accommodate the mining activity.

## SHALLOW MINING

Longwalls mined at shallow depths generally exhibit one of two different forms of subsidence behavior. These two forms have been descriptively termed block collapse and conventional behavior respectively. When the block collapse form of longwall subsidence behavior occurs all the overburden material over the excavated area: fails, breaks up, and collapses into the excavation, essentially forming a large sinkhole. On the surface the zone of failure is outlined by a step-like boundary that occurs somewhere near the panel edge and defines the boundary between solid and failed material. In contrast, for conventional subsidence behavior there is a relatively smooth transition from subsided to undisturbed area on the surface. Although the transition is relatively smooth there are still rather severe surface distortions associated with shallow workings. As indicated in Fig. 2 both the tilt and the horizontal

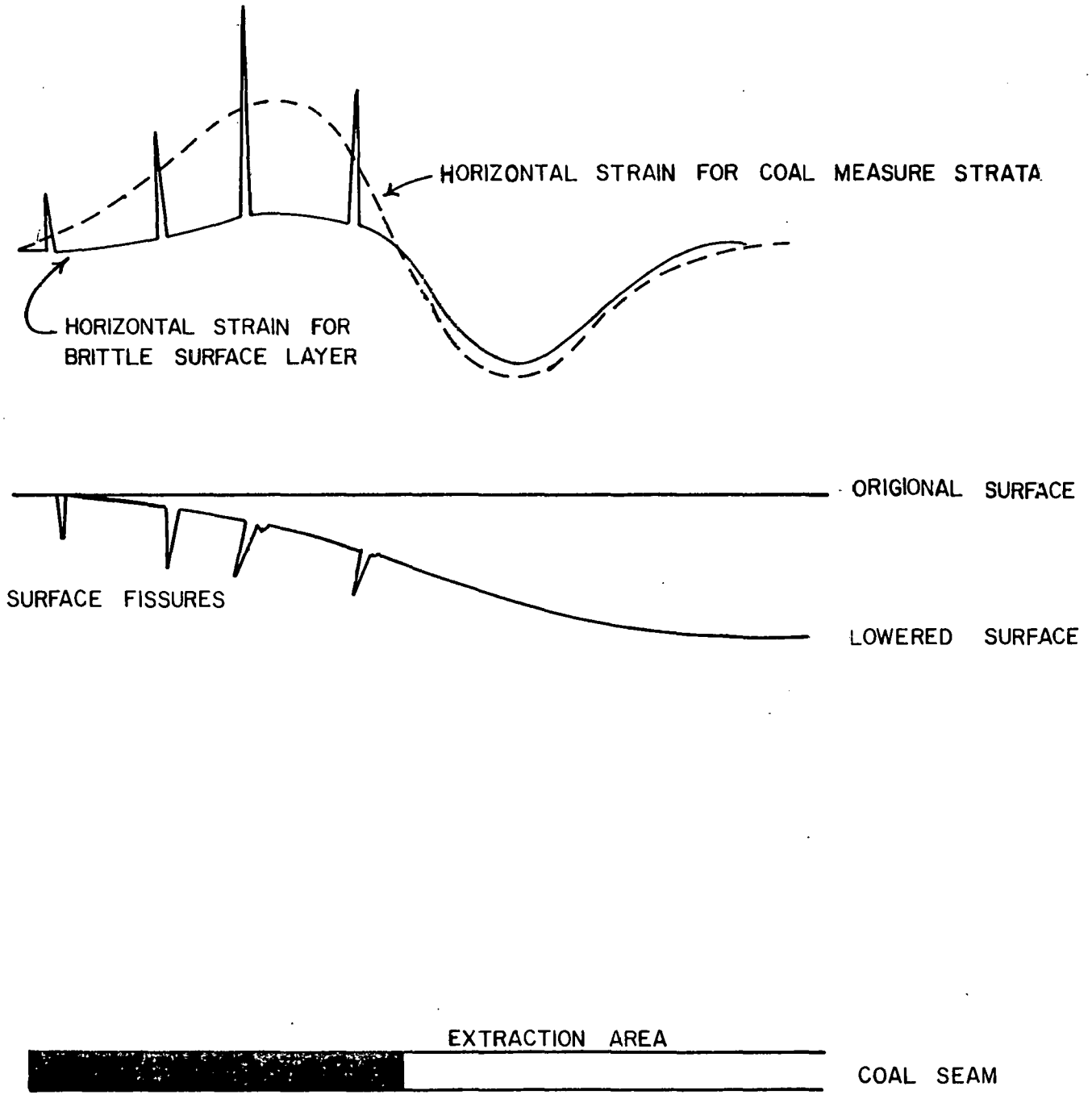



Fig. 4 SUBSIDENCE AND HORIZONTAL STRAIN FOR BRITTLE SURFACE LAYER

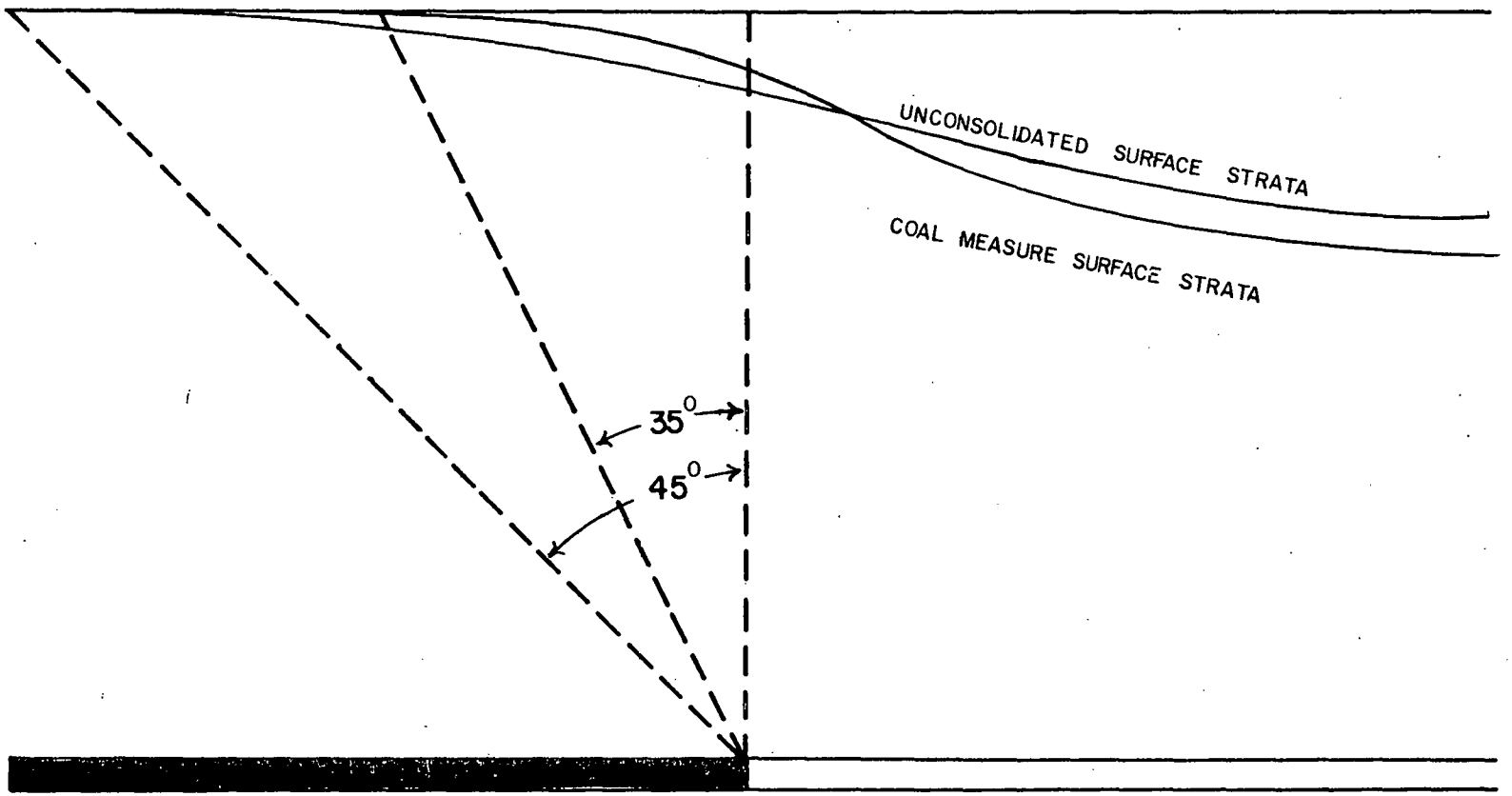



Fig.5 A COMPARASION OF SUBSIDENCE PROFILES FOR UNCONSOLIDATED vs COAL MEASURE SURFACE STRATA

strain are directly proportional to the S/h (peak subsidence/seam depth) ratio. For shallow depths this ratio is high as the depth is limited by definition, and the peak subsidence is essentially independent of depth. Accordingly the slope and horizontal strain are also corresondingly greater for shallow depths than is the case for deeper excavations. From Fig. 6, which is a plot of S/h ratio vs seam depth assuming S=5ft. and w/h=1.4 for all seam depths,it can be seen that as the depth approaches zero the S/h ratio, hence the horizontzal strains, approach infinity. A comparison of the subsidence and horizontal strains for a wide excavation (greater than 2000 ft.) at 300 and 1500 ft. depths respectively is presented in Fig. 7 and Fig. 8. These diagrams indicate that the horizontal strain is much greater for the shallow case than for the deeper case. The reason for this behavior is that for the shallow case the vertical displacements go from zero to full subsidence in a much narrower span than for the deeper seam. As a result the surface is subjected to more severe distortions for the shallow case than the deep case, this distortion in turn, results in the relatively high horizontal strains for the shallow case. As a result shallow longwall mining often causes fissuring and severe slopes on the surface, even when the subsidence response is essentially conventional. In any case both forms of subsidence behavior result in a subsidence response that is on the severe end of the response spectrum.

The geotechnical factors that tend to favor the block collapse subsidence mechanism include: high m/h (seam thickness/seam depth) ratio, unconsolidated overburden, and shallow depths. Conversely when the m/h ratio is low, the overburden consists of plastic coal measure strata, and the seam is comparatively deep, the probability that conventional behavior will occur is enhanced.

The formation of surface sinkholes, which are very severe local subsidence disturbances, is a fairly common result of shallow room and pillar mining operations in Illinois. These sinkholes are usually the result of roof failure over a relatively narrow mine opening. Over time, the roof rock gradually will weather and unravel and, in essence, the void migrates toward the surface. Once initiated this void migration may either; reach the surface to form a sinkhole, be arrested by a competent rock layer, or be arrested as the result of bulking of the failed material. As a result of analytical computations (7) it has been estimated that on the average, bulking should arrest migration at a point 3 to 5 times the seam thickness above the seam, and something like 10 times the seam thickness would be the maximum possible height to which this caving can occur. Thus, on the average, sinkholes should not form for seams greater than 5 times the seam thickness deep and seams greater than 10 times the seam thickness deep should be immune from sinkhole formation. For example, if mining is conducted in a 6 ft. thick seam, sinkholes should not form if the seam is more than 60 ft. deep.

## OTHER GEOLOGICAL FACTORS

The presence of faults and strong strata within the overburden are two other geological factors that can strongly affect the subsidence response. The presence of a fault within the overburden is often indicated by the formation of a step or discontinuity in the vertical displacements of the surface as indicated in Fig. 9. Indeed one means of determining the outcrop location of a fault, in an area subjected to subsidence ground movements, it to determine where the step forms on the surface. Although a thorough study of faulting effects has not been undertaken, there are several general trends in fault affected ground movements that have been observed (8) (4). In particular: downward step movement will occur on the mining side of the fault, the surface region near the edge of the

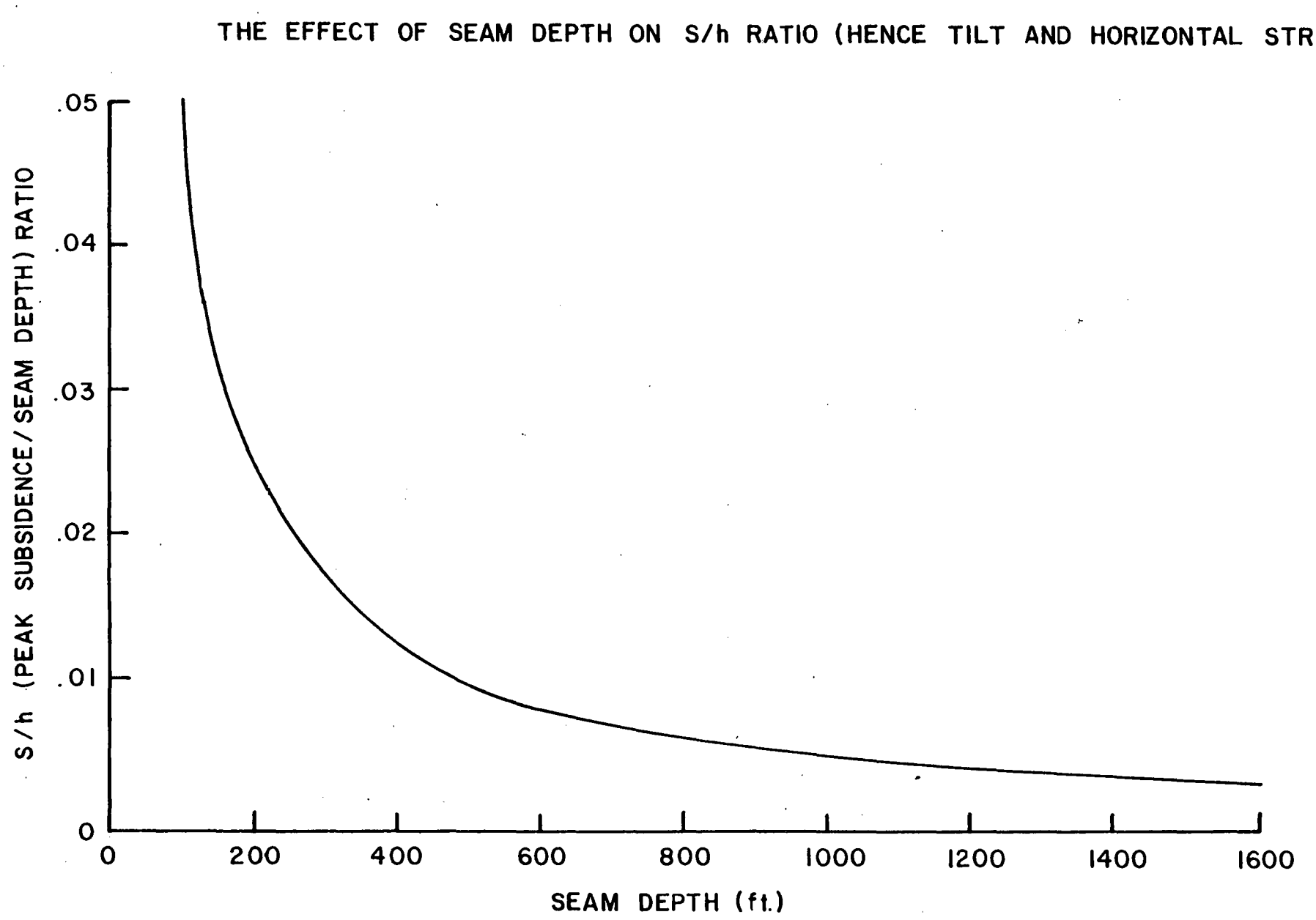
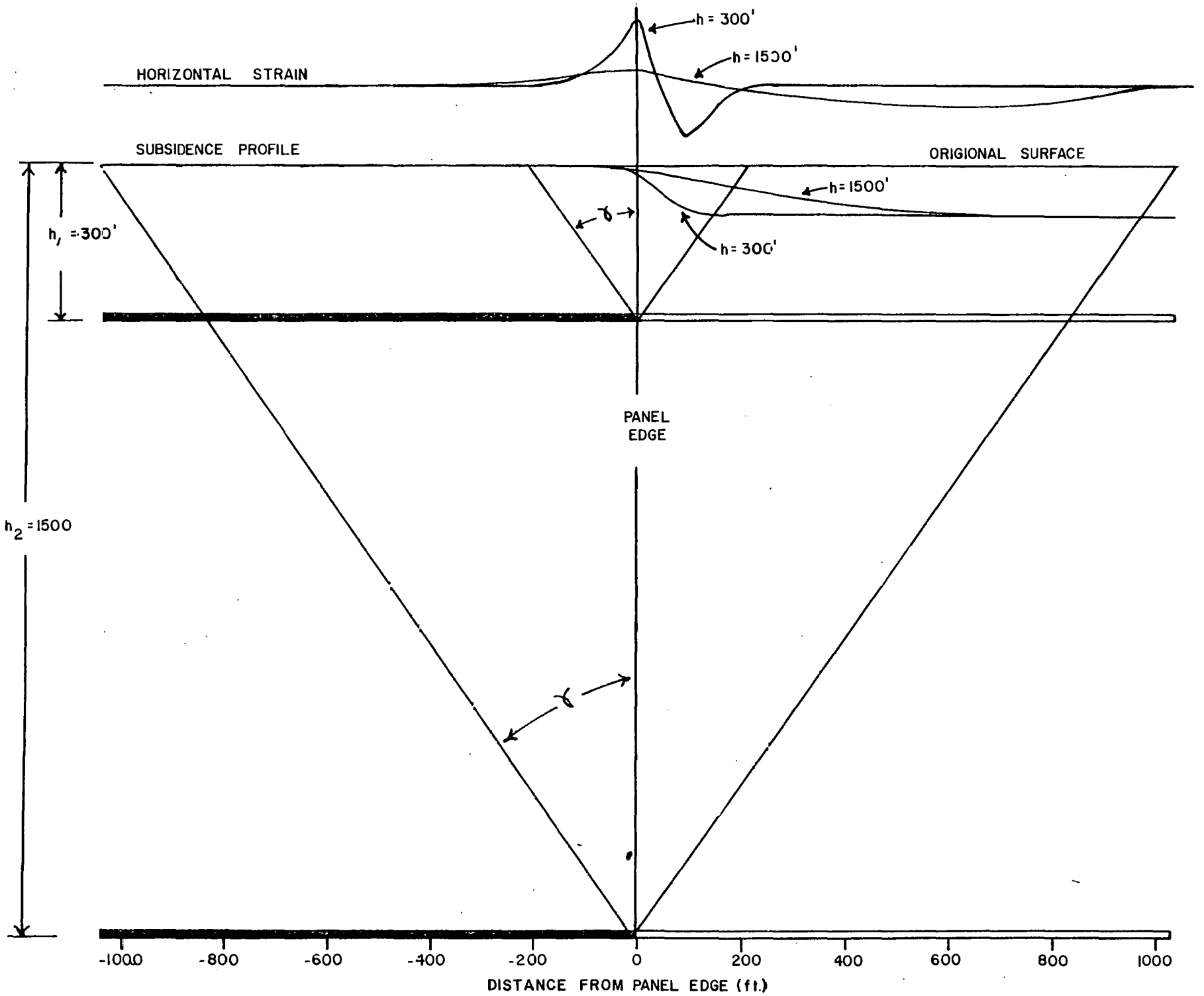

THE EFFECT OF SEAM DEPTH ON S/h RATIO (HENCE TILT AND HORIZONTAL STRAIN)
S/h (PEAK SUBSIDENCE/SEAM DEPTH) RATIO
.05
.04
.03
.02
.01
0
0
200
400
600
800
1000
1200
1400
1600
SEAM DEPTH (ft.)

Fig. 6


h = 300'
h = 1500'
HORIZONTAL STRAIN
SUBSIDENCE PROFILE
ORIGIONAL SURFACE
h = 1500'
h = 300'
h1 = 300'
PANEL EDGE
h2 = 1500
-1000
-800
-600
-400
-200
0
200
400
600
800
1000
DISTANCE FROM PANEL EDGE (ft.)

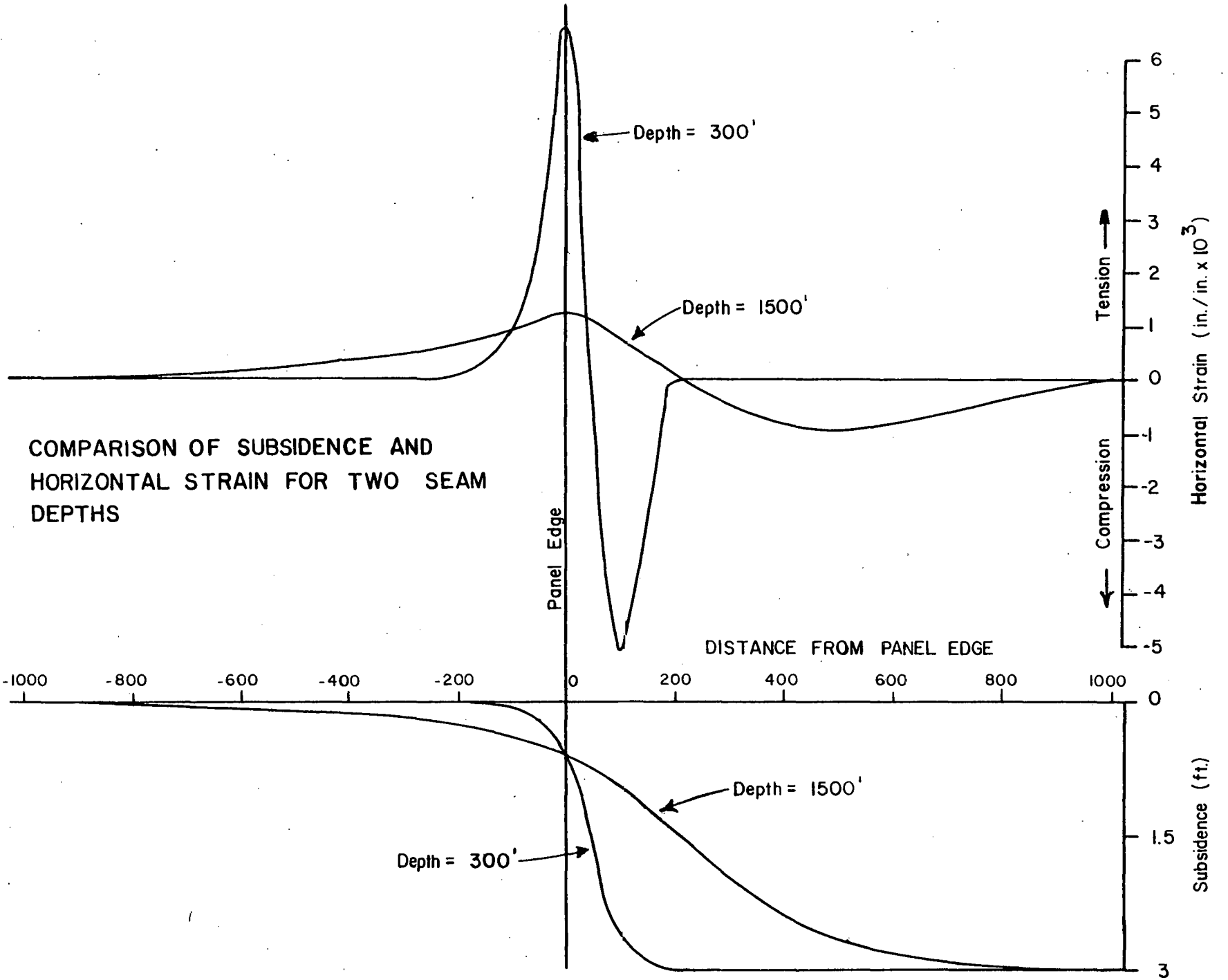

COMPARISON OF SUBSIDENCE AND HORIZONTAL STRAIN FOR TWO SEAM DEPTHS
Depth = 300'
Depth = 1500'
Panel Edge
DISTANCE FROM PANEL EDGE
-1000
-800
-600
-400
-200
0
200
400
600
800
1000
Tension
Compression
Horizontal Strain (in./in. x 10^3)
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
Depth = 1500'
Depth = 300'
Subsidence (ft.)
0
1.5
3

Fig. 8

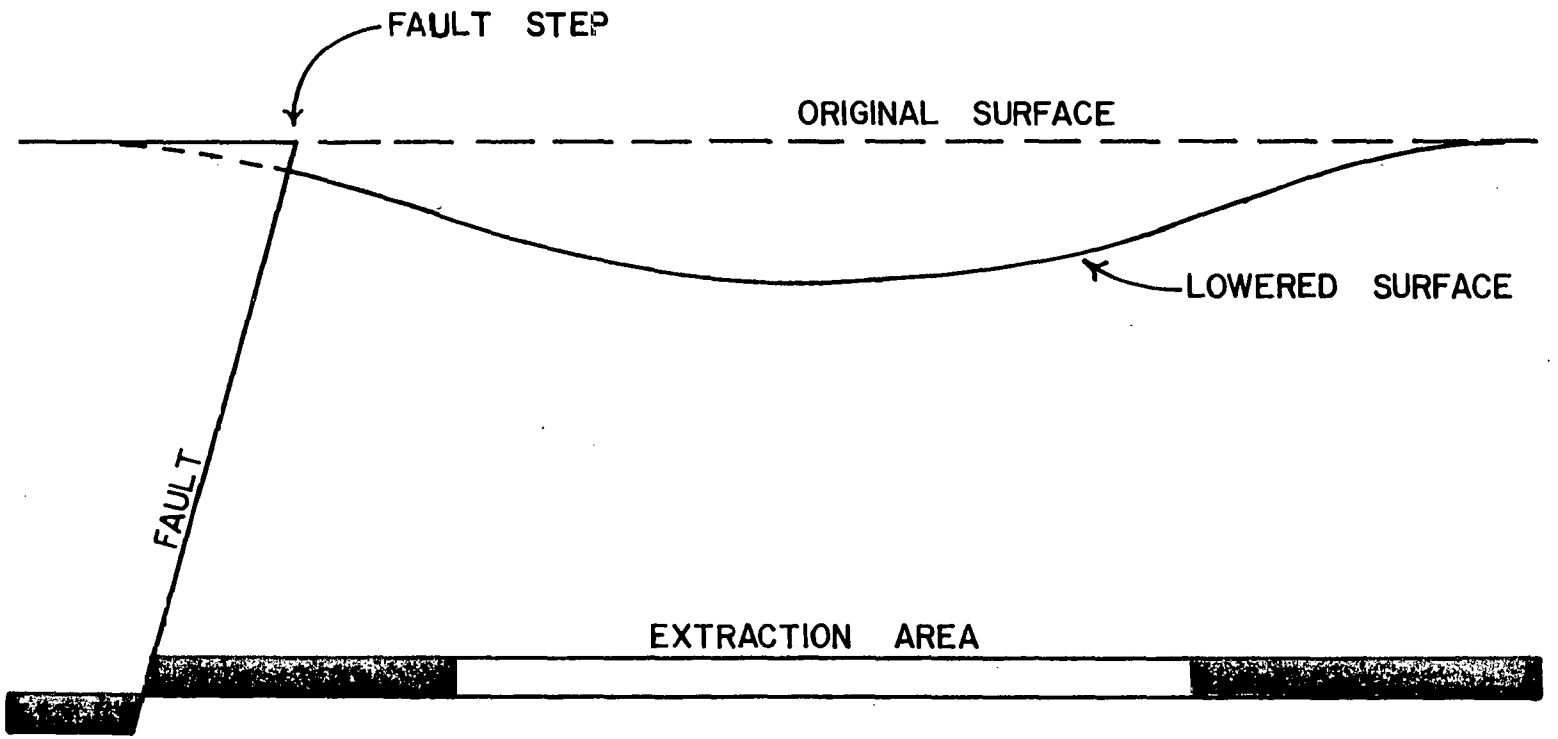



Fig. 9 SURFACE STEP AT FAULT OUTCROP

excavation is the most likely location for the step position, the size of the step is likely to be 30% of the peak subsidence or less, and previous mining in an underlying or overlying seam will promote and exaggerate step movement. The other factor that is pertinent in determining the effect of the fault is the location of the mine workings in relation to the fault. For example, excavating a wide panel parallel to the fault is more likely to result in the characteristic step than mining a narrow panel perpendicular to the fault. Also, mining under the fault plane, or approaching the fault from this direction (as indicated in Fig.9), is more likely to result in a step than mining on the other side of the fault plane.

Clearly if a structure is located over the outcrop of a fault and the step occurs, the structure is likely to be severely damaged. Such a step is likely to occur when the fault is located near the edge of a wide panel mined parallel to the fault plane. To minimize the liklihood that the step will form, narrow panels should be mined perpendicular to the fault.

The general effect of a thick strong layer within the overburden is to inhibit the downward vertical displacements, particularly over narrow excavations (4) (5). This strong strata has a tendency to bridge over narrow panels which will limit the amount of subsidence that occurs in this case. One adverse effect of this bridging is that the solid seam immediately adjacent to the excavation will be subjected to excessive loads. However, when the excavation is sufficiently wide it is likely that even a very strong layer will break and the subsidence response will be nearly normal.

## SUMMARY

The subsidence response for an excavation mined under an overburden consisting of exclusively coal measure strata is well behaved and is somewhat predictable, providing the geometrical geotechnical factors of seam thickness (m), seam depth (h), and excavation width (w) have been determined. There are, however, several additional geotechnical factors that can affect the subsidence response and these factors should be considered when predicting subsidence and/or characterizing the subsidence response. Accordingly, to assist in interpreting the subsidence response, it is important to conduct some sort of geotechnical investigations to determine whether or not any of the pertinent geotechnical factors are present.

For example, the subsidence response is very strongly affected by the make-up of the overburden, particularly the composition of the surface strata. A thick strong layer located near the surface, subjected to typical subsidence movements is likely to fissure and crack producing discontinuities in horizontal strain. In contrast, a thick layer of unconsolidated material on the surface will reduce the magnitudes of vertical displacements and horizontal strains compared to the typical coal measure case. The composition of the overburden is similarly important when considering the subsidence response for shallow mining. The formation of sinkhole-like depressions is more likely to occur for both longwall and room and pillar mining if the overburden is made up of predominantly unconsolidated material. A more conventional subsidence response can be expected if the overburden consists primarily of coal measure strata.

Another example of an overburden characteristic that can strongly affect the subsidence response is the presence of a thick layer within the overburden. A thick strong layer within the overburden will bridge over and limit the vertical displacements, hence horizontal strain and tilt, for narrow excavations. This bridging may be desirable in shallow room and pillar mining areas as sinkhole formation is inhibited by the stronger layer. On the other hand, for deeper excavations,

this bridging can cause undesirable high abutment loads in the adjacent solid coal. For wider excavations it is likely that even a thick layer will break and the subsidence response will be nearly normal.

The subsidence response can also be affected by faults located within the overburden. The location and orientation of the mine workings with respect to the fault plane will have a significant impact on the effect that faults can be expected to have on the subsidence response. Clearly it is important to conduct sufficient geotechnical investigations to determine the presence or absence of these features, prior to attempting to predict subsidence and/or characterize the subsidence response, as the form of the subsidence response is strongly affected by the various geotechnical factors previously described.

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# SUBSIDENCE - PREVENTION OR CONTROL

by
C. D. Breeds M. Karmis C. Haycocks

Assistant Professors and Professor
Department of Mining and Minerals Engineering
Virginia Polytechnic Institute and State University
Blacksburg, Virginia

1. Introduction - Rep. Paul Simon D-Ill., and Rep. Morris Udall, D-Ariz. have suggested that as much as $30 million needs to be appropriated annually to pay for subsidence damage. The first reaction, and one method suggested by the G.A.O. (General Accounting Office) is to prevent subsidence, at a projected cost, when backfilling, of $22,000/acre (G.A.O), $32,000/acre (Ref. 6) and $58,806/acre (The independent Coal Operator); an economically unacceptable solution.

This paper will examine an alternative solution readily accepted in other foreigh coalfields. That is the concept of control, or accomodation of subsidence movements.

Both underground and surface methods of lessening the effects of mining subsidence will be discussed. In addition, two case studies will be presented to illustrate the success of one surface and one underground method. The case studies were carried out as part of sponsored research with the National Coal Board (U.K.)

2. Subsidence Over Longwall Workings - Factors which affect the severity of mining induced structural damage may be grouped into three categories:

a. Mining Factor(s)
b. Site Factor(s)
c. Structure Factor(s)

2.1 Mining Factors - Surface subsidence due purely to extraction processes and dimensions is a function of a) depth and, b) width of workings, c) extracted section, d) seam inclination, e) method of support, f) rate of advance, g) position of old workings, and h) the angle of draw.

The basis of predicting the combined effect of these parameters was first established by Orchard (1). By analyzing large quantities of field data he showed that a relationship existed between subsidence expressed as a percentage of the extracted seam thickness (S/m) and the dimensionless width/depth ratio (W/h). Table 1 shows the effect of this dimensionless ratio on vertical subsidence, strain and tilt.

The damage forming aspects of subsidence are more often associated with tilt, curvature and strain rather than pure vertical displacement. Tilt (differential subsidence) and curvature (differential tilt) are a direct consequence of the shape of the subsidence trough while strain has two components, (1) that due to curvature and (2) that due to the lateral migration of the surface towards the center of the excavation. The dual nature of strain is not generally acknowledged, and as prediction methods are based on direct field measurements of spatial displacement, the effect of curvature is only singled out where it may cause adverse deflections in foundations.

The orthodox movements described

above develop within the coal measure strata and are transmitted with varying amplitude through the overlying strata. Geotechnical conditions associated with this upper crust give rise to anomalous movements, which in broad terms are due to site factors.

2.2 Site Factors - In 1968, Shadbolt and Mabe (2) gave a qualitative review of the role of the site factor and suggested that anomalies were due to variations in:

(a) Soil Properties
(b) Rock Properties
(c) Hydrogeology
and (d) Geological Discontinuities

A rigorous discussion of these parameters has been given by Shadbolt et al (3) who conclude, "This paper has demonstrated that the phenomenon of ground movement due to mining subsidence can be accurately predicted. In those cases where anomalies arise, their behavior can be explained providing the surface geology is accurately known and properly interpreted." Thus the importance of interpreting the effects of geotechnical influences during pre-calculation cannot be too highly stressed. A quantitative assessment of the effect of surface geology on the magnitude of mining subsidence has been presented by Whittaker and Breeds (4) They stated that differences in subsidence are not conclusive, but maximum surface tensional ground strain values have been found to be higher than with the other rock types. Conversely no discernible differences in maximum compressive strain have been observed for the three rock types. Thus in a majority of cases it is the tensile strain component which provides anomalous or abnormal damage.

Abnormal damage may also occur when a site has been subjected to mining induced displacements prior to construction. In such a situation, the magnitude of horizontal displacement and hence surface damage may double. Breeds (5) has described a case study where a large kilm was built on ground which had been subjected to 1.22 mm/m of extension. Later workings subjected the ground to 1.6 mm/m of compression causing extensive, abnormal danage to the structure. This resulted from the ground returning first to its original length and then to a state of compression.

2.3 Structure Factors - The response of surface structures to the ground movements previously described varies according to (a) size and shape, (b) design of foundations, (c) type of super structure, (d) method of construction and quality of materials and (e) age and standard of maintenance and repair. The accurate prediction of surface damage and associated cost requires a careful analysis of all these factors in addition to those grouped under 'mining' and 'site'. Once an evaluation has been made, alternatives must be considered. Broadly speaking these concern either prevention or control.

3. Methods of Prevention - In short, subsidence can only be prevented by leaving the coal or mineral in place. Stowing methods which it is claimed 'prevent' subsidence are only instrumented in decreasing the level of surface damage when used in conjunction with longwall mining. In cases where old room and pillar workings collapse due to the interactive degrading effect of air and water penetration, backfilling has provided permanent support, but at a cost far exceeding that gained from the coal removed. It is the contention of this paper that a better, more economical solution can be devised from applying methods of control.

4. Methods of Control

4.1 Underground Methods

a) Coal Pillars - Coal pillars may be left for permanent support of structures of high economic or socio-political value. Figure 1 gives some indication of the coal which is lost in protecting a structure 100 feet long and 50 feet wide. In addition to lost revenue, this method of structure protection may cause disruption of the underground working environment and concentration of stresses around pillar edges.

A more acceptable approach to t problem may be obtained from methods or partial extraction.

b) Partial Extraction Methods - hortwall and Pillar - This method of extraction incorporates short longwall panels (¼ depth wide) separated by ¼ depth pillars. Subsidence effects are reduced, by this mining method, to give:

(i) a shallow, relatively flat bottomed trough of subsidence

(ii) associated strains of less than ± 0.5 mm/m

(iii) associated slopes of less than 1.0 mm/m

Generally, this system of extraction yielding up to 60% of the reserve is successful in minimizing surface damage, but is wasteful of reserves. There is also one drawback to the system. This occurs when a partial extraction system is itself undermined and involves the phenomenon termed interaction. Strains which would normally peak over virgin support edges migrate in the direction of the old buttresses resulting in unpredictable-abnormal damage at the surface. This phenomenon has also been recorded with respect to room and pillar mining.

Room and Pillar - For many years it was considered that the room and pillar mining method 'prevented' subsidence. Only recently has the catastrophic nature of subsidence over room and pillar mines become apparent resulting in expensive remedial measures.

The factors which affect abandoned room and pillar workings leading ultimately to subsidence include:

(i) increased cover leads, due to population expansion cause increases in pillar stresses

(ii) new workings cause a redistribution of strata pressures initiating pillar collapse

(iii) a soft floor promotes subsidence as pillar dig in

(iv) air slack and moisture penetration degrede pillars, increasing roof spans leading ultimately to failure.

Solid Stowing - Many authors have noted successful application of solid stowing methods, Aynsley and Hewitt (7), Marr (8), Whetton (9), Schulte (10) and Brauner (11). Generally, when solid stowing is applied in conjunction with longwall mining subsidence and related effects are reduced by up to 50%. Unfortunately little data is available regarding the cost of solid stowing systems.

Mine Layout - There have been three methods suggested for subsidence control. Perhaps the easiest concept is the correct positioning of longwall panels in relation to surface facilities. Conditions will relate to permissible settlements within surface structures, necessitating analysis of the structure factor.

In addition to layout orientation, it is possible to design a mining system so that traveling strains are cancelled and tranverse strain migrate away from structure to be protected. This method is termed 'harmonic mining' and is illustrated in Figure 2.

4.2 Surface Methods - The transmission of movement from the surface strata to the foundation and hence to the structure is governed by properties of the surface strata on which the foundation is constructed and by the type of foundation used. Site prevention methods will be designed to comply with local conditions and predicted mining movements. The following is a brief summary of some methods at present used to accomodate movements.

Site Precautions - Trenching:

The objective of trenching is to lessen the effects of tension and compression on a structure, (i.e.) to create a weakness in the surface where strain may be dissipated. The effectiveness of the trench with depend on the following factors,

(i) depth of trench

(ii) properties of the surface strata

(iii) size and construction of foundation

(iv) weight of structure.

The depth of the trench will define the degree of isolation afforded to the piece of ground on which the structure is built; ideally, the deeper the better, although cost prohibits deep excavations.

Table 2 gives some examples where trenching has substantially reduced structural damage.

Structural Modifications -

Structural modifications may be

made where predicted damage costs are larger than the cost of precautionary works and remedial measures. The objective is to provide flexibility, and this may be achieved by,

(i) isolating sections of the building by cutting pathways and connecting corridors, walls, etc.

(ii) providing cuts or slots in girder ties to accomodate movements otherwise dissipated at points of weakness within the tram.

In each case, other 'safety precautions' must be used to protect services, personnel and equipment

(iii) flexible couplings inserted in pipes carrying gas, water, etc.

(iv) windows taped to prevent violent shattering

(v) supports placed beneath dangerous archways and entrances.

Special Construction - One idea is to build structures in mining areas that will not be damaged by mining subsidence. Such a large scale building system has been developed by the Nottingham County Council's Architects Department, (U.K). The system (C.L.A.S.P.) involves an extremely flexible structure founded on a 5 inch, reinforced raft foundation. The flexural strength of the foundation, and the flexibility of the superstructure allows the composite to accomodate abnormal subsidence movements. Case studies of its performance have been given by Swain (12) and Breeds (5).

5. Case Study I: A Large Industrial Complex

5.1 Mining Factor(s) - a) Past Mining - Permanent movements from previous workings had resulted in a line of extension in the Western part of the site as shown in Figure 3.

b) Present Workings - Figure 3 shows the layout of panels under the site. A system of partial extraction has been used, so that a minimum of disruption resulted in what is a heavily developed, industrial site. The mining dimensions and the maximum, related, subsidence parameters are summarized for each of the considered structures, in Table 3.

5.2 Site Factor(s) - The near surface strata are approximately 24 ft. of magnesian limestone which are underlain by 36 ft. of permian marl. Magnesian limestone has a well developed jointing system so that abnormal concentrations of strain leading to abnormal damage was anticipated.

5.3 Structure Factor(s) - a) Main Office Block (B 10)

This is a two story brick building set on a heavily reinforced 5 ft. deep, rigid foundation consisting of latticed spread footings. The roof is flat, of sandwich construction and presented the only anticipated problem insofar as drainage was concerned.

Machine Shop (C 2) - This structure was originally a hangar and consisted of steel framed, aluminum clad walls with multi-pitched aluminum clad rooms. The structure houses over 100 pieces of high precision machinery, ranging from simple lather to large raise bores.

Chemical Plants and Press Shop (C3)

The original construction was similar to the machine shops. However, the nature of facilities inside the structure provided some concern, namely:

(i) brick lined furnaces

(ii) vacuum presses and furnaces

(iii) chemical acid baths and tanks

5.4 Predicted Damage and Precautionary Works - The predicted ground movements were such that only minor inconveniences were anticipated for the major structures. These would include roof drainage difficulties and potential disruption of facilities in the chemical plant. A large fuel pipeline leading the the main runway T 14 was emptied, and isolated from the ground to prevent any loss of fuel.

5.5 The Investigation - The site gave an ideal opportunity to investigate the effects of a partial extraction scheme on a variety of fixed plant and structures. Several monitoring systems were set up, including:

(i) precise leveling of stations shown in Figure 3

(ii) steel taping between ground stations

(iii) direct tilt measurements using the B.A.C. (British Aircraft Corporation) electrolevel on over 100 large machines and test beds.

(iv) photo elastic stress plugs to measure deformations in the large

rigid foundation of structure B 10.

5.6 Results of the Survey - At he time of writing, T18's panel had finished, T 32's had worked 0.5 x depth past the complex and T 19's had advanced only 150 ft. Figure 4 shows the observed subsidence profile which developed over T 18's panel; a maximum amplitude of 5 inches is associated with a maximum tilt of 1/1000. Figure 5 shows values of subsidence and strain that developed in the ground around C 3.

The cumulative effect of T 18's and T 32's on the surface environment was negligible with B1, B4, B10, C2, C3 and their contents remaining unaffected. The success of this partial extraction scheme means that future sites of similar size and importance may be undermined, subjecting similar plant and machinery to the following values of subsidence, tilt and strain.

Maximum subsidence - 5 inches
Maximum tilt - 1/1000
Maximum strain - 0.025 - +0.025%

6. Case Study 2 - A Large Factory Complex - 6.1 Mining Factor(s) - a) Past Mining - Five seams had affected the area prior to 1960, and the effect of these was to leave the site in a state of extension. Thus, with the predicted movements from the new workings, abnormal compression effects were anticipated.

b) Present Workings - The predicted subsidence parameters and their evaluation with respect to damage are summarized in Table 4. Figure 6 shows the plan layout of workings.

6.2 Site Factor(s) - The structures are built on lower magnesian limestone which has a well developed jointing system. To the East of the site, the middle permian marl outcrops are separated from the surface limestone by a fault running North-West to South-East. Thus geotechnical influences dictated the extraction layout, providing subsidence parameters which suggested a severe level of damage to the structure.

6.3 Structure Factor(s) - The ain building is a single story structure occupying a ground area of some 62,00 sq. ft. The factory roof consisting of glazed northlights is supported by roof trusses which in turn rest on pillars spaced throughout the factory. The floor consists mainly of concrete and internal walls are brick varying from 3-12 inches in thickness.

6.4 Predicted Damage and Precautionary Methods - The main damage forming movements would be:

(i) traveling extension and compression

(ii) transverse permanent compression

Damage due to Traveling Movements - Calculations based on the mining factors showed that the factory would be stretched by 3 inches, then shortened by 1=½ inches. These movements would be transitional and would cause minor damage to services and internal walls. The vast majority of anticipated damage would therefore be due to transverse compression.

Transverse Damage - Anticipated transverse compression gave a predicted shortening of the structure of 8 inches. This suggested the following typical damage:

(i) Floor heave
(ii) Window and door loosening
(iii) Fractured glass
(iv) Fractured services
and (v) Distortion and fracturing of the roof trusses.

An estimate of the cost of repairing physical damage was $175,000(minimum) to $265,000 (maximum).[1] It was therefore decided that preventive works be used.

Precautionary Works - Preventive works were carried out in both the structure and the ground. These are as follows:

(1) Trenching - A trench was excavated to below foundation level around the main structure; the total length of trenching was approximately 1210 ft.

Services were isolated in the trench and flexible couplings were inserted to allow movement as the trench compressed. The trench was backfilled with clinker.

Auxiliary Precautions - The boundary wall was cut at strategic location leaving the largest section 100 ft. in length. The structure and foundation

[1] Costs have been converted to give 1979 equivalent costs.

were cut parallel to the directions of face advance to allow for dissipation of transverse compression. Additional work involved the taping of windows to prevent violent shattering.

6.5 The Investigation - This site was used to investigate the effect of preventive measures on a large structure subjected to critical movements.

Measurements include:

(a) Precise measurements of vertical settlement over full transverse section

(b) Measurement of strain over full transverse section

(c) Measurement of differential movements at wall cuts

(d) Measurements of differential movement across the trench in transverse and traveling directions.

6.6 Results of the Survey - Maximum subsidence of 45 inches was associated with a maximum tilt of 3.6 mm/m.

Performance of the Trench - Figure 7 shows trench closure for the measuring points 1-17. It can be seen that the East-side trench was particularly effective absorbing between 14 and 19.5 mm of compression while bay 1-2 on the western side closed by 19 mm.

Performance of the wall cuts - Figure 8 shows the relative movements of the isolated wall ends. Movement at the corner cuts is a consequence of transverse and traveling strains.

Structural Damage - Traveling damage developed as anticipated in the form of

i) cracks in brick work and plaster ceilings
ii) Loosened wall tiles
iii) Distortion of roof trusses
iv) Small cracks in the boundary wall.

Drainage was inconvenienced by the backward tilting of the structure, however, this was not a permanent problem.

Transverse movements consisting of subsidence and differential displacements resulting in permanent compression, caused concentrations of damaged brickwork along the line of maximum compression. The majority of ancilliary damage was confined to the lifting of floor boards over the center of the excavation.

Trenching proved particularly effective in this case study. It successfully reduced the predicted "very serious" damage to something approaching the "slight" category.

One aspect of the damage was a consequence of the traveling trench. As the face advanced towards the structure, cracks appeared in the main structure. Normally, these would close as the face worked past the site. However, compression associated with the traveling subsidence wave was absorbed by the trench and the cracks remained opened.

In the authors opinion, structures may best be protected against compressive subsidence damage by two parallel trenches positioned to absorb transverse compression.

7. Conclusion - In each of the case studies outlined, damage and hence repair costs were decreased by using preventive methods.

In each case, the revenue produced by the coal far outweighed the cost of preventive works and remedial measures.

In an established subsidence program, the following procedure is recommended:

1) Predict damage forming movements - Mining Factor(s)

2) Gauge effect of geology - Site Factor(s)

3) Gauge response of structure - Structure Factor(s)

4) Assign preventive works as illustrated in the text

5) Measure the effectiveness and economic impact of the preventive works.

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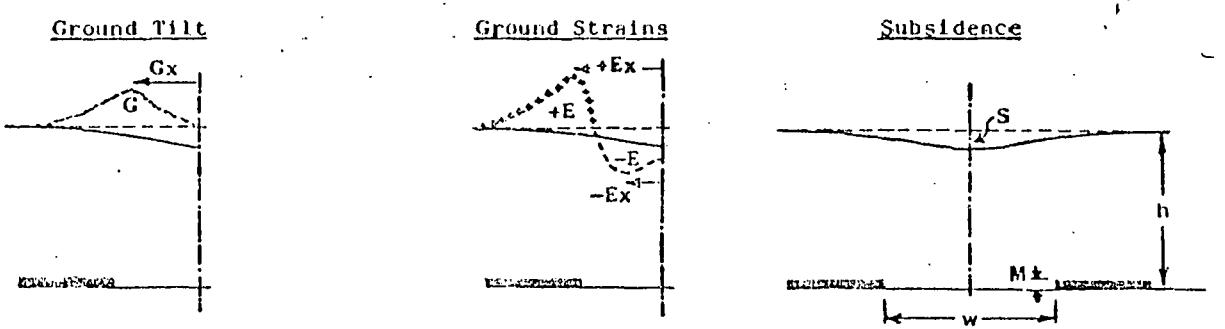
TABLE 1: Values of maximum subsidence, surface ground strains and tilt due to longwall mining (Compiled from the N.C.B. Subsidence Engineers' Handbook, 1966, 1975).

| Longwall width/depth (w/h) | $^1/_6$ | $^1/_5$ | $^1/_4$ | $^1/_3$ | $^1/_2$ | $^3/_4$ | 1.0 | 1.2 | 1.4 |
|---|---|---|---|---|---|---|---|---|---|
| Max. subsidence/extracted seam height S/M; caved wastes | 8% | 12% | 15% | 25% | 45% | 70% | 84% | 90% | 90% (max.) |
| **Coefficients for deducing magnitude and position of maximum ground strains and tilt** | | | | | | | | | |
| Max. strain due to compression (-E) | $2.3\frac{S}{h}$ | $2.2\frac{S}{h}$ | $2.15\frac{S}{h}$ | $1.9\frac{S}{h}$ | $1.35\frac{S}{h}$ | $0.75\frac{S}{h}$ | $0.55\frac{S}{h}$ | $0.5\frac{S}{h}$ | $0.5\frac{S}{h}$ |
| Position of -E from centre line (-Ex) | 0 | 0 | 0 | 0 | 0.02h | 0.10h | 0.20h | 0.29h | 0.39h |
| Max. strain due to extension (+E) | $0.45\frac{S}{h}$ | $0.5\frac{S}{h}$ | $0.65\frac{S}{h}$ | $0.75\frac{S}{h}$ | $0.8\frac{S}{h}$ | $0.65\frac{S}{h}$ | $0.65\frac{S}{h}$ | $0.65\frac{S}{h}$ | $0.65\frac{S}{h}$ |
| Position of +E from centre line (+Ex) | 0.50h | 0.49h | 0.42h | 0.34h | 0.32h | 0.40h | 0.51h | 0.61h | 0.70h |
| Max. ground tilt (G) (at transition pt.) | $1.9\frac{S}{h}$ | $2.2\frac{S}{h}$ | $2.6\frac{S}{h}$ | $3.15\frac{S}{h}$ | $3.35\frac{S}{h}$ | $2.85\frac{S}{h}$ | $2.75\frac{S}{h}$ | $2.75\frac{S}{h}$ | $2.75\frac{S}{h}$ |
| Position of G from centre line (Gx) | 0.34h | 0.32h | 0.27h | 0.22h | 0.21h | 0.26h | 0.37h | 0.46h | 0.56h |



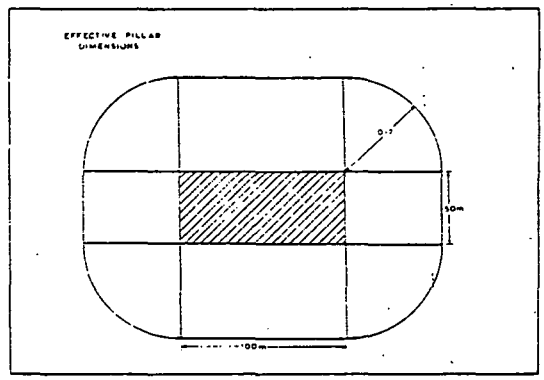

Example. Longwall width = 200m, depth = 400m, extracted seam height = 2m

Since w/h = 1/2, max. subsidence (S) = 45% (2m) = 0.90m at the centre line

Max. strain due to compression (-E) = $1.35 \times \frac{0.9}{400}$ = 0.0030 (or 3.0mm/m) at 8m from centre line

Max. strain due to extension (+E) = $0.8 \times \frac{0.9}{400}$ = 0.0018 (or 1.8mm/m) at 128m from centre

Max. ground tilt (G) = $3.35 \times \frac{0.9}{400}$ = 0.0075 (or 7.5mm/m) at 84m from centre




| Depth to seam (m) | Seam thickness t (m) | Lost Coal | |
|---|---|---|---|
| | | m³ | tons |
| 100 | 1m | 41,396 | 69,230 |
| 200 | 1m | 108,583 | 181,594 |
| 300 | 1m | [illegible] | [illegible] |
| 500 | 1m | 494,894 | 827,661 |
| 700 | 1m | 906,324 | 1,515,354 |
| 1000 | 1m | 1,756,580 | 2,934,361 |

Figure 1 Lost Coal From Varying Seam Thickness (After Breeds, 1976)

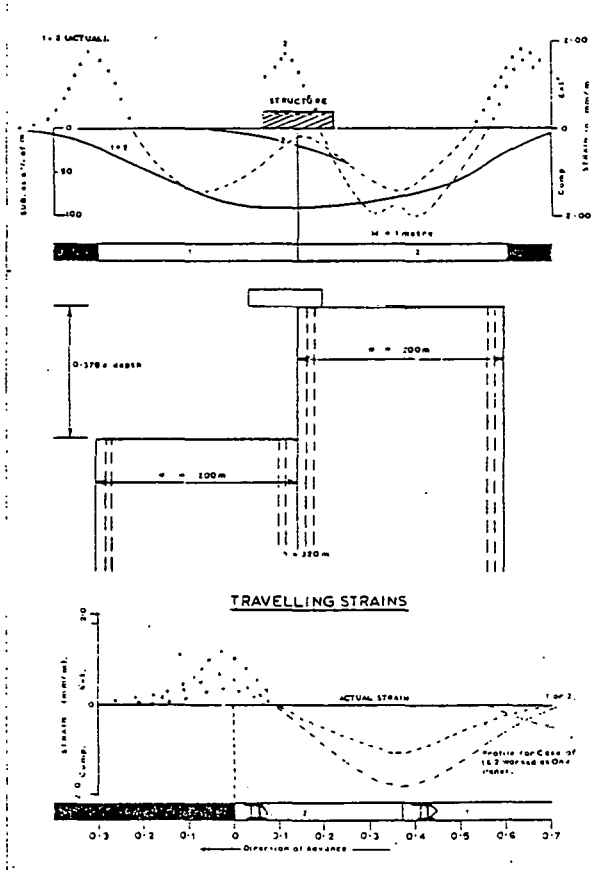



Figure 2 Example of Harmonic Working (After Breeds, 1976)

TABLE 2:Schedule of cases where "trenching" has been used for preventive purposes (after Whittaker & Breeds, 1977).

| Case No | Colliery and Seam/s | Mining Dimensions | | | | Anticipated Movements | | | Structure to be protected | Anticipated class of damage without any preventive works | Location of Trenches | Details of Trenches | | | | Degree of Damage which developed |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | w | h | m | w/h | + E | - E | p | | | | Depth | Length | Volume | Cost | |
| C8 | Selston Blackshale | 124m | 310m | 1.43m | 0.40 | 0.0010 | 0.0030 | 2,320m | Railway Bridge at Moorgreen. Abutment walls 21m in length | Slight - Appreciable | On two sides of each abutment | 1.8m | 15.5m | 17.0 c.m | £56 | None |
| C14 | Pye Hill Blackshale | 246m | 263m | 1.68m | 0.94 | 0.0020 | 0.0035 | 1,020m | Eastwood Hall 95 x 50 metres | Very severe | Around all main buildings | 0.4 m to 3.0 m | 552 m | 572 c.m. | | Slight/appreciable |
| C33 | Gedling High Hazles | 560m | 448m | 1.30m | 1.25 | 0.0008 | 0.0013 | 12,700m | St. Peter & St. Pauls Church, Shelford 33 x 16 metres | Slight | On western end only | 0.9m | 27 m | 15 c.m. | £59 | Very slight |
| C34 | Radford Tupton | 504m | 272m | 0.89m | 1.85 | 0.0015 | 0.0015 | 8,330m | Robert Shaw School, Nottingham 69 x 63 metres | Appreciable | On both ends & on part of sides of two main buildings | 0.8 m to 1.9 m | 119 m | 102 c.m. | £800 | Slight |
| C35 | Linby High Main | 266m | 439m | 1.22m | 0.61 | 0.0015 | 0.0020 | 7,140m | St. Mary's Church Blidworth 24 x 12 metres | Slight - Appreciable | On three sides of structure | 0.5 m to 1.4 m | 50 m | 32 c.m. | £337 | Very slight |
| C44 | Gedling High Hazles | 192m | 315m | 1.37m | 0.61 | 0.0017 | | 20,000m | St. Ann's Hospital, Nottingham 69 x 54 metres | Appreciable | On N.W. side of Hospital | 1.2m | 34 m | 25 c.m. | £80 | Slight |
| C45 | Moorgreen 2nd Waterloo | 267m | 133m | 1.35m | 2.01 | 0.0030 | 0.0030 | 1,330m | Kimberley Secondary School 74 x 54 metres | Very severe | On four sides of School | 1.4m | 265 m | 221 c.m. | £1,709 | Severe |
| C54 | Babbington Low Main | 242m | 347m | 0.91m | 0.70 | 0.0007 | 0.0015 | 7,690m | St. Patrick's Church Nuthall 24 x 11 metres | Slight | On East and West sides and part along North and South sides | 1.0m | 79 m | 40 c.m. | £357 | Very slight |
| C55 | Gedling High Hazles | 272m | 284m | 1.22m | 0.95 | 0.0015 | 0.0022 | 9,090m | St. Jame's Church Porchester 26 x 22 metres | Appreciable | On three sides of Church | 1.1m | 72 m | 40 c.m. | £314 | Slight |
| C56 | Gedling High Hazles | 288m | 304m | 1.12m | 0.95 | 0.0020 | 0.0020 | 3,700m | St. John's Church Carlton 40 x 24 metres | Appreciable | On four sides of Church | 1.2m | 106 m | 79 c.m. | £503 | Slight |
| C57 | Calverton High Main | 239m | 444m | 1.12m | 0.54 | 0.0010 | | 12,500m | St. Peter & St.Pauls Church, Oxton 24 x 18 metres | Slight | On three sides of Church | 1.4m | 38 m | 26 c.m. | £116 | Very slight |
| C69 | Bentinck 2nd Waterloo | 160m | 297m | 1.5m | 0.61 | 0.0013 | 0.0035 | 2,660m | Aristoc & Martins Factories at Kirkby in Ashfield 122 x 92 metres | Very severe | On four sides of both structures | 1.8m | 434 m | 484 c.m. | £5,000 | Appreciable |

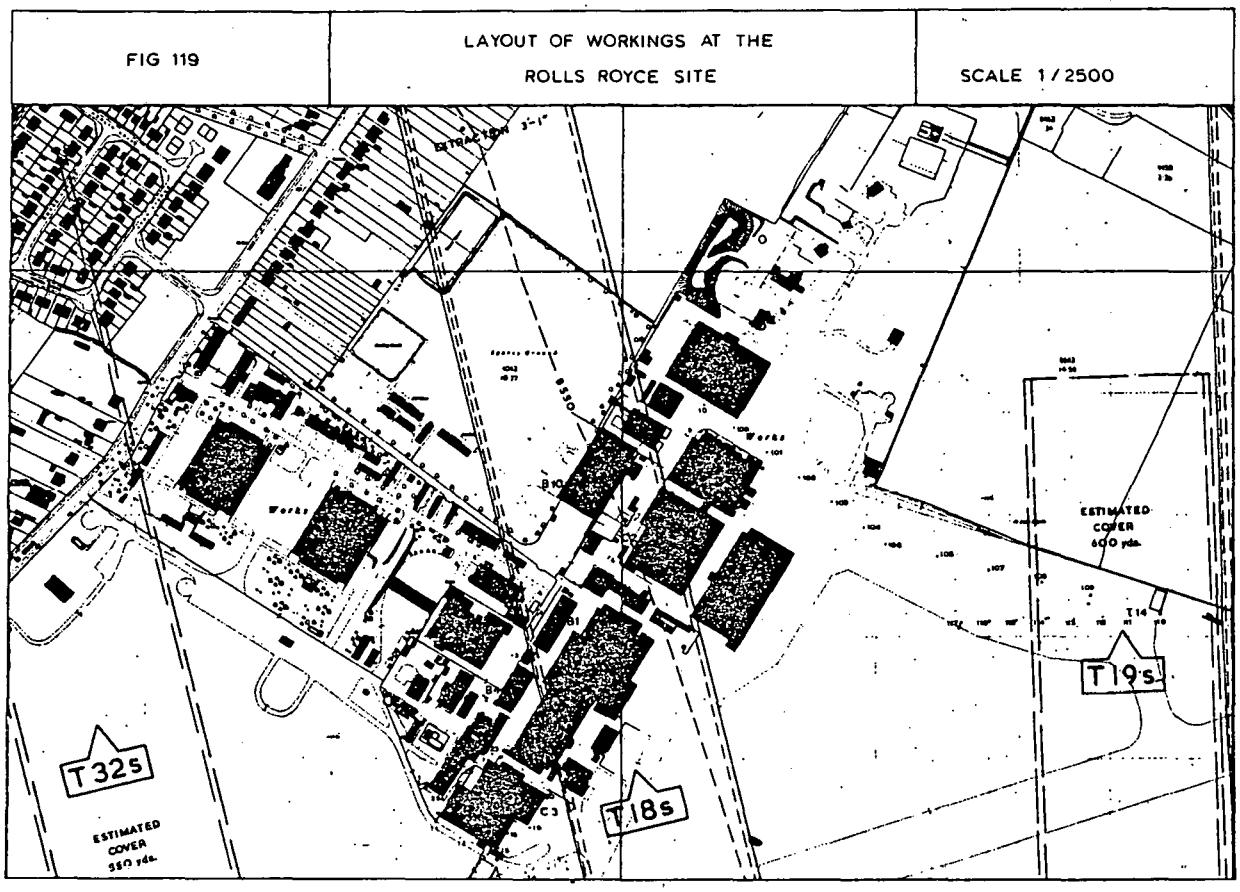



Figure 3 Panel Layout - Case Study Number 1

TABLE 3: Predicted Ground Movements - Case Study Number 1

| Structure | Maximum Subsidence (inch) | Maximum Strain (mm/m) | Maximum Tilt (mm/m) | Dimensions (ft) |
|---|---|---|---|---|
| B10 | 150 | -0.5 | 0.3 | 190 x 100 |
| B1 & B4 | 100 | -0.35 | 1.0 | 160 x 55 |
| C2 | 120 | -0.35 | 1.0 | 215 x 190 |
| T14 | 273 | -0.5 | 0.7 | 60 x 25 |
| C3 | 35 | +0.25 | 0.5 | 200 x 168 |

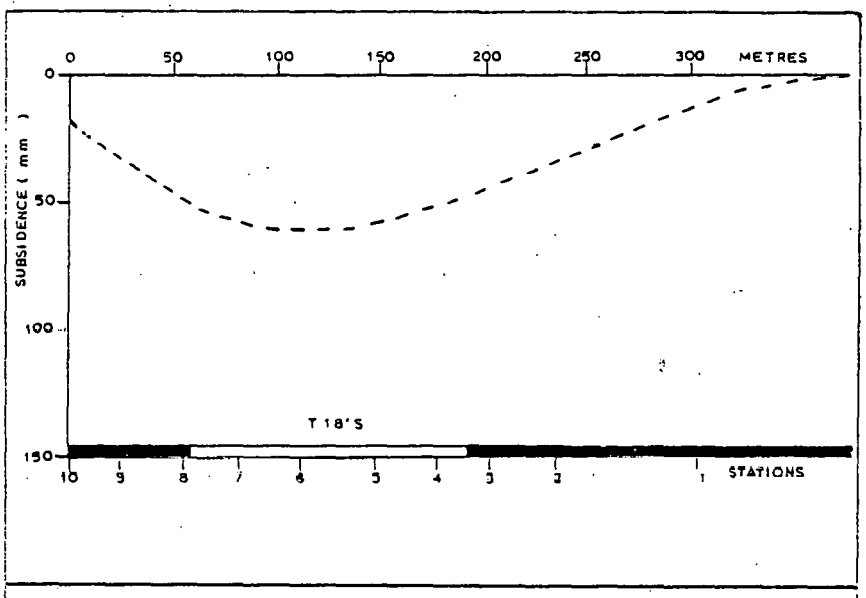



FIGURE 4

OBSERVED TRANSVERSE SUBSIDENCE PROFILE FOR T18'S

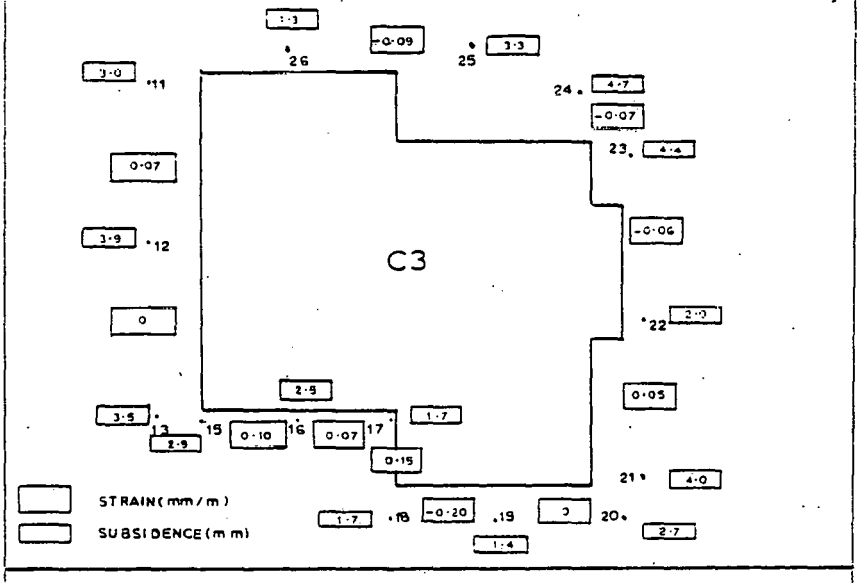
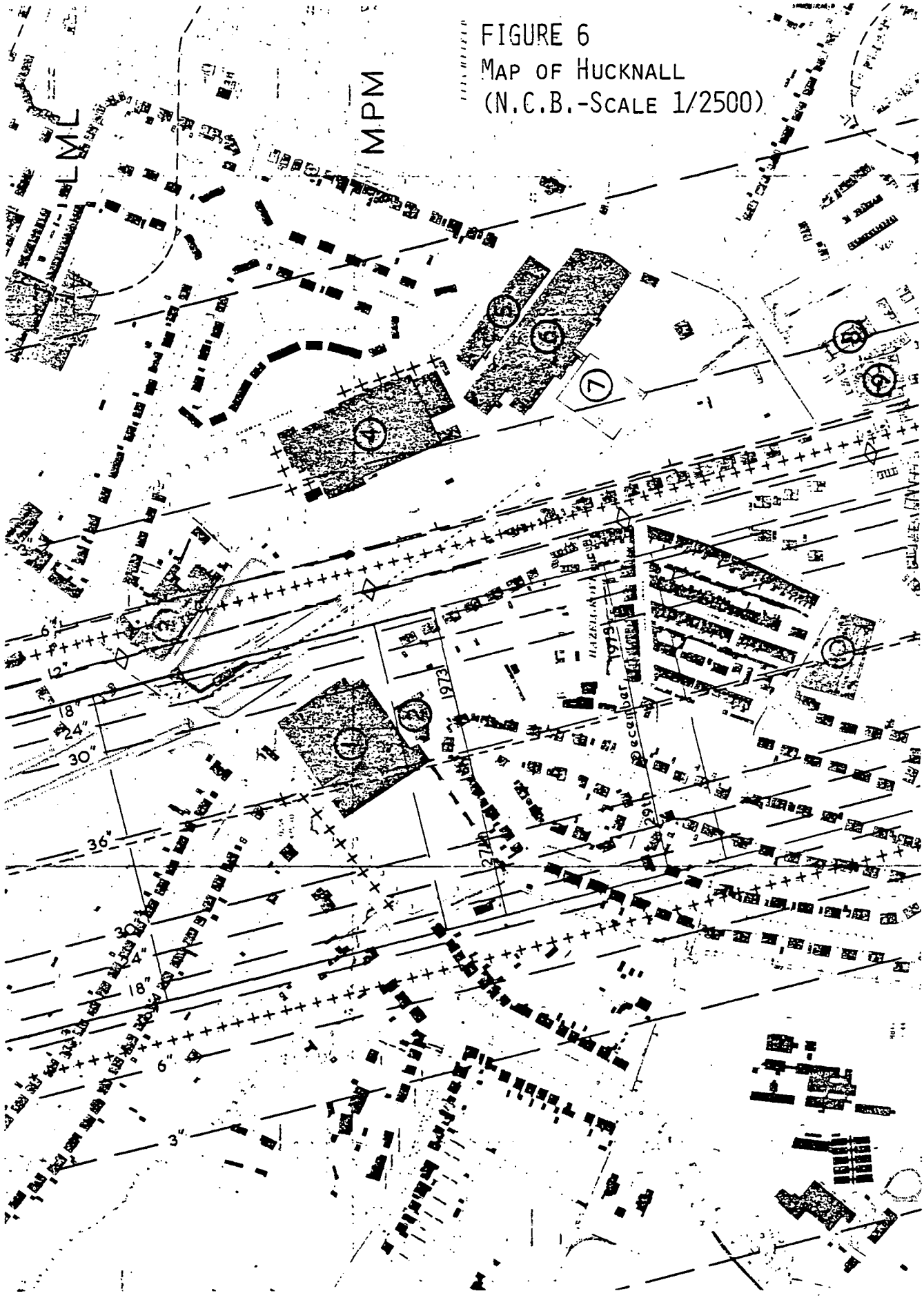



FIGURE 5

GROUND DISPLACEMENTS FROM T32'S

FIGURE 6
MAP OF HUCKNALL
(N.C.B.-SCALE 1/2500)



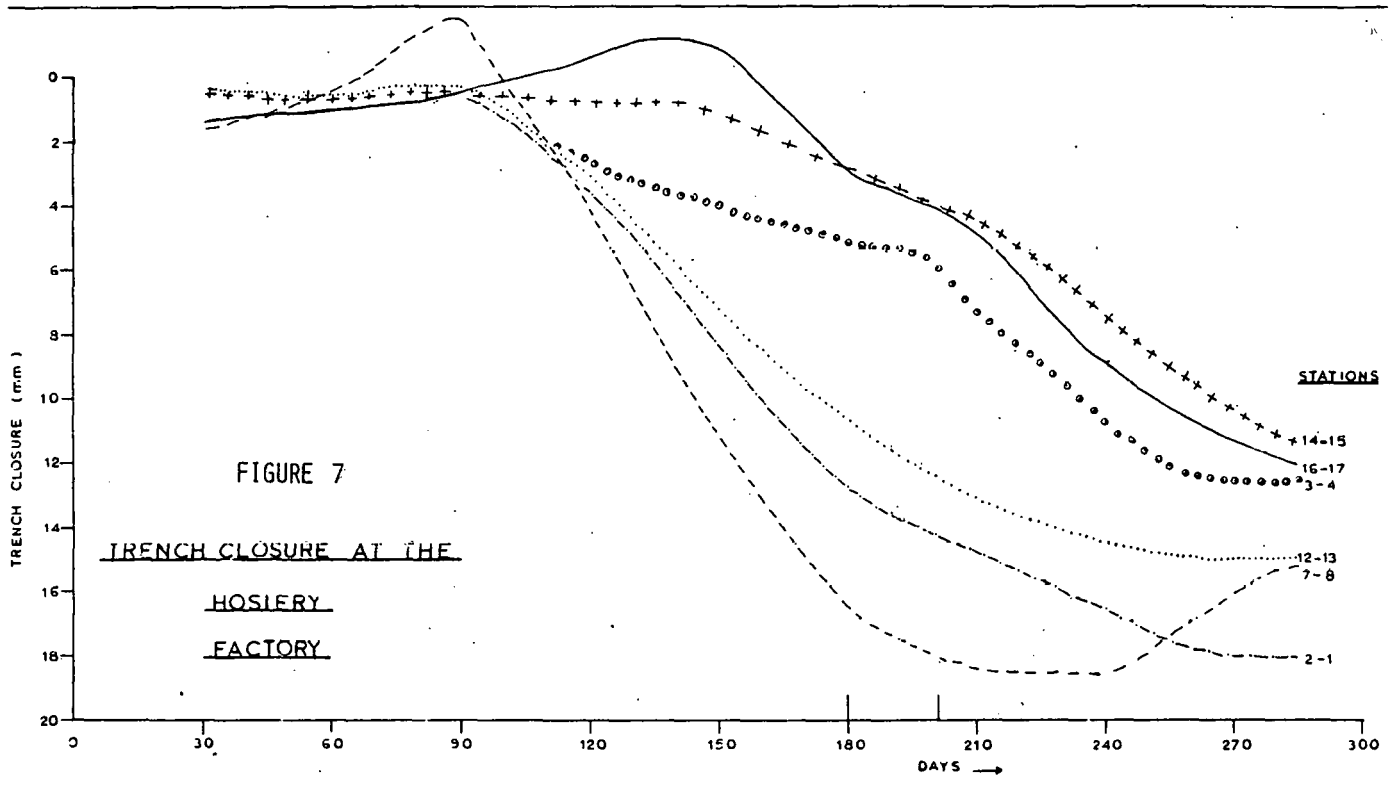



FIGURE 7

TRENCH CLOSURE AT THE HOSIERY FACTORY

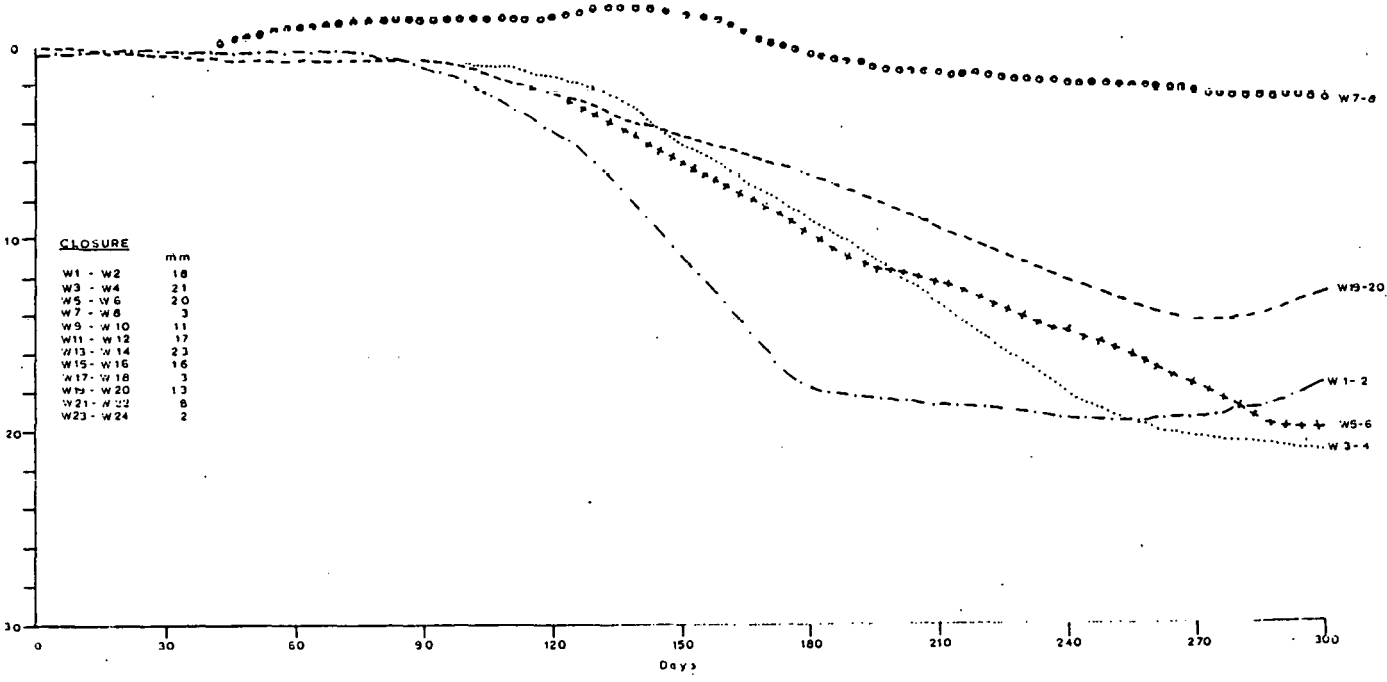



FIGURE 8

CLOSURE OF THE CUTS IN THE BOUNDARY WALL

# LIST OF PARTICIPANTS

John F. Abel, Jr.
Colorado School of Mines
Golden, CO 80401

J. Aggson
11244 West Mexico Drive
Denver, CO 80226

Michael J. Amoroso
201 N. Shore Land
R.R. 4
Marion, IL 62959

Raymond N. Ampy
175-8 Evergreen Terrace
Carbondale, IL 62901

Barbara L. Appelin
1002 Sumner Street
Carlinville, IL 62626

Jack D. Arthur
Office of Surface Mining
Indianapolis, IN

Nolan Augenbaugh
University of Missouri-Rolla
Rolla, MO 65401

Jitendra M. Avasthi
945 Hamlet Court #17
Monroeville, PA 15146

Tapan Baksi
136-7 Southern Hills
Carbondale, IL 62901

David Barr
125 Mining Boulevard
University of Missouri - Rolla
Rolla, MO 65401

Sukhendu Barua
516 South Rawlings #415C
Carbondale, IL 62901

R.A. Bauer
400 Hessel Boulevard
Champaign, IL 61820

Emery A. Beard
P.O. Box 330
Harrisburg, IL 62946

Charles Bollier
906 Mesa Verde Court
Taylorville, IL 62568

S.A. Bortz
10 West 35th Street
Chicago, IL 60616

C.D. Breeds
207A Holden Hall
Blacksburg, VA 24061

Robert W. Bruhn
570 Beatty Road
Monroeville, PA 15146

Dennis Bryant
310 E. Main Cross
Greenville, KY 42345

Thomas R. Bur
100 West Crystal Lake Road
Burnsville, MN 55337

Joseph Campbell
515 South Lincoln #23
Carbondale, IL 62901

R. Caudle
R.R. 1
Carbondale, IL 62901

Yoginder P. Chugh
1618 Tina Drive
Murphysboro, IL 62966

Man Mohan Comar
9 Greenway Plaza
Room 1466
Houston, TX 77046

Peter J. Conroy
1550 Northwest Highway
Park Ridge, IL 60068

Allen J. Costello
1003 Hillcrest
Sparta, IL 62286

R. Melton Fox
Route 5
Box 343A
Bessemer, AL 35020

Robert Cox
202 Woodland Hills
Tuscaloose, AL 35405

John Crelling
109 North Lark Lane
Carbondale, IL 62901

Mike Crow
400 North Oakland #16
Carbondale, IL 62901

Ernest A. Curth
US Department of Energy
4800 Farbes Avenue
Pittsburgh, PA 15213

H.H. Damberger
Illinois Geological Survey
200 Natural Resources Building
Urbana, IL 61801

Billie Dukes
613 North 8th Street
Murphysboro, IL 62966

Linda F. Dutcher
P.O. Box 124
Carbondale, IL 62901

John Edl
P.O. Box 2292
Carbondale, IL 62901

Paul J. Ehret
13 Tieman #9
Collinsville, IL 62234

William F. Eichfeld
P.O. Box 131
Carterville, IL 62918

David E. Elam
526 E. Buena Vista
Chester, IL 62233

Dale Elifrets
125 Mining Boulevard
University of Missouri - Rolla
Rolla, MO 65401

John L. Ellenberger
3608 Washington Avenue
Finleyville, PA 15332

A. Fred Ems
8022 S. Lamar
Littleton, CO 80123

Gary R. Fulton
916 West Main
Clark Dietz Eng. Inc.
Carbondale, IL 62901

William J. Garrow
John T. Boyd Company
Oliver Building - 4th Floor
Pittsburg, PA 15222

Subhen Ghosh
P.O. Box 2813
Carbondale, IL 62901

Robert D. Gibson
519 N. Kansas
Edwardsville, IL 62025

Gary E. Gray
555 17th Street
Denver, CO 80202

James B. Gulliford
105 North Rod Lane
Carbondale, IL 62901

Rama Nand Gupta
National Science Foundation
Washington

Charles J. Haas
Rock Mech. & Expl. Res. Cntr.
University of Missouri - Rolla
Rolla, MO 65401

Bruce Ham
39 Upper Lancaster Road
Ascot, Queensland, Australia

Denver Harper
Indiana Geological Survey
611 North Walnut Grove
Bloomington, IN 47401

Russ K. Harris
P.O. Box 2099
Houston, TX 77001

Charles Hayduk
1603 West Walnut
Carbondale, IL 62901

George H. Heidinger
250 Lewis Lane #80
Carbondale, IL 62901

Norman C. Hester
Kentucky Geological Survey
Lexington, KY 40506

Keith R. Honey
Box 566
Sesser, IL 62884

M.E. Hopkins
600 West 6th
Benton, IL 62812

Robert C. Howell
1376 Fontaine Road
Lexington, KY 40502

J. Janes
500 North DuQuoin Street
Benton, IL 62812

Dick K. Jeng
P.O. Box 2099
Houston, TX 77068

Marvin E. Johnson
300 North Smith Street
Carbondale, IL 62901

Kerry A. Johnston
1515 Windsor Drive #301
Arlington Heights, IL 60004

Leroy Kettren
1503 Alexander Avenue
Streamwood, IL 60103

William Kester
535 Nuthatch Drive
Zionsville, IN 46077

Issam E. Kheniser
905 E. Park #33
Carbondale, IL 62901

Cecil Kilgrove
105 West Bond
Benton, IL 62812

Robert P. King
10225 E. Grand #B105
Denver, CO 80231

Harry B. Kircher
Box 45
Southern Illinois University
Edwardsville, IL 62026

E.M. Klein
P.O. Box 2099
Shell Oil
Houston, TX 77001

Kewal K. Kohli
West Virginia University
Room No. 118 - White Hall
Morgantown, WV 26506

L.J. Kovacs
380 North Main
Mansfield, OH 44907

Hans-Friedrich Krausse
Am Kanal 9
D-4716 Olfen
Fed. Rep. Germany

Robert J. Krumm
707 Hilda
Edwardsville, IL 62025

Asim K. Kundu
516 South Rawlings #201A
Carbondale, IL 62901

Howard L. Leckey
555 17th Street
Denver, CO

Anthony S. Louveau
R.R. 2
Evansville, IL 62242

Guy N. McDowell, Jr.
R.R. 7 Box 220
Marion, KY

Stephen R. McCommons
303 S. McLeansboro
Benton, IL 62812

Bob McKelvey
500 North DuQuoin
Benton, IL 62812

Dana G. Meier
P.O. Box 144
Keensburg, IL 62852

William G. Meister
P.O. Box 2099
Houston, TX 77001

Satya Deb Misra
221 S. 14th Street
Murphysboro, IL 62966

Roger Missavage
R.R. 1
West Frankfort, IL 62896

C. P. Mangelsdorf
835 7th Street
Oakmont, PA 15139

Charles Medvick
Route 6 Box 140A
Marion, IL 62959

Bennie Morgan
411 E. Main Street
Morganfield, KY 42437

Mark Morgan
824 North Morgan Street
Morganfield, KY 42437

Michael L. Morris
P.O. Box 496
Carlinville, IL 62626

Robert Morse
US 51 South
El Paso, IL 61738

Michael Nowobilski
2313 Branch Road
Champaign, IL 61820

Guy Padgett
Consolidation Coal
101 Plaza East Boulevard
Evansville, IN 47715

Lester T. Pedersen
Battelle Pacific N.W.
P.O. Box 999
Richland, WA 99352

James L. Pfifer
Kerr-McGee Coal Corp.
P.O. Box 25861
Oklahoma City, OK 73125

Robert Pocreva
503 North Main
Trenton, IL 62293

Larry R. Powell
US Bureau of Mines
P.O. Box 1660
Twin Cities, MN 55111

Robert B. Pride
Box 566
Sesser, IL 62884

Duane K. Pulliam
128 E. Main
Benton, IL 62812

Russell Randolph
1622 Devon Drive
Indianapolis, IN 46226

Ken Redding
101 Plaza East Boulevard
Suite 309
Consolidation Coal
Evansville, IN 47715

Robert J. Reynolds
105 S. Meridian
Indianapolis, IN 46225

J.C. Riggs
R.R. 2 Box 240C
Carterville, IL 62918

John Rockaway
Box 146 Route 4
Rolla, MO 65401

Frank Ruskey
Denver Mining Research Center
Denver, CO 80033

T. Ryncarz
813 South University #F
Carbondale, IL 62901

Larry E. Sample
99 North Poplar
Sesser, IL 62884

William P. Santy
500 S. Dayton
Marion, IL 62959

Thomas Sawyer
211 Westernaire Drive
Marion, IL 62959

F.W. Schmechel
701 Barham
Johnston City, IL 62951

Karl W. Schuler
9208 Lona Lane
Albuquerque, NM 87111

Patrick Scher
4523 S. Garway
Littleton, CO 80123

J.J. Scott
Lecomma Star Route
Rolla, MO 65401

Lyle Sendlein
R.R. 2
Carbondale, IL 62901

Donald Simpson
503 E. Main Street
Benton, IL 62812

Madan M. Singh
2514 Wisconsin Avenue
Downers Grove, IL 60515

Frank J. Skudrzyk
28 Woodcrest MHP
Rolla, MO 65401

Lindell Sneed
500 North DuQuoin
Benton, IL 62812

David Stritzel
304 North Maple
Christopher, IL 62822

Mike Thompson
1200 South Hadfield
Marion, IL 62959

Richard Thill
US Bureau of Mines
P.O. Box 1660
Twin Cities, MN 55111

Robert Travelstead
Route #1
Carrier Mills, IL 62917

Raja Upadhyay
4572 S. Robb Street
Littleton, CO 80123

John Utgaard
516 South Hays
Carbondale, IL 62901

A. Van Besien
R.R. 2
Carbondale, IL 62901

George Verley
R.R. 4 Box 200
Marion, IL 62959

Stephen Webster
213 Wabash Avenue
Belleville, IL 62221

Grady White
625 E. Garfield
Belleville, IL 62221

Thomas Whitfield
US Steel Corporation
P.O. Box 42
Benton, IL 62812

Carl E. Williams
General Delivery
Valier, IL

Fred Wright
University of Kentucky
Lexington, KY 40506