NIOSH

IC 9453

INFORMATION CIRCULAR 9453

Proceedings: New Technology For Coal Mine Roof Support



U.S. DEPARTMENT OF HEALTH AND HUMAN SERVICES
Public Health Service
Centers for Disease Control and Prevention
National Institute for Occupational Safety and Health

CDC
CENTERS FOR DISEASE CONTROL AND PREVENTION

Information Circular 9453

Proceedings: New Technology For Coal Mine Roof Support

U.S. DEPARTMENT OF HEALTH AND HUMAN SERVICES
Public Health Service
Centers for Disease Control and Prevention
National Institute for Occupational Safety and Health
Pittsburgh Research Laboratory
Pittsburgh, PA

October 2000

ORDERING INFORMATION

Copies of National Institute for Occupational Safety and Health (NIOSH) documents and information about occupational safety and health are available from

NIOSH–Publications Dissemination
4676 Columbia Parkway
Cincinnati, OH 45226-1998

FAX:	513-533-8573
Telphone:	1-800-35-NIOSH (1-800-356-4674)
E-mail:	pubstaft@cdc.gov
Web site:	www.cdc.gov/niosh

This document is the public domain and may be freely copied or reprinted.

Disclaimer: Mention of any company or product does not constitute endorsement by NIOSH.

DHHS (NIOSH) Publication No. 2000-151

CONTENTS

Page

Abstract 1

GENERAL SUPPORT CONSIDERATIONS

Roof and rib fall incidents and statistics: a recent profile, by D. M. Pappas, E. R. Bauer, and C. Mark, Ph.D. 3
Fundamentals of coal mine roof support, by C. Mark, Ph.D., and T. M. Barczak 23

PRIMARY ROOF SUPPORT

Trends in roof bolt application, by D. R. Dolinar and S. K. Bhatt 43
Assessing coal mine roof stability through roof fall analysis, by G. M. Molinda, C. Mark, Ph.D., and D. R. Dolinar . 53
Load behavior of grouted bolts in sedimentary rock, by S. P. Signer 73
Summary of field measurements of roof bolt performance, by C. Mark, Ph.D., D. R. Dolinar, and T. M. Mucho 81
Skin failure of roof and rib and support techniques in underground coal mines, by E. R. Bauer and D. R. Dolinar . . . 99
Design of roof bolt systems, by C. Mark, Ph.D. 111

SECONDARY ROOF SUPPORT

Design methodology for standing secondary roof support systems, by T. M. Barczak, T. P. Mucho, and D. R. Dolinar 133
Optimizing secondary roof support with the NIOSH Support Technology Optimization Program (STOP), by T. M. Barczak 151
Cable support in longwall gate roads, by D. R. Dolinar and L. A. Martin 165
Material handling considerations for secondary roof support systems, by T. M. Barczak 193

LONGWALL SHIELDS

NIOSH safety performance testing protocols for standing roof supports and longwall shields, by T. M. Barczak 207
Examining longwall shield failures from an engineering design and operational perspective, by T. M. Barczak 223
Factors to consider when purchasing a new set of longwall shields, by T. M. Barczak 245

GROUND CONTROL INSTRUMENTATION

Instruments for monitoring stability of underground openings, by M. K. Larson, D. R. Tesarik, J. B. Seymour, and R. L. Rains 253
Evaluation of ground support at a trona mine using instrumented cable and rebar bolts, by L. A. Martin, C. A. Dreesbach, and R. Pakalnis, Ph.D. 271

UNIT OF MEASURE ABBREVIATIONS USED IN THIS REPORT

cm	centimeter	ksi	kips per inch
cm^2	square centimeter	kΩ	kilohm
ft	foot	lb	pound
ft^2	square foot	lb/d	pound per day
ft^3	cubic foot	lbf	pound of force
gal/min	gallon per minute	m	meter
GPa	gigapascal	m^2	square meter
hr	hour	m^3	cubic meter
in	inch	min	minute
in^2	square inch	mm	millimeter
in^3	cubic inch	MPa	megapascal
kcal/min	kilocalories per minute	N	newton
kg	kilogram	psf	pound per square foot
KHz	kilohertz	psi	pound per square inch
kN	kilonewton	sec	second
kN/d	kilonewton per day	V	volt
kPa	kilopascal		

PROCEEDINGS: NEW TECHNOLOGY FOR COAL MINE ROOF SUPPORT

Edited by Christopher Mark, Ph.D.,[1] Dennis R. Dolinar,[2] Robert J. Tuchman,[3]
Thomas M. Barczak,[4] Stephen P. Signer[5] and Priscilla F. Wopat[6]

ABSTRACT

Roof falls continue to be the greatest single safety hazard faced by underground coal miners. During 1996-99, 44 coal miners lost their lives in rock falls, and nearly 2,400 were injured. In addition, nearly 6,000 noninjury roof collapses were reported. Roof supports are installed to protect the miners, but support system failures contributed to most of these incidents.

Reducing the terrible toll taken by ground falls continues to be a major goal of research by the National Institute for Occupational Safety and Health (NIOSH). The purpose of these proceedings is to provide the mining community with a comprehensive survey of coal mine roof supports. Drawing on many years of research undertaken by the NIOSH Pittsburgh and Spokane Research Laboratories, this volume describes what types of support are available, how they work, and when they should be used. The major subjects covered include roof bolts, standing roof supports, cable supports, and longwall shields. Some special topics are also addressed, including an analysis of roof fall accident statistics, techniques for better skin control, material handling considerations, and longwall mining through recovery rooms.

This proceedings volume also contains information on several important new technologies, which are described here for the first time:

- Guidelines for selecting roof bolt length, pattern, and capacity that were derived from statistical analysis of the roof fall experience at 37 underground mines;
- A new design method for longwall tailgate supports; and
- A technique for measuring loads developed within cable bolts.

The papers in these proceedings were presented at open industry briefings conducted by NIOSH on New Technology for Coal Mine Roof Support. The briefings were held in Norton, VA, Charleston, WV, Evansville, IN, Tuscaloosa, AL, Price, UT, Glenwood Springs, CO, and Washington, PA. The papers were also presented at the Mine Safety and Health Administration (MSHA) Preventative Roof-Rib Outreach Program (PROP) Seminar, which was held at the National Mine Health and Safety Academy near Beckley, WV.

[1]Supervisory physical scientist, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.
[2]Mining engineer, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.
[3]Technical writer-editor, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.
[4]Research physicist, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.
[5]Mining engineer, Spokane Research Laboratory, National Institute for Occupational Safety and Health, Spokane, WA.
[6]Technical publications editor, Spokane Research Laboratory, National Institute for Occupational Safety and Health, Spokane, WA.

ROOF AND RIB FALL INCIDENTS AND STATISTICS: A RECENT PROFILE

By Deno M. Pappas,[1] Eric R. Bauer,[2] and Christopher Mark, Ph.D.[3]

ABSTRACT

During 1998-99, groundfall incidents resulted in 27 fatalities and were responsible for over 70% of all deaths in U.S. underground coal mines. To obtain a better understanding of where and why these incidents occurred, a comprehensive analysis of groundfall injuries and fatalities was conducted. The first portion of the study examined various factors associated with roof and rib fall injuries and reportable roof fall noninjuries that occurred during 1995-98. The study found that the room-and-pillar mining method has twice the groundfall incident rate than the longwall method. Mine locations with high groundfall rates seem to correlate to regions where there is a higher concentration of problematic coalbeds. For example, the Illinois Basin has very high groundfall rates, which can be traced back to several key coalbeds-Kentucky No. 13, Herrin/No. 6/ Kentucky No. 11, and Springfield No. 5/Kentucky No. 9. High rib fall rates were found in mines located in thick seams. Groundfall rates were found to be 30% to 40% higher during the months of July through September, possibly due to high humidity that may cause the shale mine roof to deteriorate.

The second part of the study examined the root causes of failure by reviewing all groundfall fatality reports for 1996-99. Primary and secondary hazard factors were assigned to each groundfall incident. The primary factors resulting in these groundfall fatalities were pillar extraction, traveling under unsupported roof, skin failure, construction, longwall faces, intersections, and geologic discontinuities. Defining prominent ground control incident trends and hazards will identify areas where additional study is needed and where innovative solutions need to be developed to reduce these severe occupational hazards.

[1]Civil engineer.
[2]Mining engineer.
[3]Supervisory physical scientist.
Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

Underground coal mining has always been recognized as one of the most hazardous occupations in the United States. Since 1910, more than 85,000 underground miners have lost their lives while mining coal. Approximately 47% of these fatalities, involving 40,000 miners, have occurred because of falls of roof and rib, which is a greater proportion than for any other type of incident classification. As shown in figure 1, more than 1,000 roof and rib fall fatalities occurred annually until the early 1930s. Decreases in roof and rib fatalities in the 1930s and late 1940s seem to coincide with drops in production due to slowdowns in the economy during to the Great Depression and the post-World War II recession. The use of roof bolts and the mechanization of the mining industry in the early 1950s considerably improved productivity and required a smaller workforce. As the number of miners worked underground decreased, so did the number of groundfall fatalities. However, the actual rate of roof and rib fatalities did not significantly drop until after the early 1960s. This drop may be due to several factors, including greater use of mechanical and then resin-grouted roof bolts, the Federal Coal Mine Health and Safety Act of 1969, research by the former U.S.Bureau of Mines (USBM) in the development of automated temporary roof supports (ATRS) and canopies, and the creation of the Mine Safety and Health Administration (MSHA), which mandated ground control safety and training programs [Pappas 1987].

Although the frequency of roof and rib fatalities has significantly decreased, from 1,300 fatalities in 1910 to 14 fatalities in 1998, groundfalls still injured 850 workers in 1998, resulting in 26,000 days lost. In addition to the injuries, more than 1,800 noninjury roof falls occurred in 1998. These noninjury roof falls were usually massive falls that extended beyond the height of the bolts, damaged equipment, stopped production, or disrupted ventilation. It has been estimated that the total cost of groundfall injuries during 1985-89 was $123.9 million, or approximately $1 million for every fatality and $6,835 for every injury [Peters and Randolph 1991].

The USBM identified increases and/or patterns that influenced groundfall injury rates during 1980-84 associated with mining method, mining height, geographic location, and mine size [Pappas 1987]. This study found that during that

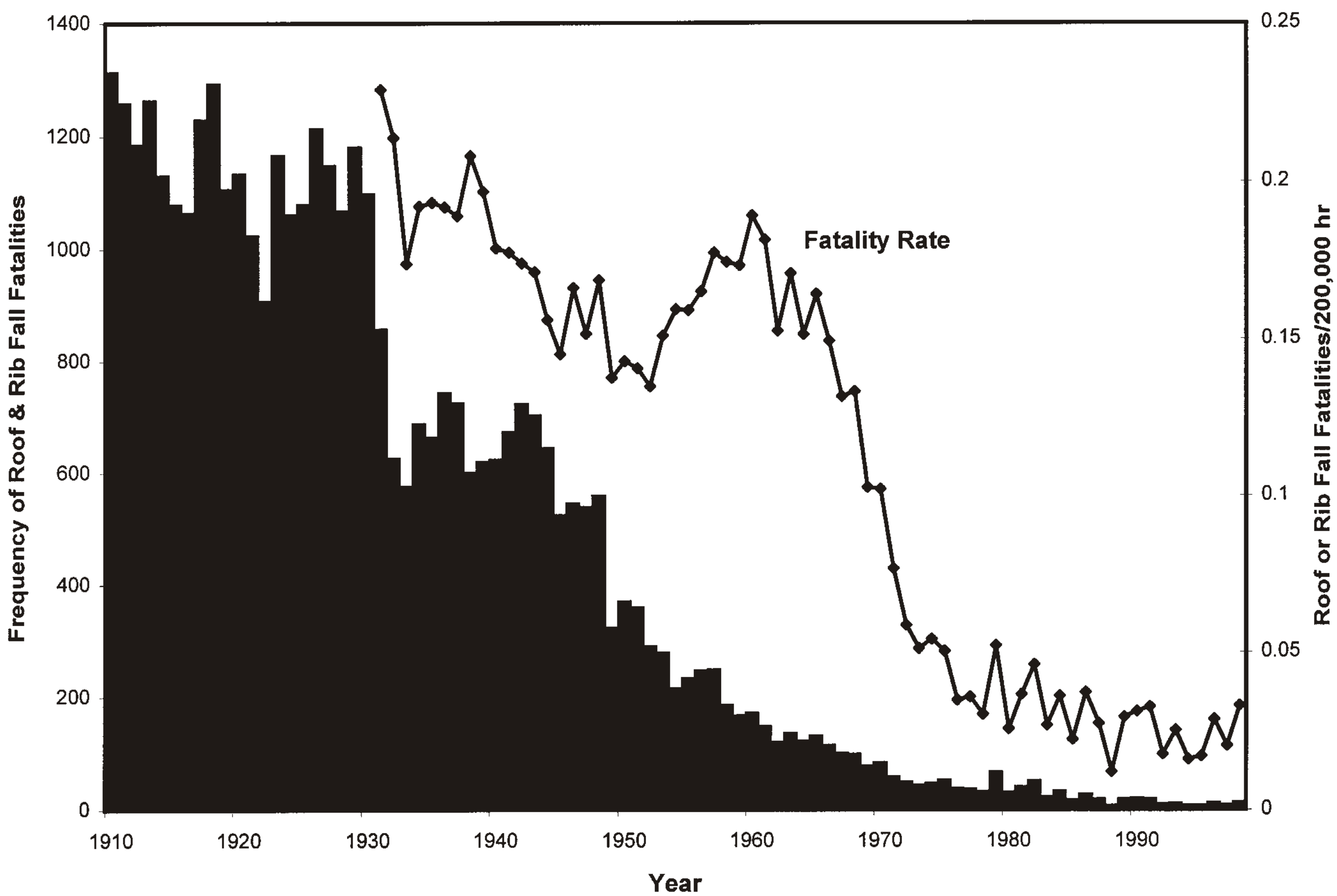


Figure 1.—Historical overview of groundfall fatalities, 1910-98. (Source: MSHA data.)

period, small mines (<50 employees) had severe groundfall fatality incidence rates about 5 to 20 times the fatality rate of medium- and large-sized mines. A slightly different study conducted by Fotta et al. [1995] found that in the Appalachian region in the early 1990s, small-mine groundfall fatality rates were elevated, but were significantly decreasing from levels in previous years. Although these studies are not identical, they indicate that dramatic changes were occurring in underground coal mines that were dynamically influencing groundfall patterns and characteristics. A major factor that may have influenced these fatality rate changes is the dramatic rise in use of the longwall mining method. Longwall mining in the United States has revolutionized and streamlined the process of mining underground coal by vastly improving mine productivity and miner safety. When the USBM study was conducted in 1984, longwall mining accounted for about 18% of the underground production; by 1994, however, longwall production doubled to about 37%. These more efficient longwall mines may have resulted in many smaller room-and-pillar mines closing or becoming more productive and safer in order to stay competitive with the longwall mines. This may have resulted in the dramatic decrease in small-mine fatality rates. Consequently, the goal of this study is to update groundfall fatality, injury, and noninjury incidence rates where the significant factor of longwall mining can be evaluated separately from room-and-pillar mining operations. The various attributes examined include mining method, location, mine size, seam height, coalbed, and seasonal patterns. In addition, specific hazards contributing to groundfall fatalities are discussed based on details compiled from MSHA fatality reports.

METHODOLOGY

Raw data for this study were obtained from the MSHA accident and address (listing of all operating mines) databases. The study examined the close-out data for the period 1995-98 for underground coal mines, excluding contractors. Ground control incidents included all roof and rib falls listed in the database, as well as incidents classified as "machinery" where the source of injury was caving rock. The narratives of these machinery-related groundfalls were reviewed to classify whether the incident was a roof or rib fall. For this study, roof and rib fall incidents were categorized into four groups:

1. Roof fall injury incidents resulting in death, in a permanent disability, in days away from work or days of restricted work activity, or in no lost workdays or restricted activity (degree of injury: 1-6).
2. Noninjury reportable roof fall incident (degree of injury: 0). 30 CFR 50.20-5 requires that every roof fall in active workings that occurs above the roof bolt anchorage, impairs ventilation, or impedes passage be reported.
3. Fatal roof or rib fall incident (degree of injury: 1).
4. Rib fall injury incident resulting in the same injuries listed in item 1 above.

The noninjury reportable roof fall incidence rate is important because it consistently tracks catastrophic roof failures according to MSHA regulations. A high noninjury roof fall rate may indicate situations where severe roof control problems are occurring since these incidents are associated with massive roof failure.

All incidents are converted into an incidence rate by computing the number of roof or rib fall incidents divided by the number of hours worked underground per 200,000 hr. The 200,000 hr approximates the number of hours worked by 100 full-time miners per year.

The two methods of mining-room-and-pillar versus longwall-were separated out for most of the analyses. The longwall mines were identified for each year by review of the annual longwall census and knowledgeable longwall mining individuals. It was determined that most longwall mines are dedicated exclusively to mining longwalls and gate road development; these mines were not proportioned by mining method. Therefore, all production originating from these designated mines were determined to be longwall mines; everything else was designated as room-and-pillar mine.

Major coal-producing locations in the United States were examined to identify trends. The eastern Kentucky region has minimal longwall production; by the end of the study period, no longwall was operational. Although the eastern Kentucky rates are listed, they should not be considered in the study since they represent very little production. Eastern Pennsylvania's anthracite coalfields are rather unique due to their folded stratigraphy, which is more similar to hard-rock mines than bituminous coal mines. Ground control conditions in eastern Pennsylvania are not equivalent to the other coal-mining locations, but are listed for consistency purposes.

Seam height used in the MSHA database is defined as the average height of coal seam currently being mined. There may be some variation in the seam height at the actual site of the incident versus the seam height listed in the MSHA database. Although the fatality reports record the actual working seam height at the site of the incident, this information is not available for all of the other types of injuries. To be consistent, this study will use the MSHA database seam height. Three categories of seam height are usually selected based on distribution of employee hours and other constraints [Fotta and Mallett 1997]. Thin-seam heights are ≤42 in, medium-seam heights are 43-60 in, and thick-seam heights are ≥61 in. Approximately 1.3% of the total underground hours are from mines where no seam height was recorded or was misreported and so were excluded from this portion of the study.

Mine size is based on the average annual number of employees working in the underground mine. Three categories of mine size that were selected are based on distribution of employee hours. Small-sized mines have ≤49 workers, medium-sized mines have 50 to 149 workers, and large-sized mines have ≥150 workers.

To obtain a greater understanding of the specific hazards associated with roof or rib fall fatalities, all underground groundfall fatality reports were examined for 1996-99. The fatality reports provide much more information than can be obtained from the MSHA injury narratives. The cause of failure for each groundfall fatality was reviewed by several individuals to minimize subjectivity and were categorized into eight primary and secondary hazard groups.

MINE CHARACTERISTICS

Table 1 breaks out several characteristics associated with room-and-pillar coal mines, such as the mining method, location, mine size, and seam height. Table 2 lists the same attributes associated with longwall mines. Specific factors that quantify these mine attributes during 1995-98 include the number of hours worked underground, number of mines, frequency of roof and rib fall injuries, and associated incidence rates.

MINING METHOD

According to figure 2, the roof fall injury incidence rate for room-and-pillar mines is more than double the longwall roof fall rate. On the other hand, the rib incidence rate for the two methods is nearly identical. The significantly lower longwall roof fall rate may be related to the continuous roof protection provided by the longwall face supports at the active mining face and the greater number of support workers located outby the mining face. The nearly identical rib fall rate probably reflects the thicker seams that are mined using longwalls. Table 3 compares the two mining methods. It is interesting to note that even though longwall mines represent only 6% of the underground coal mines, they are significantly larger on average, accounting for 48% of all the underground coal produced and 22% of all roof falls. Room-and-pillar operations represent 94% of the mines and account for 52% of the tonnage and 78% of all roof falls. It is also of interest that the percentage of reportable noninjury roof falls is very similar

Table 1.—Attributes of roof and rib fall injuries at room-and-pillar mines, 1995-98

Room-and-pillar characteristics	Hours worked		Mines		Roof falls[2]		Rib falls[2]		Incident rate	
	No.	%	No.	%	No.	%	No.	%	Roof falls per 200,000 hr	Rib falls per 200,000 hr
Location:										
Eastern PA	1,090,084	0.5	180	5.0	2	0.1	3	0.9	0.37	0.55
Western PA	16,332,658	7.6	189	5.3	134	6.1	15	4.4	1.64	0.18
Northern WV/OH/MD	9,829,896	4.6	185	5.2	85	3.8	9	2.6	1.73	0.18
Central WV	51,605,701	24.2	908	25.5	504	22.8	114	33.4	1.95	0.44
VA	24,543,233	11.5	583	16.4	273	12.4	32	9.4	2.22	0.26
Eastern KY	35,843,027	16.8	719	20.2	341	15.4	40	11.7	1.90	0.22
Central KY and TN	28,965,438	13.6	557	15.6	311	14.1	51	15.0	2.15	0.35
IL/IN	22,411,104	10.5	76	2.1	314	14.2	36	10.6	2.80	0.32
Western U.S.	5,584,663	2.6	76	2.1	48	2.2	35	10.3	1.72	1.25
Western KY	15,279,890	7.2	73	2.0	163	7.4	4	1.2	2.13	0.05
AL	2,150,106	1.0	19	0.5	35	1.6	2	0.6	3.26	0.19
Mine size:										
<50 workers	96,827,951	45.3	3,049	85.5	963	43.6	128	37.5	1.99	0.26
50-149 workers	80,891,498	37.9	439	12.3	855	38.7	145	42.5	2.11	0.36
>149 workers	35,916,351	16.8	77	2.2	392	17.7	68	19.9	2.18	0.38
Seam height:[3]										
<43 in	60,797,201	28.5	1,581	44.3	540	24.4	56	16.4	1.78	0.18
43-60 in	86,382,352	40.4	1,177	33.0	957	43.3	115	33.7	2.22	0.27
>60 in	63,556,005	29.7	706	19.8	686	31.0	168	49.3	2.16	0.53
All room-and-pillar mines	213,635,800	—	3,565	—	2,210	—	341	—	2.07	0.32

[1]The total number of mines for 1995-98 is not mutually exclusive (e.g., if a mine operated all 4 years, it is counted four times).
[2]All falls resulting in degree of injury of 1 to 6.
[3]Approximately 1.3% of the hours worked were at mines that did not report or misreported the seam height and are excluded.

Table 2.—Attributes of roof and rib fall injuries at longwall mines, 1995-98

Longwall characteristics	Hours worked No.	Hours worked %	Mines No.	Mines %	Roof falls[2] No.	Roof falls[2] %	Rib falls[2] No.	Rib falls[2] %	Incident rate: Roof falls per 200,000 hr	Incident rate: Rib falls per 200,000 hr
Location:										
Eastern PA	—	—	—	—	—	—	—	—	—	—
Western PA	23,198,139	16.1	29	11.7	114	17.8	29	13.5	0.98	0.25
Northern WV/OH/MD	35,971,363	24.9	57	23.1	100	15.6	20	9.3	0.56	0.11
Central WV	14,040,850	9.7	23	9.3	80	12.5	18	8.4	1.14	0.26
VA	7,454,159	5.2	16	6.5	23	3.6	3	1.4	0.62	0.08
Eastern KY	341,450	0.2	1	0.4	2	0.3	—	—	1.17	—
Central KY and TN	3,537,063	2.5	7	2.8	24	3.8	8	3.7	1.36	0.45
IL/IN	10,787,674	7.5	18	7.3	91	14.2	32	14.9	1.69	0.59
Western U.S.	16,881,139	11.7	56	22.7	35	5.5	62	28.8	0.41	0.73
Western KY	5,886,406	4.1	9	3.6	75	11.7	10	4.7	2.55	0.34
AL	26,087,525	18.1	31	12.6	96	15.0	33	15.3	0.74	0.25
Mine size:										
<50 workers	376,146	0.3	9	3.6	3	0.5	—	—	1.60	—
50-149 workers	10,374,482	7.2	46	18.6	40	6.3	29	13.5	0.77	0.56
>149 workers	133,435,140	92.5	192	77.7	597	93.3	186	86.5	0.89	0.28
Seam height:[3]										
<43 in	—	—	—	—	—	—	—	—	—	—
43-60 in	34,490,334	23.9	55	22.3	145	22.7	33	15.3	0.84	0.19
>60 in	108,855,545	75.5	188	76.1	491	76.7	182	84.7	0.90	0.33
All longwall	144,185,768	—	247	—	640	—	215	—	0.89	0.30

[1]The total number of mines for 1995-98 is not mutually exclusive (e.g., if a mine operated all 4 years, it is counted four times).
[2]All falls resulting in degree of injury of 1 to 6.
[3]Approximately 0.5% of the hours worked were at mines that did not report or misreported the seam height and are excluded.

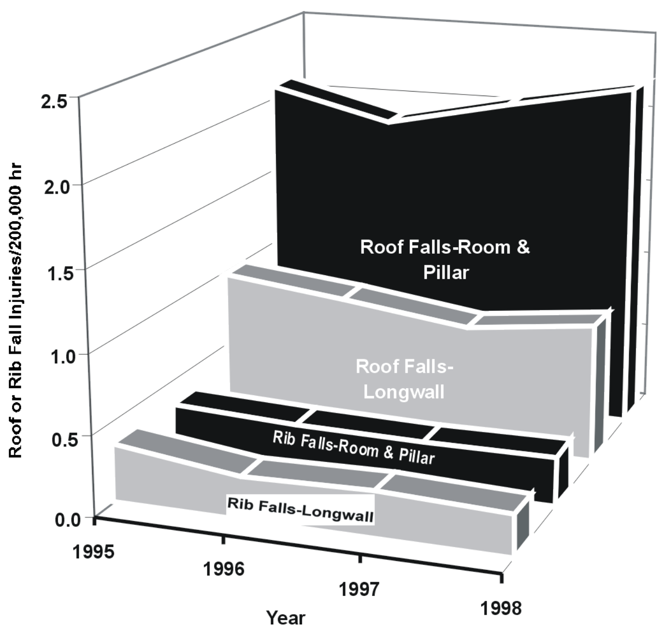


Figure 2.—Roof and rib fall injury rates by mining method, 1995-98. (Source: MSHA data.)

to the percentage of roof fall injuries for both mining methods. This may indicate that the geotechnical conditions that result in roof falls may be a function of the mining method used. Since there is such a large and distinct difference between the two mining methods, the remaining characteristic evaluations will break out the mining methods separately.

LOCATIONS

Examination of roof fall injury rates at longwall operations (figure 3) in all coal mining districts in the United States found that western Kentucky and Illinois/Indiana have roof fall injury rates that are 3 and 1.7 times the national longwall average, respectively. Conversely, the western United States, northern West Virginia/Ohio/Maryland, and Virginia have a longwall rate that is at least 25% less than the national average. To determine if these same trends are observable with massive roof falls, noninjury reportable roof falls were evaluated. Figure 4 shows a similar trend; western Kentucky and the Illinois Basin were found to have significantly higher longwall noninjury roof fall incident rates, and Alabama, Virginia, and northern West Virginia/Ohio/Maryland had lower rates. High fatality rates were identified in longwalls in southeastern Kentucky/Tennessee, the western United States, and Illinois/Indiana, while Virginia and western Kentucky had zero fatalities in longwall mines (figure 5). Longwall rib fall rates were high in the western United States, Illinois/Indiana, and southeastern Kentucky/Tennessee and low in Virginia and northern West Virginia/Ohio/Maryland (figure 6).

Locality trends of room-and-pillar operations found Alabama and Illinois/Indiana with high roof fall injury rates (figure 3); no location was found to a have significantly low rates except eastern Pennsylvania. Noninjury roof fall rates revealed that western Kentucky and northern West Virginia/Ohio/Maryland exceeded the national room-and-pillar mine average by 140% and 40% (figure 4), respectively, and eastern

Table 3.—Comparison of mining methods, 1995-98

Attribute	Mining method: Room-and-pillar No.	%	Longwall No.	%	Combined
Production:					
Tons produced	861,172,448	52	783,644,012	48	1,644,816,460
Underground hours worked	213,635,800	60	144,185,768	40	357,821,568
Workers	109,425	62	65,976	38	175,401
Mines[1]	3,565	94	247	6	3,812
Groundfalls:					
Roof fall injuries[2]	2,210	78	640	22	2,850
Rib fall injuries[2]	341	61	215	39	556
Roof fall noninjuries	6,093	80	1,543	20	7,636
Incident rates:[3]					
Roof fall injuries per 200,000 hr	2.07	—	0.89	—	1.59
Rib fall injuries per 200,000 hr	0.32	—	0.3	—	0.31
Roof fall noninjuries per 200,000 hr	5.70	—	2.14	—	4.27

[1]The total number of mines for 1995-98 is not mutually exclusive (e.g., if a mine worked all 4 years, it is counted four times).

[2]All falls resulting in degree of injury of 1 to 6.

[3]The combined rate is the total number of incidents divided by the total number of hours worked.

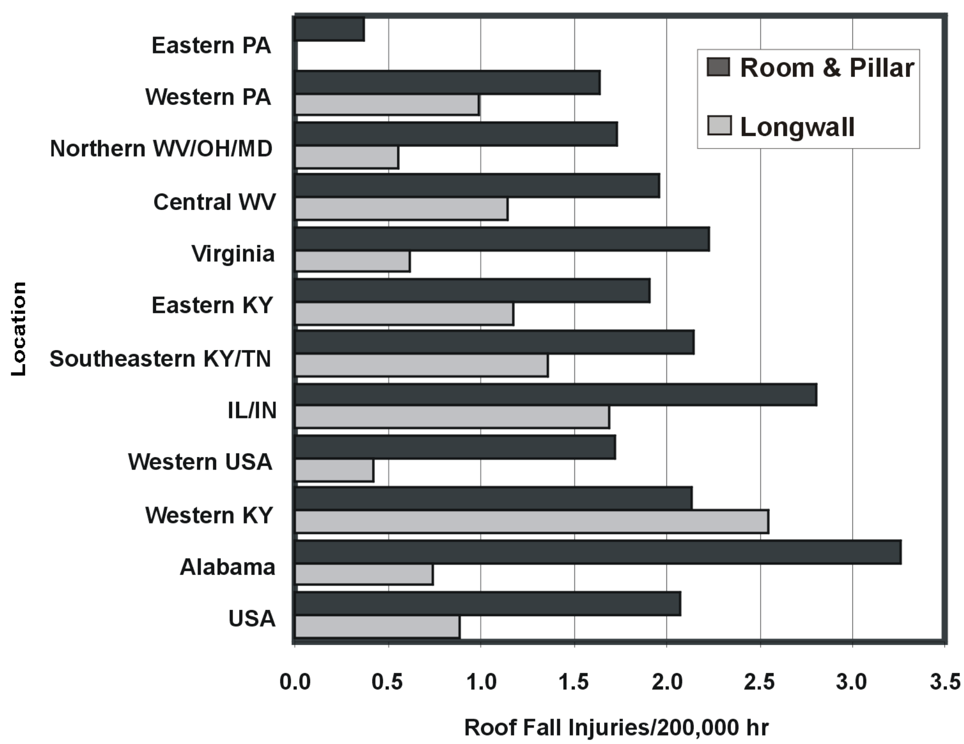


Figure 3.—Roof fall injury rates by location, 1995-98. (Source: MSHA data.)

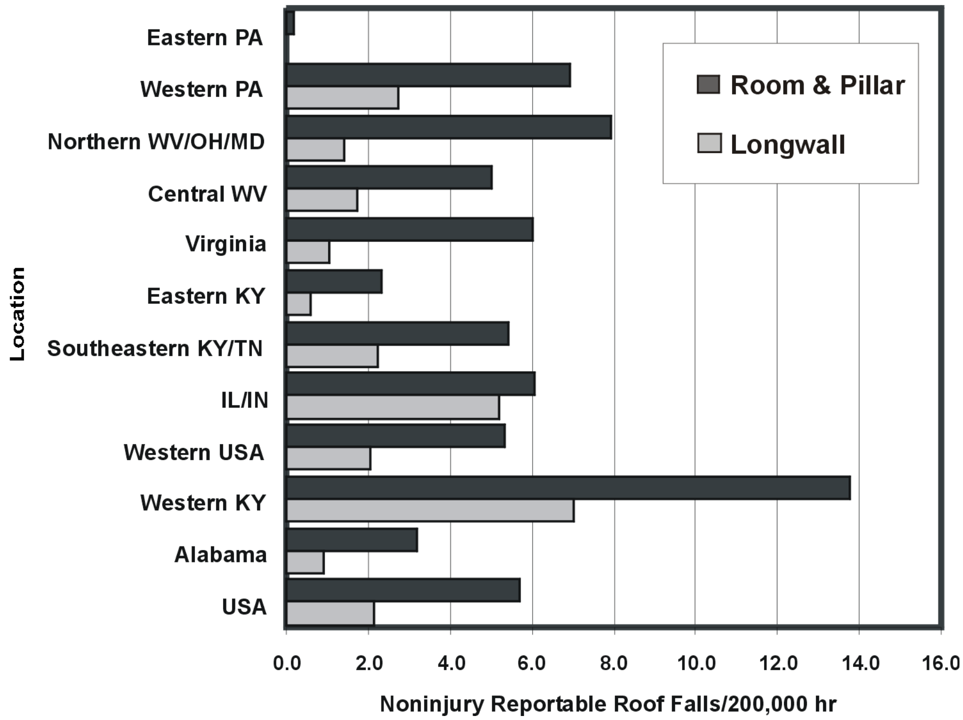


Figure 4.—Roof fall noninjury rates by location, 1995-98. (Source: MSHA data.)

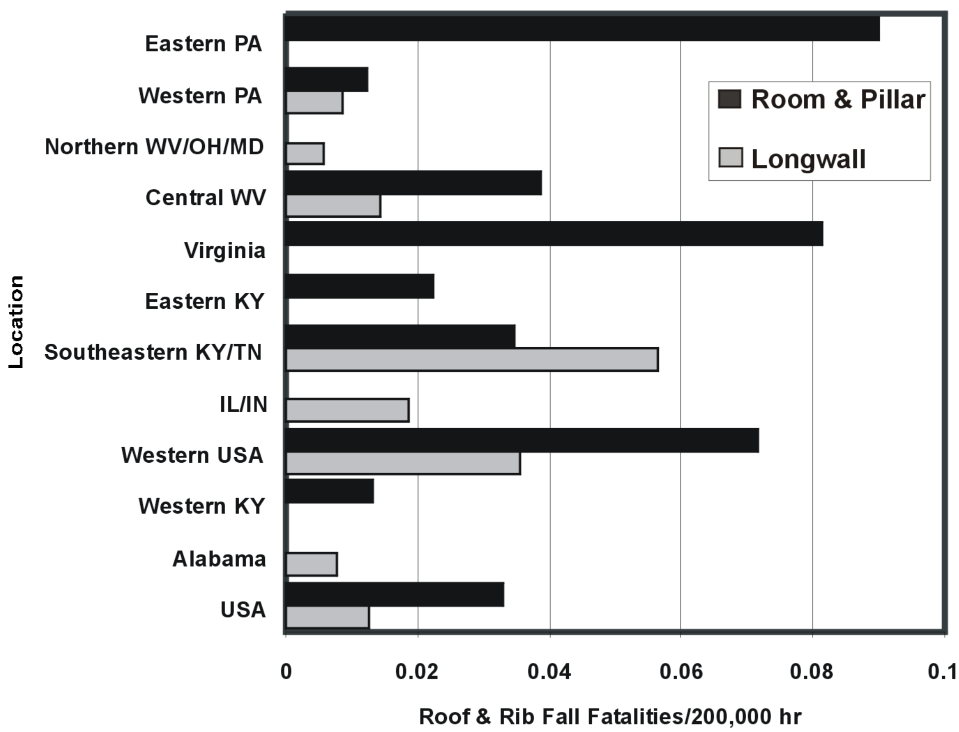


Figure 5.—Roof and rib fall fatality rates by location, 1995-98. (Source: MSHA data.)

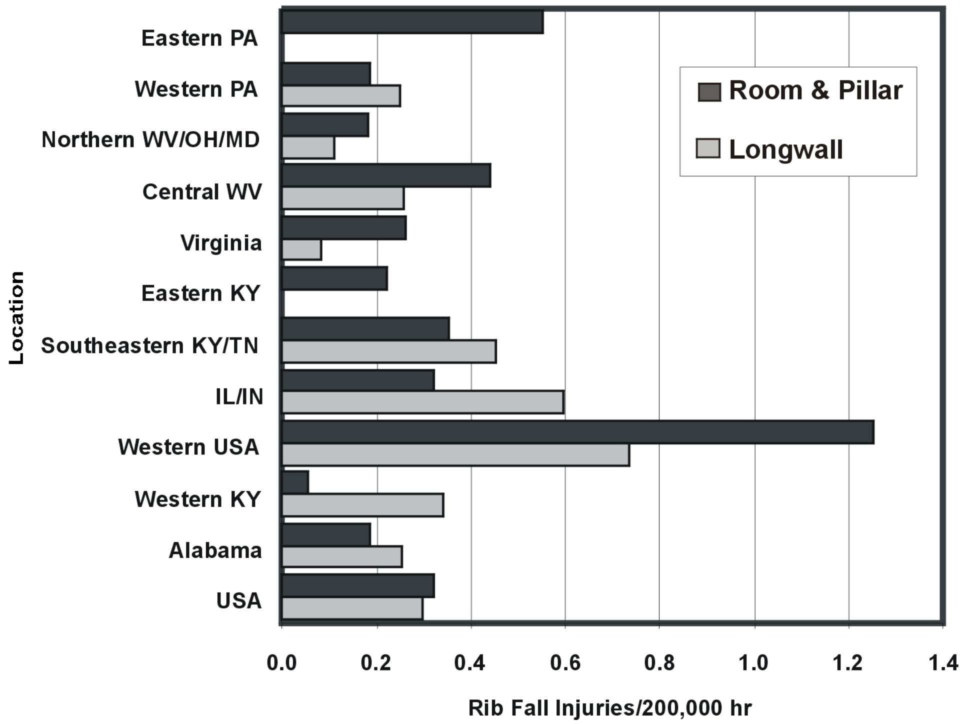


Figure 6.—Rib fall injury rates by location, 1995-98. (Source: MSHA data.)

Kentucky, eastern Pennsylvania, and Alabama were below the national average. Figure 5 displays the high groundfall fatality rates occurring at room-and-pillar mines in eastern Pennsylvania, Virginia, and the western United States and zero fatalities in northern West Virginia/Ohio/Maryland, Alabama, and Illinois/Indiana. The rib fall injury rate at room-and-pillar mines was four times the national average for the western United States; other high rates were found in eastern Pennsylvania and central West Virginia. Low rib fall rates occurred in western Kentucky, western Pennsylvania, and northern West Virginia/Ohio/Maryland (figure 6). High groundfall fatality and rib fall rates were found in eastern Pennsylvania and may be associated with the unusual folded stratigraphy of anthracite mines (figures 5-6). It is interesting to note that room-and-pillar mines in northern West Virginia/Ohio/Maryland had a noninjury reportable roof fall rate that is 40% higher than the national rate, yet the injury roof fall rate is 16% lower than the average, its rib injury rate is 44% less than average, and its fatality rate during this period was zero (figures 3-6).

Table 4 gives an overview of all of the high (at least 25% higher than the national average) and low (at least 25% lower than the national average) rates, ranking the locality for each type of groundfall rate. It is evident that for longwall mining,

Table 4.—Overview of coal mining locations with extreme groundfall rates, 1995-98

Mine type/rate	Above the national rate by 25%	Below the national rate by 25%
Room-and-pillar mines:		
Roof fall injury rate	IL/IN, AL	Eastern PA.
Roof fall noninjury rate	Western KY, northern WV/OH/MD	Eastern KY, AL, eastern PA.
Roof/rib fatality rate	Eastern PA, VA, Western U.S.	IL/IN, northern WV/OH/MD, AL.
Rib fall injury rate	Western U.S., eastern PA. central WV	Western KY, western PA, northern WV/OH/MD.
Longwall mines:		
Roof fall injury rate	Western KY, IL/IN, southeastern KY/TN	Western U.S., northern WV/OH/MD, VA.
Roof fall noninjury rate	Western KY, IL/IN	Eastern KY, AL, VA, northern WV/OH/MD.
Roof/rib fatality rate	Southeastern KY/TN, western U.S., IL/IN	VA, eastern KY, western KY.
Rib fall injury rate	Western U.S., IL/IN, southeastern KY/TN	VA, northern WV/OH/MD.

western Kentucky, southeastern Kentucky/Tennessee, and Illinois/Indiana are fairly consistently listed for each rate type, indicating that these regions have higher risk of groundfall hazards than any other parts of the country. Localities that had consistently lower risk of groundfall hazards for longwall operations include Virginia and northern West Virginia/Ohio/Maryland. Eastern Kentucky is listed with low rates; however, longwall activity in this region is almost negligible and is excluded. Room-and-pillar operations indicate a higher risk in the western United States and eastern Pennsylvania and a lower risk in northern West Virginia/Ohio/Maryland and Alabama.

Combining both mining methods shown in table 4 reveals that Illinois/Indiana and western Kentucky are consistently listed for both mining methods and for nearly all groundfall rate types. The unique coalbed conditions of this area may be an overriding factor producing this regional trend and will be discussed further in the "Coalbed" section below. Also evident in table 4 is the high rib fall injury rate in the western United States for both mining methods. The high groundfall fatality rate in the western United States may also be attributed to fatal rib falls. These occurrences in the western United States may be related to the higher and unstable ribs in the western United States, as well as deeper overburdens, which are more prone to bump and burst.

SEAM HEIGHT AND MINE SIZE

Previous studies [Fotta et al. 1997] have examined relationships between underground coal mine injury rates and mining height and have emphasized the importance of controlling the analysis for mining method and mine size. Mines operating in thin seams (<43 in) tend to be smaller mines that exclusively use the room-and-pillar extraction method. All longwall mines operate in medium or thick seams (>43 in) and are predominantly large mines (>149 workers). To control for mining method, all longwall mines are excluded from the study. It should be noted that there is not a wide distribution of mine sizes and seam heights for longwalls mines, so excluding longwalls will not overlook any seam height or mine size trends. Controlling the study for room-and-pillar mines, mine size, and seam height produces the groundfall incidence rate trends shown in figures 7 through 10. The roof fall injury incidence rates shown in figure 7 do not show any significant trends (±25% of the national average). However, the noninjury roof fall incidence rates (figure 8) reveal that small mines (<50 workers) and large mines (>149 workers) in thick seams (>61 in) have a significantly greater risk of massive roof falls. Conversely, mines in thin seams with small- and medium-sized workforces have a significantly lower noninjury roof fall rate. This trend slightly deviates with the groundfall fatality incidence rates shown in figure 9. Small mines in thin seams have a groundfall fatality rate that exceeds the national average by 44%, whereas small mines located in thick seams have a groundfall fatality rate that is 53% lower than the national average. With regard to rib fall injury incidence rates (figure 10), small- and medium-sized mines in thick seams exceed the national average by over 100% and 60%, respectively. Conversely, small- and medium-sized mines in thin seams have a significantly lower rib fall incidence rate (42% and 36% lower than the national average, respectively).

A comparison of all groundfall incidence rate trends in the table 5 generally shows a higher risk of groundfalls for thick seams for most mine sizes. By contrast, small thin-seam mines have an extraordinarily high fatality rate. A reverse trend occurs for groundfall rates of lower risk, particularly for small- and medium-sized mines in thin seams and for the fatality rate of small mines in thick-seam mines. Perhaps the lack of cabs and canopies in small, low seams results in higher groundfall fatality rates, especially when massive falls occur. Another explanation may be that not all of the groundfall injuries that occur in small mines are reported.

Parallel trends occur for rib fall incidence rates, with high rib fall rates at small- and medium-sized, thick-seam mines and low rates in similar-sized mines with thin seams. As the mining height increases, a greater surface area of the rib is exposed and at risk of becoming unstable or prone to collapse. Perhaps these high ribs are adequately supported in the large-sized mines, whereas small- and medium-sized mines may have a minimal staff to maintain the unstable ribs, little capital to purchase and install rib bolts, or they may be located in unusual coalbeds with complex ground control problems.

COALBED

To determine if certain coalbeds are more susceptible to groundfalls, the coalbeds where groundfalls occurred were evaluated. Since this is not a defined parameter in the MSHA database, a listing of all underground coal mines and associated coalbed names was obtained from the U.S. Department of Energy's Energy Information Agency and merged with the MSHA address information associated with every underground coal mine. All MSHA district offices were surveyed to find any missing coalbed names. Coalbed names were not identified for 15% of the room-and-pillar mines, which accounts for 2.4% of room-and-pillar mine hours worked; these were mostly small mines. All longwall coalbeds were accounted for.

Since more than 122 coalbeds were identified that produced coal during 1995-98, a process was developed to select the more significant coalbeds. Removed from the analysis were all coalbeds with fewer than 400,000 hr worked or with fewer than 4 incidents (for each type of incident: roof fall injury, rib fall injury, or noninjury reportable roof fall). The more significant coalbeds were determined by calculating the percentage that the coalbed groundfall rate exceeded the national average rate. All coalbeds that exceeded that national rate by at least 25% are listed regionally for each mining method in tables 6-7. The

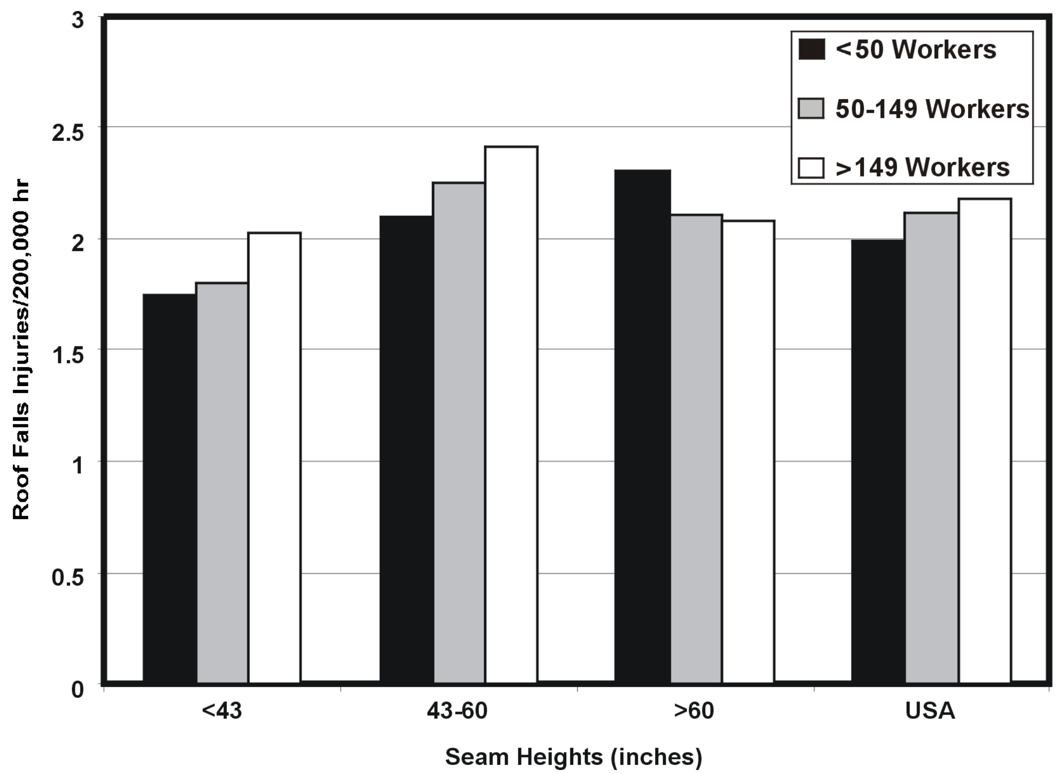


Figure 7.—Roof fall injury rates by mine size and seam height for room-and-pillar mines, 1995-98. (Source: MSHA data.)

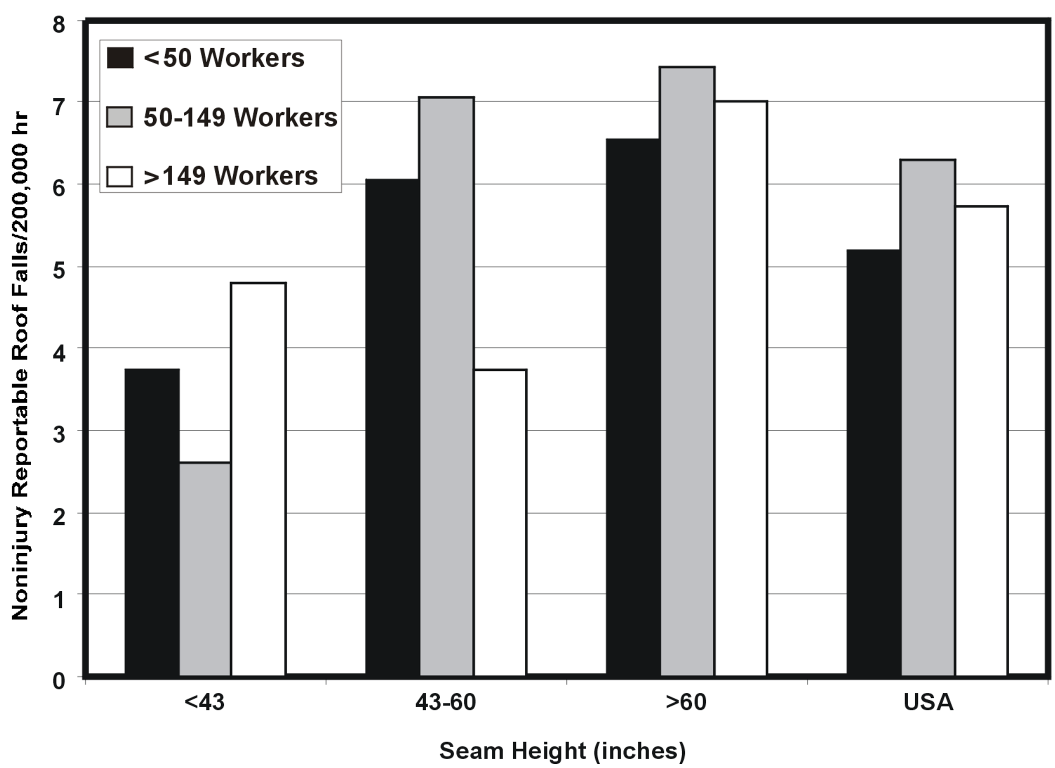


Figure 8.—Roof fall noninjury rates by mine size and seam height for room-and-pillar mines, 1995-98. (Source: MSHA data.)

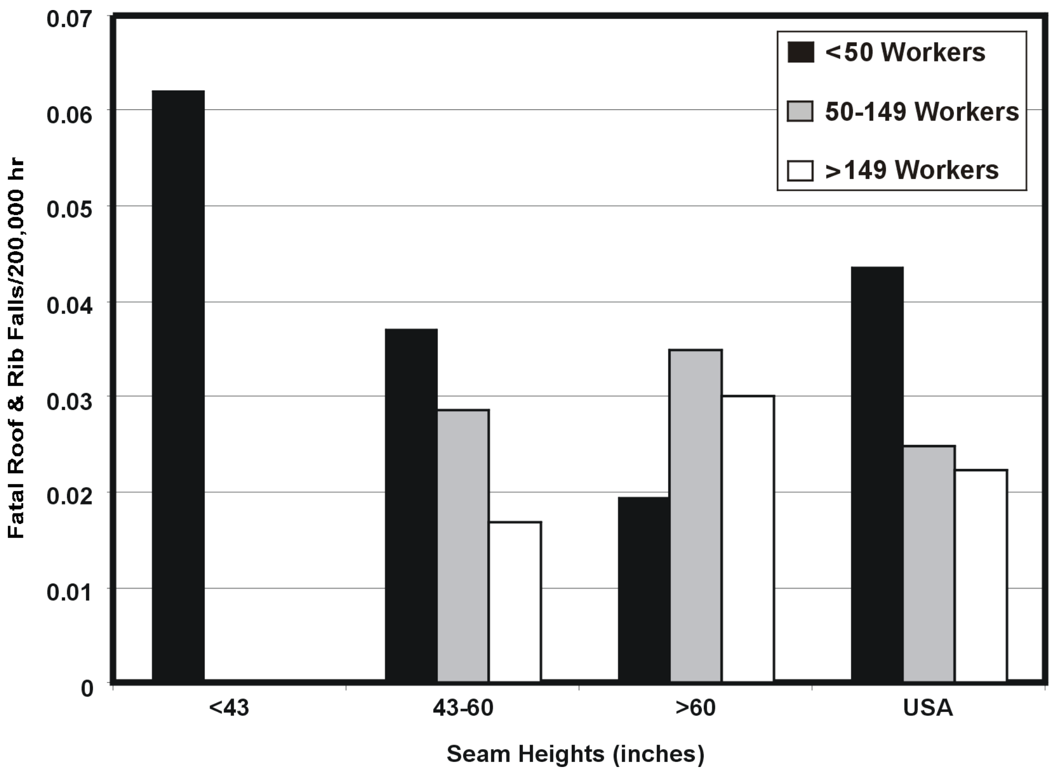


Figure 9.—Roof and rib fall fatality rates by mine size and seam height for room-and-pillar mines, 1995-98. (Source: MSHA data.)

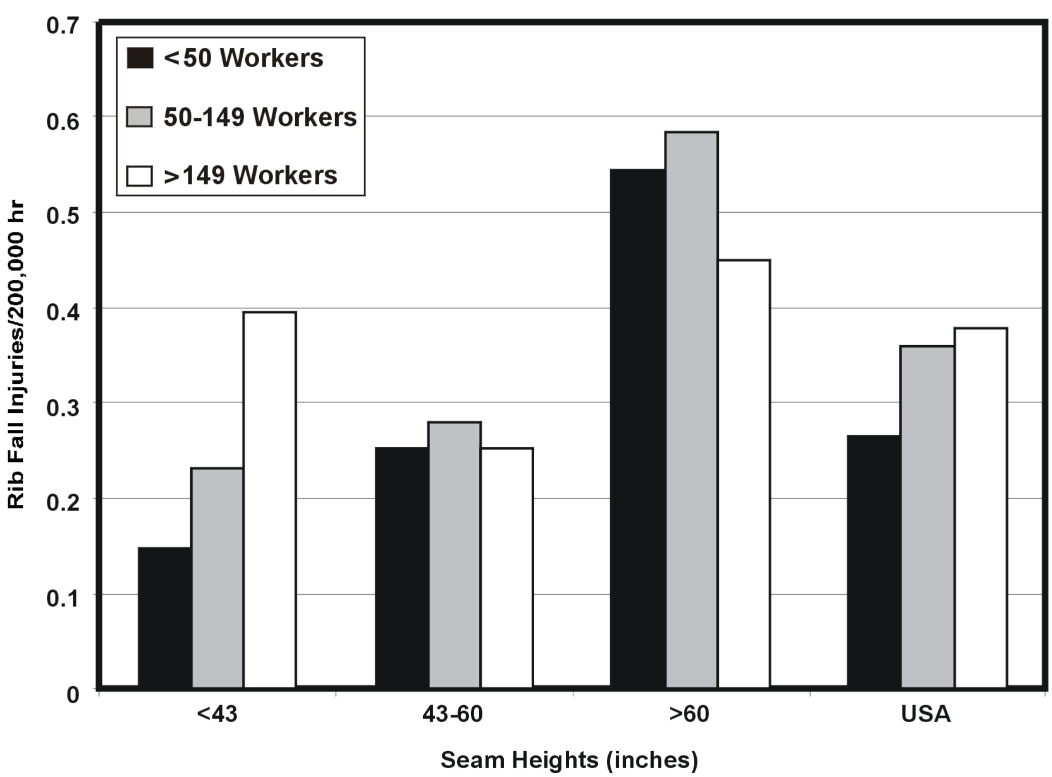


Figure 10.—Rib fall injury rates by mine size and seam height for room-and-pillar mines, 1995-98. (Source: MSHA data.)

Table 5.—Overview of mine size and seam height factors with extreme groundfall rates, 1995-98

Mine type/rate	Above the national rate by 25%	Below the national rate by 25%
Small mines (<50 workers):		
Roof fall injury rate	None	None.
Roof fall noninjury rate	Thick seam	Thin seam.
Roof/rib fatality rate	Thin seam	Thick seam.
Rib fall injury rate	Thick seam	Thin seam.
Medium mines (50-149 workers):		
Roof fall injury rate	None	None.
Roof fall noninjury rate	None	Thin seam.
Roof/rib fatality rate	Thick seam	Thin seam.
Rib fall injury rate	Thick seam	Thin seam.
Large mines (>149 workers):		
Roof fall injury rate	None	None.
Roof fall noninjury rate	Thick seam	Medium seam.
Roof/rib fatality rate	Thick seam	Thin seam.
Rib fall injury rate	None	Medium seam.

NOTE: A thin seam is ≤42 in; a medium seam is 43-60 in; a thick seam is ≥61 in. "None" indicates that seam thickness and mine size did not result in significant change (±25%) in incidence rate.

broad regions shown in the tables do not match the more specific locations listed earlier in the paper. Many of these coalbeds in specific locations were found to overlap, and broader regions were found to be a better indicator of defining the location of the coalbed.

The most startling indication from previewing the room-and-pillar coalbed listings in table 6 is the severe roof fall rates in Illinois Basin coalbeds that affect 95% of the coal mines in that region. This same trend is found in the longwall coalbed listing for the Illinois Basin, shown in table 7, which affects 100% of the longwall mines in the region. The notorious coalbeds include the Kentucky No. 13, Herrin/No. 6/Kentucky No. 11, and Springfield No. 5/Kentucky No. 9. These coalbeds have injury and noninjury roof fall rates that significantly exceed the national average for both mining methods, as well as high rib fall injury rates for longwall mines. These high groundfall incident rates in the Illinois Basin seem to correlate with the high rates indicated in a previous section of this paper that examined various mining localities, most notably Illinois/Indiana and western Kentucky. Obviously, some unique set of geological circumstances or stress fields are producing this regional concentration of groundfalls.

Other troubling trends from this analysis are the high injury and noninjury roof fall rates associated with 64% of the room-and-pillar mines in the northern Appalachian coalbeds, specifically the Sewickley, Redstone, Pittsburgh, Bakerstown, and Upper and Lower Freeport Coalbeds (table 6). These high roof fall injury rates are not carried over into the longwall mines of this region, except for the Upper Freeport and Sewickley Coalbeds. Possibly the geology in this region is more flexible and conducive for abutment load transfers that typically occur with longwall mining. However, the longwall mines in the Sewickley and Upper Freeport Coalbeds of the northern Appalachian region, which represent only 10% of the mines, have very high noninjury reportable roof fall rates. Perhaps the long-term standup time for these coalbeds is considerably shorter due to the high content of degradable shales that comprise the immediate roof.

Coalbeds that engender difficult longwall mining conditions seem to be located in the regions of central Appalachia and the western United States. Over 55% of the longwall mines in the central Appalachian region are located in coalbeds that have groundfall incident rates that exceed the national average by at least 25%. By contrast, only 29% of the room-and-pillar mines in this same region have rates that exceed the national average. Specifically problematic are the Eagle and Hazard No. 4 Coalbeds, which significantly exceed the national longwall rate for roof fall injuries and noninjury roof fall injury rates. Although the number of room-and-pillar mines in the central Appalachian region with problematic coalbeds do not represent a majority of mines, there are many coalbeds with high roof fall rates, including the Ben Creek/Blair, Upper Banner, Jawbone/Iaeger, and Walnut Mountain. In the same region, severe room-and-pillar rib fall injuries that exceed the national rate by over 100% include the Pocahontas No. 12, Eagle, Peerless, Powellton, and Amburgy/Low Splint Coalbeds.

Another problematic area is the western United States, with high rib fall rates that occur in 80% of the region's thick-seam longwall mines. Longwall coalbeds that have rib fall rates that exceed the national rate by over 100% include the Upper Hiawatha, Hiawatha, Blind Canyon, Wadge, and B Seam. Room-and-pillar coalbeds with high rib fall rates include the D Seam and B Seam. The thick seams and deep overburdens associated with the western United States probably contribute to the rib control problems and make the rib faces more prone to mountain bumps.

Table 6.—Room-and-pillar coalbeds with extreme groundfall rates, 1995-98

Region/coalbed[1]	DOE-EIA coalbed ID No.	Underground, hr	No. of mines[2]	Regional mines represented, %	Percentage above national rate		
					Roof injury	Rib injury	Roof noninjury
Northern Appalachian:							
Sewickley	29	727,722	26	—	—	—	223
Redstone	33	830,127	23	—	51	—	82
Pittsburgh	36	1,960,405	37	—	78	—	57
Bakerstown/Freeport	62	401,283	6	—	45	—	—
Upper Freeport	71	7,783,479	82	—	—	—	25
Lower Freeport	74	1,329,040	14	—	—	—	45
Upper Kittanning	76	6,505,446	53	—	—	—	27
Total	—	19,537,502	241	64.44	—	—	—
Central Appalachian:							
Hazard No. 8	100	677,371	11	—	28	—	—
Hazard No. 7/High Splint	104	592,004	14	—	—	—	54
Coalburg/Hazard No. 6	111	11,558,414	143	—	30	—	—
Winifrede/Hazard No. 5	121	4,138,977	45	—	—	51	—
Hatfield/No. 9	127	614,743	3	—	—	—	25
Walnut Mountain	128	488,422	12	—	—	—	180
Hernshaw/Whitesburg	137	1,787,018	37	—	41	—	—
Amburgy/Low Splint	142	6,700,453	112	—	—	96	—
Peerless	167	1,352,385	18	—	—	132	—
Powellton	170	4,253,833	34	—	—	121	—
Eagle	176	2,614,897	48	—	—	164	33
Bens Creek/Blair	177	890,525	17	—	139	—	136
Glamorgan	185	1,855,293	52	—	35	—	—
Splash Dam	210	3,504,704	85	—	—	25	—
Upper Banner	214	904,347	36	—	60	—	28
Jawbone/Iaeger	266	3,755,667	84	—	26	—	38
Lower Iaeger/No. 4	269	476,014	15	—	—	—	47
Pocahontas No. 12	311	2,939,232	27	—	—	199	—
Total	—	49,104,299	793	28.66	—	—	—
Southern Appalachian:							
Gholson	223	1,182,661	4	—	137	—	—
Total	—	1,182,661	4	21.05	—	—	—
Illinois Basin:							
Danville/No. 7	480	2,228,338	8	—	47	—	—
KY No. 13	482	784,239	4	—	147	—	937
Herrin/No. 6/KY No. 11	484	14,943,246	58	—	36	—	—
No. 5/Springfield/KY No. 9	489	18,216,127	63	—	—	—	87
Western KY No. 4	520	900,097	9	—	—	—	99
Total	—	37,072,047	142	95.30	—	—	—
Western United States:							
B Seam	1753	744,443	5	—	—	574	—
D Seam	1755	621,980	6	—	102	1,210	47
Cameo	1770	401,341	7	—	—	—	127
Lower O'Connor	1830	675,148	4	—	29	—	56
Total	—	2,442,912	22	28.95	—	—	—

[1]This analysis excludes coalbeds with fewer than 400,000 hr worked and fewer than four groundfall incidents.
[2]The total number of mines for 1995-98 is not mutually exclusive (e.g., if a mine worked all 4 years, it is counted four times).

Table 7.—Longwall coalbeds with extreme groundfall rates, 1995-98

Region/coalbed[1]	DOE-EIA coalbed ID No.	Underground, hr	No. of mines[2]	Regional mines represented, %	Percentage above national rate		
					Roof injury	Rib injury	Roof noninjury
Northern Appalachian:							
Sewickley	29	741,827	2	—	—	—	505
Upper Freeport	71	2,737,966	7	—	73	—	200
Total	—	3,479,793	9	10.47	—	—	—
Central Appalachian:							
Hazard No. 4	135	1,729,817	4	—	95	—	30
Alma/Elkhorn No. 1/Blue Gem	157	2,193,175	6	—	34	84	—
Imoboden/Warfield	168	1,259,612	5	—	43	—	—
Eagle	176	5,093,936	11	—	46	71	71
Total	—	10,276,540	26	55.32	—	—	—
Southern Appalachian:							
Pratt/Corona	227	2,040,393	4	—	—	—	143
Total	—	2,040,393	4	12.90	—	—	—
Illinois Basin:							
KY No. 13	482	3,254,456	4	—	357	44	285
Herrin/No. 6/KY No. 11	484	6,780,473	14	—	49	88	148
Springfield/No. 5/KY No. 9	489	6,639,151	9	—	87	62	144
Total	—	16,674,080	27	100.00	—	—	—
Western United States:							
Wattis	1236	1,225,605	3	—	—	—	83
Wadge/Roland of Tuff	1750	2,388,299	4	—	—	153	—
B Seam	1753	2,692,812	9	—	—	149	87
Lower O'Connor	1830	2,264,391	8	—	—	78	—
Castle Gate B/Upper O'Connor	1832	829,604	4	—	—	—	46
Hiawatha	1846	2,340,148	9	—	—	187	—
Upper Hiawatha	1847	1,059,750	4	—	—	280	—
Blind Canyon	1855	1,694,672	4	—	—	256	—
Total	—	14,495,281	45	80.36	—	—	—

[1]This analysis excludes coalbeds with fewer than 400,000 hr worked and fewer than four groundfall incidents.
[2]The total number of mines for 1995-98 is not mutually exclusive (e.g., if a mine worked all 4 years, it is counted four times).

SEASONAL PATTERNS

The chronological quarterly groundfall rates were evaluated to determine if seasonal patterns, such as fluctuations in temperature, barometric pressure, and humidity, might affect the number of groundfall incidents. Since the western United States has mostly an arid climate with minimum fluctuations in humidity, incidents in Colorado, New Mexico, Utah and Wyoming were excluded. Although monthly production data are not compiled, quarterly hours worked were accessed from MSHA's Terra database. This allowed the groundfall incidents to be normalized based on the quarterly employee hours worked underground. According to figure 11, the roof fall injury incidence rate is fairly consistent, except for the third quarter (July to September), where the incident rate peaks 30% higher than the other three quarters. A similar pattern occurs with the noninjury reportable roof fall rate, as shown in figure 12. The fall rate is fairly consistent until the third quarter, where the noninjury roof fall incident rate peaks 48% higher than the first two quarters. Using the noninjury reportable roof fall rate is an even better indicator of unstable ground conditions, since they usually result in massive falls and are required to be reported. Possibly this trend shows that mine air becomes more humid during the summer months and the moisture is disintegrating the shale roof, resulting in large groundfalls. It is interesting to note that in figure 12 the noninjury fall rate for the fourth quarter (October to December) is slightly elevated compared to the first two quarters. Perhaps the third quarter trend is continuing into the first month of the fourth quarter (October).

Other studies have found similar seasonal patterns. Stateham and Radcliffe [1978] found that humidity has a strong influence on roof fall occurrence rates. Their results indicated that the probability of a roof fall is highest in August and lowest in February.

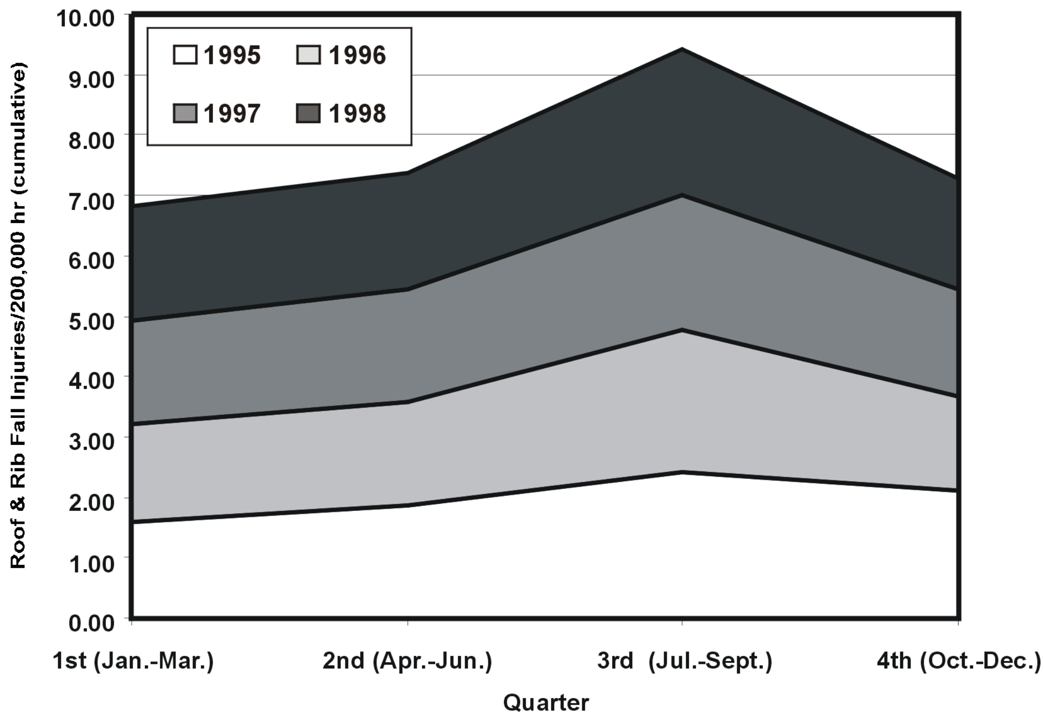


Figure 11.—Roof and rib fall injury rates by quarter, excluding the western United States, 1995-98. (Source: MSHA data.)

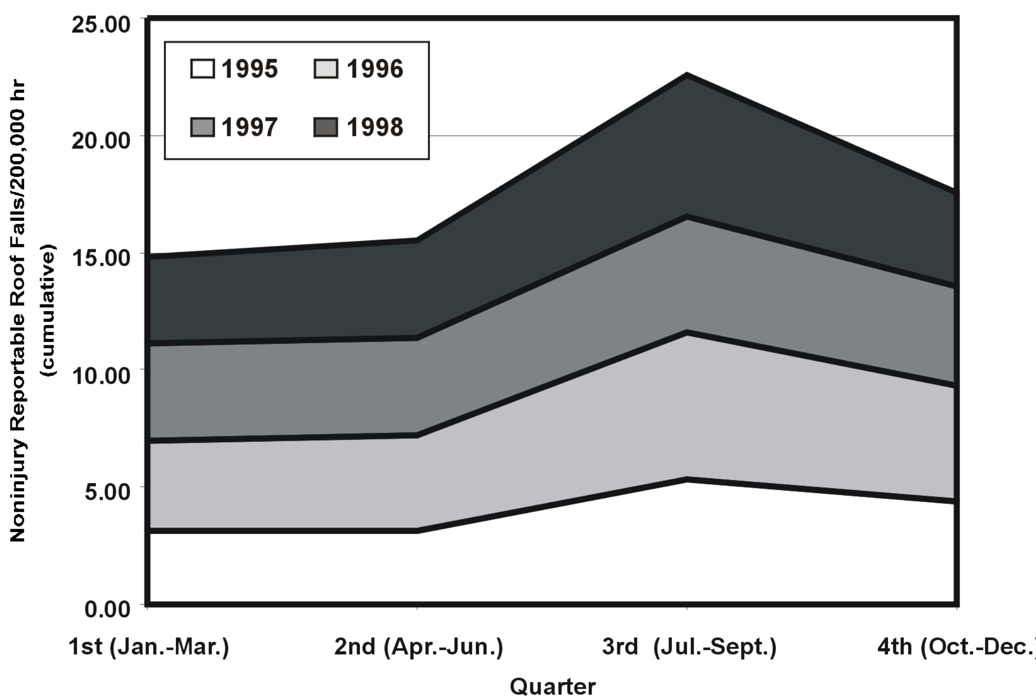


Figure 12.—Roof fall noninjury rates by quarter, excluding the western United States, 1995-98. (Source: MSHA data.)

OVERVIEW OF FATALITY REPORTS ASSOCIATED WITH GROUND CONTROL HAZARDS

During 1996-99, 49 underground coal miners were killed in 46 separate incidents. Table 8 lists the frequency of primary and secondary factors that contributed to these incidents. In some cases, more than one hazard was involved. For example, 12 fatalities occurred during pillar extraction; three of the incidents resulted from premature collapses in intersections.

UNSUPPORTED ROOF

When miners go under unsupported roof, they are completely unprotected. According to table 8, approximately 24% of all roof fall fatalities during the study period occurred when miners traveled under unsupported roof. While there are no grounds for complacency, the recent record does represent an improvement from a decade ago, when nearly 50% of groundfall fatalities occurred beneath unsupported roof [Peters 1992]. The improvement was achieved through new equipment, enforcement, and a persistent educational campaign.

By definition, roof support activities occur very close to unsupported roof. Therefore, it is not surprising that most of the fatal accidents involved roof bolt operators or other miners engaged in roof support. Based on the accident record, single-head roof bolt machines seem to be a risk factor. Roof control plans carefully specify the sequence of bolt installation with single-head machines to avoid placing the operator inby support. If these guidelines are not followed, the roof bolt operator can be at risk.

During the early 1990s, the USBM conducted an extensive series of interviews with miners to determine why they risk going under unsupported roof [Peters 1992]. The most common response was that they had unintentionally walked out beyond the supports. The most effective countermeasure, then, is to ensure that all areas of unsupported roof are clearly posted with highly visible warning devices.

Relatively simple procedures or technologies can be implemented to reduce the temptation for workers to intentionally go beyond support. However, training is also essential. Other USBM studies [Mallett et al. 1992] argue that verbal admonitions and threats of discipline are less effective than training that graphically imparts the severe consequences of roof falls. A series of three videos was prepared in which actual miners are interviewed about roof fall accidents that they had experienced. The videos also emphasize the impact of roof fall accidents on people other than the one caught in the fall. These highly effective videos, together with training manuals, are available from MSHA's National Mine Health and Safety Academy near Beckley, WV.

ROOF SKIN FAILURES

Skin failures are incidents that do not involve failure of the roof support elements, but result from rock spalling from between roof bolts, around ATRS systems, or from ribs. They are of particular concern because they cause injuries and fatalities to workers who should have been protected by supports. In 1997, 98% of the 810 roof and rib injuries suffered by mine workers were attributed to skin failures [Bauer et al. 1999]. Because groundfall fatalities are usually the result of massive roof failure, this study found only 12% of the roof falls were related to smaller scale roof skin failures.

Roof skin failures almost always involve pieces of rock that are less than 2 ft thick. About 40% of the 669 roof skin injuries in 1997 involved roof bolt operators and occurred beneath temporary support. The other roof skin injuries occurred beneath permanent support and involved workers in a wide variety of activities. Common roof skin control techniques include oversized plates, header boards, wood planks, steel straps, mesh, and (in rare instances) spray coatings (sealants).

RIB FAILURES

During 1996-99, rib failures resulted in seven fatalities, or 14% of all groundfall fatalities, as shown in table 8. Only one

Table 8.—Hazards associated with fatal groundfalls, 1996-99

Hazard	No. of fatalities			Percentage		
	Primary	Secondary[1]	Total	Primary	Secondary[1]	Total
Geologic	1	10	11	2	20	22
Roof skin	6	0	6	12	0	12
Rib	7	2	9	14	4	18
Pillaring	12	2	14	24	4	29
Inby unsupported roof	12	0	12	24	0	24
Intersection	3	5	8	6	10	16
Longwall face	4	0	4	8	0	8
Construction	4	3	7	8	6	14
Total	49	22	71	100	44	—

[1]A secondary hazard was assigned to some groundfall fatalities, but not in all cases.

of these fatal injuries was to a face worker, while five were mechanics and electricians performing their duties well outby the face. Nearly 80% of the 128 rib injuries that occurred in 1997 took place beneath permanently supported roof. Nonfatal rib injuries resulted in an average of 43 lost workdays each versus 25 days for the average roof skin injury.

Seam height is the single greatest factor contributing to rib failures. The seam height was >8 ft in all six of the fatalities and >10 ft in three of them. The incidence of rib injuries increases dramatically once the seam height reaches 7 ft. No rib support was used in any of the six fatal accidents.

Rib failure is often associated with rock partings and/or discontinuities within the pillar or with overhanging brows created by roof drawrock. The most effective rib supports use full planks or mesh held in place by roof bolts.

PILLAR RECOVERY

The process of pillar recovery removes the main support for the overburden and allows the ground to cave. As a result, the pillar line is an extremely dynamic and highly stressed environment. Safety depends on controlling the caving through proper extraction sequencing and roof support. In some mines, mobile roof supports have replaced timber supports for each stage of pillar recovery.

According to table 8, pillaring fatalities are directly attributed to 24% of all groundfall fatalities, including three multiple incidents. However, a recent study estimated that pillar recovery accounts for only 10% of the coal mined underground [Mark et al. 1997]. Nearly 50% of these fatal pillaring incidents involved geologic discontinuities, such as slips and slickensides. Even mines that used additional support for these discontinuities were unable to prevent the massive roof failures. These failures often occur suddenly with little warning and result in collapses where MSHA is unable to find any violations. During 1987-96, Mark et al. [1997] found that almost 50% of the pillaring fatalities occurred during the recovery of the final lift or pushout. Since 1996, however, only 20%, or three incidents, involved last-lift incidents. Several incidents also involved situations where excessive cuts were taken, which caused the large exposed intersection to collapse.

Pillar recovery can also be difficult under deep cover. During 1996-98, nearly one-half of the pillar recovery fatalities occurred where the depth of cover exceeded 650 ft. Under deep cover, barrier pillars and special mining sequences may be required.

GEOLOGIC DISCONTINUITIES

Geologic features such as slips, slickensides, clay veins, kettlebottoms, and ancient stream channels have been closely linked with many groundfall fatalities. These hazardous geologic structures are found predominantly in the Appalachian coal basins [Chase 1992]. Geologic discontinuities were mostly identified as the secondary contributing causes of failure in 20% of all groundfall fatalities (table 8).

Slickensides and slips were found to be the primary and secondary causes of failure of four similar massive groundfalls in 1997 that resulted in four fatalities and seven injuries. All of the falls were so massive that they overran the permanent support, resulting in the collapse of the bolted intersection. Two of the occurrences were attributed primarily to pillaring, which triggered the unstable slips and slickensided joints to collapse. Special precautions need to be taken near the outcrop, where the presence of groundwater and weathered joints (sometimes called hill seams) can reduce roof competence. In general, pillar recovery should not be conducted when the distance to the outcrop is <150 ft.

LONGWALL FACES

The longwall system of mining, which extracts immense coal panels and allows the roof to cave behind the face, presents a unique ground control situation. The total extraction of the coal causes high stress concentrations along the face and in the gate roads and may pose severe ground control problems depending on the competency of the immediate and main roof rock and the sizing of the gate road pillars [Listak and Pappas 1990].

According to figure 8, approximately 8% of groundfall deaths are associated with longwall face mining. Two incidents involved a similar work activity of installing wire mesh in preparation for recovering the longwall face equipment. Usually, a large redistribution of stresses occurs as the longwall face approaches the recovery room, which may weaken the roof directly above the face. In the first incident, a slickensided piece of top coal fell from the face and struck the victim located under the shield supports. In the second case, a piece of binder rock fell from the face, hitting the victim located between the pan line and the longwall face.

CONSTRUCTION

Construction relates to any type of outby mining or resupport of the coal or roof strata. Examples include cutting the roof higher to install an overcast or belt line (boom hole) or rehabilitating roof fall areas.

It seems unlikely that these types of incidents would happen with multiple frequency. However, during the study period seven construction fatalities, or 14% of groundfall fatalities, occurred as either a primary or secondary cause (table 8). Four of the construction incidents were related to boom holes, two to overcast construction, and one to rehabilitating a high roof fall area. Several of the boom hole and overcast incidents occurred because the ATRS was not of sufficient height to support the roof. This resulted in the use of other temporary support methods where the procedure was not properly followed. Also, two incidents occurred while the victim went under unsupported roof following boom hole shots. There seems to be some confusion concerning the proper procedure in supporting boom holes. The rehabilitation incident occurred while installing steel arches. The victim was under the last arch that had been installed and slid a mud sill under the unsupported roof when the roof fell. The fall struck the inby edge of the last arch, then toppled under the arches.

INTERSECTION STABILITY

Thousands of intersections are driven each year and create diagonal spans of 25-40 ft, well over the normal width of an entry. The hazards of wide spans can increase when pillar corners are rounded for machine travel (turnouts) or when rib spalling increases the span. According to table 8, 6% of roof fall fatalities are primarily caused by oversized intersections and 10% are a secondary cause. In 1996, there were 2,105 non-injury reportable roof falls. More than 71% of these occurred in intersections despite the fact that intersections probably account for less than 25% of all drivage underground.

Intersection spans are often measured as the sum of the diagonals. Because the rock load increases in proportion to the cube of the span, even a small increase in the span can greatly reduce the stability of an intersection. For example, widening the entry from 18 to 20 ft increases the rock load from 96 to 132 tons. A study at a mine in western Pennsylvania found that 83% of the roof falls occurred in 13% of the intersections where the sum of the diagonals exceeded 70 ft [Molinda et al. 1998].

Many roof control plans specify the maximum spans that are allowed. Mining sequences can also be designed to limit the number, location, and size of turnouts and to restrict turnouts to specific entries. Extra primary support, such as longer roof bolts, installed within intersections can also be very effective in reducing the likelihood of roof falls. On the other hand, replacing four-way intersections with three-ways may be not be an effective control technique. Three-way intersections are more stable, but since it normally takes two three-way intersections to replace one four-way intersections, the total number of falls is likely to increase [Molinda et al. 1998].

CONCLUSIONS

The effects of groundfall incidents are extensive, ranging from the economic loss of equipment and production to fatal and nonfatal injuries that result in lasting physical and financial impairments suffered by the victims and the victim's family. In addition, the mining industry is severely impacted by these injuries, as well as thousands of noninjuries that damage equipment, stop production, or disrupt ventilation. This study of roof and rib fall injuries and noninjury rates controlled for mining method, seam height, and mine size, and resulted in the identification of the following incident trends:

- The longwall mining method results in less than one-half the roof fall injury rate compared to than the room-and-pillar method. However, the rib injury rate for both mining methods is nearly identical.
- Longwall mining accounts for 48% of the production and 40% of the hours worked, but results in only 22% of the roof fall injuries.
- Longwall mines in western Kentucky and Illinois/Indiana have significantly higher roof fall rates. Northern West Virginia/Ohio/Maryland, Virginia, and the western United States have significantly lower roof fall rates.
- Room-and-pillar mines in western Kentucky have a very high noninjury roof fall rate.
- For both mining methods, rib fall injury rates are significantly higher in the western United States.
- For noninjury falls in room-and-pillar mines, small mines (<50 workers) and large mines (>149 workers) in thick seams (>60 in) have a significantly higher risk of massive roof falls. Conversely, mines located in thin seams with small- and medium-sized workforces have a significantly lower massive fall rate.
- The fatality rate for room-and-pillar mines is very high for small mines in thin seams, but is very low for small mines in thick seams.
- Room-and-pillar mines in small- and medium-sized mines in thin seams have a significantly lower rib fall rate. Small- and medium-sized mines in thick seams have a significantly higher rate.
- For coalbeds in which both methods are used, severe groundfall rates were identified in the Illinois Basin, especially for the Kentucky No. 13, Herrin/No. 6/Kentucky No. 11, and Springfield No. 5/Kentucky No. 9 Coalbeds.
- For room-and-pillar coalbeds, northern Appalachian coalbeds, most notably the Sewickley, Redstone, Pittsburgh, and Bakerstown, have severe roof fall rates. Many coalbeds in the central Appalachian region have high roof fall rates, especially Bens Creek/Blair, Upper Banner, Jawbone/Iaeger, and Walnut Mountain.
- Longwall coalbeds in the central Appalachian coalfields with very high groundfall rates include the Eagle and Hazard No. 4.
- Severe rib fall rates were found for several coalbeds in the western United States. Difficult room-and-pillar coalbeds include the D Seam and B Seam. Problematic longwall coalbeds include the Upper Hiawatha, Hiawatha, Blind Canyon, Wadge and B Seam.

• Severe room-and-pillar rib fall rates were found in coalbeds in the central Appalachian region, including the Pocahontas No. 12, Eagle, Peerless, Powellton, and Amburgy/ Low Splint Coalbeds.

• A review of seasonal patterns revealed that the third quarter (July to September) has a 30%-40% increased risk of injury and noninjury groundfalls. This may be due to higher humidity levels.

To better understand why these groundfalls occurred, all of the detailed fatality reports were analyzed, and the following groundfall hazards contributing to the groundfall were identified:

• Pillaring is the leading cause of fatal groundfall failures. Many of these incidents were triggered by geologic discontinuities present in the roof strata, such as slips and slickensides. These failures often occurred suddenly and with little warning.

• A prevailing factor contributing to groundfall fatalities was miners going under unsupported roof. In recent years, fatalities from this risky activity has dropped significantly; however, it still contributes to 24% of all groundfall fatalities.

• Rib failure is a major hazard associated with groundfall fatalities and is often associated with rock partings and/or discontinuities within the pillar or with overhanging brows created by roof drawrock. None of the mines where rib fatalities occurred used any type of rib mesh or bolting.

• Roof skin failures are of particular concern because they caused 12% of groundfall fatalities and many nonfatal injuries to workers who should have been protected by supports.

• Construction-related groundfalls were associated with 14% of the primary or secondary causes of the fatalities. Several boom hole and overcast incidents occurred because the ATRS was not of sufficient height to support the roof.

• Several longwall-mining-related groundfall fatalities resulted during installation of wire mesh as the longwall face approached the recovery room.

These groundfall statistical characteristics and fatality report trends offer the most current profile of roof and rib falls in the United States. This study identifies areas where additional research is needed so that innovative solutions can be developed to reduce these severe hazards to underground coal mine workers.

REFERENCES

Bauer ER, Pappas DM, Dolinar DR, McCall FE, Babich DR [1999]. Skin failure of roof and rib in underground coal mines. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 108-114.

Chase FE [1992]. Geologic structures that affect Appalachian coal mines. In: Proceedings of the Preventing Coal Mine Groundfall Accidents; How To Identify and Respond to Geologic Hazards and Prevent Unsafe Worker Behavior. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9332, pp. 3-14.

Fotta B, Mallett LG [1997]. Effect of mining height on injury rates in U.S. underground nonlongwall bituminous coal mines. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9447.

Fotta B, Turin FC, Murphy JN [1995]. Appalachian coal-mining health and safety trends. Min Eng *47*(12):1115-1119.

Listak JM, Pappas DM [1990]. Longwall automation: a ground control perspective. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9244.

Mallett LG, Vaught C, Peters RH [1992]. Training that encourages miners to avoid unsupported roof. In: Preventing Coal Mine Groundfall Accidents; How To Identify and Respond to Geologic Hazards and Prevent Unsafe Worker Behavior. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9332, pp. 32-45.

Mark C, McCall FE, Pappas DM [1997]. A statistical overview of retreat mining of coal pillars in the U.S. In: Peng SS, ed. Proceedings of the 16th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 204-210.

Molinda GC, Bauer ER, Babich DR, Pappas DM [1998]. Factors influencing intersection stability in U.S. coal mines. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 267-275.

Pappas DM [1987]. Roof and rib fall accident and cost statistics: an in-depth study. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9151.

Peters RH [1992]. Miners views on how to prevent people from going under unsupported roof. In: Preventing Coal Mine Groundfall Accidents; How To Identify and Respond to Geologic Hazards and Prevent Unsafe Worker Behavior. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, RI 9332, pp. 25-31.

Peters RH, Randolph RF [1991]. Miners' views about why people go under unsupported roof and how to stop them. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9300.

Stateham RM, Radcliffe DE [1978]. A cyclic effect in coal mine roof stability. Washington, DC: U.S. Department of the Interior, Bureau of Mines, RI 8291.

FUNDAMENTALS OF COAL MINE ROOF SUPPORT

By Christopher Mark, Ph.D.,[1] and Thomas M. Barczak[2]

ABSTRACT

Roof supports can only be understood in conjunction with the rock structure that they support. The strength of the rock depends on geology, and the loads are applied primarily by the in situ and mining-induced stresses. Other factors, such as wider spans and retreat or multiple-seam mining, can also reduce the stability of mine openings. Roof supports are used to help stabilize these openings, but their performance characteristics must be properly matched to the loading environment and ground behavior if they are to succeed. Roof supports include both intrinsic supports, such as roof bolts, and standing supports. The key characteristics of any support include its maximum load-carrying capacity, stiffness, and residual strength. Other important factors are the timing of installation, the stability of the support as it is loaded, and the capability of the support system to provide skin control. This paper explains in practical terms how supports work and the important factors in ensuring that a good support design and application strategy are developed.

[1]Supervisory physical scientist.
[2]Research physicist.
Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

Roof support is essential to the safety of every underground miner. It has three primary functions:

- To prevent major collapses of the mine roof;
- To protect miners from small rock falls that can occur from the immediate roof skin; and
- To control deformations so that mine openings remain serviceable for both access and escape, as well as for ventilation of the mine workings.

Roof supports interact with the ground to create a stable rock structure. With any structure, an engineering analysis begins with evaluations of two fundamental factors:

- The strength of the different components of the structure; and
- The forces that are loading it.

Rock structures are unique in that the strength of one essential component, the rock itself, can seldom be determined accurately. Similarly, the ground stresses are rarely well understood. Ground control engineers have had to develop novel techniques to compensate for these deficiencies.

This paper begins with a summary of the factors affecting the integrity of mine roof structures. Next, it discusses the function and properties of mine roof support. It concludes with a framework for understanding how the supports and the ground interact with each other to provide a stable mine opening.

FACTORS AFFECTING THE INTEGRITY OF MINE STRUCTURES

An assessment of the integrity of any mine structure must begin with an analysis of (1) the structural integrity and strength of the roof rock, (2) the excavation geometry, and (3) the forces applied to the mine roof.

ROCK STRENGTH

Rock strength traditionally is estimated from laboratory tests. The uniaxial compressive test is the most commonly used. Figure 1 shows the approximate range of compressive strengths observed in U.S. coal measure rock. Triaxial tests, where the rock is confined, more accurately simulate the three-dimensional stress that rock typically encounters underground. Shear tests of bedding planes can be very helpful in evaluating the likelihood of slip, but are rarely performed in the United States. These three types of tests are shown in figure 2.

Rock tests are severely limited in that they are conducted on small samples of intact rock. The strength of the rock mass in mine roof is, however, determined largely by the presence of cracks, bedding planes, and other natural discontinuities. Rock mass classification systems were developed to help quantify their effects.

The Coal Mine Roof Rating (CMRR) focuses on the specific features that commonly occur in coal measure rock. It weighs the individual geotechnical factors that determine roof competence, including—

- The uniaxial compressive strength of the intact rock;
- The spacing and persistence of discontinuities like bedding planes and slickensides;
- The cohesion and roughness of the discontinuities; and
- The presence of ground water and the moisture sensitivity of the rock.

Simple index tests and observations are used to rate each of these parameters, which are then combined into a single rating on a scale from 0 to 100.

The CMRR can be calculated from underground exposures like roof falls and overcasts [Molinda and Mark 1994] or from exploratory drill core [Mark and Molinda 1996]. In the case of drill core, point load tests are used to estimate the compressive strength and the cohesion. A computer program is currently being developed to aid in the collection, interpretation, and presentation of CMRR data.

The CMRR incorporates most of the geologic factors that affect the mine roof. It does not address large-scale features, like faults, sandstone channel margins, or igneous dikes. Such features may cause major disruptions in relatively small areas and should be treated individually.

CMRR values have been obtained from hundreds of coal mines throughout the United States and abroad. Figure 3 shows that the northern Appalachian coalfields typically have the weakest roof in the United States; the strongest roof is found in Utah. Ground conditions and roof bolt densities from three major coal mining countries are compared in figure 4 [Mark 1999b]. Roof bolt design guidelines are presented elsewhere in these Proceedings [Mark 2000].

ROOF SPAN

In underground coal mining, the excavation geometry does not vary much, but the span can be very important. The basic principle that governs the relationship between stability and the span was first formulated by Austrian tunneling engineers [Bieniawski 1989]:

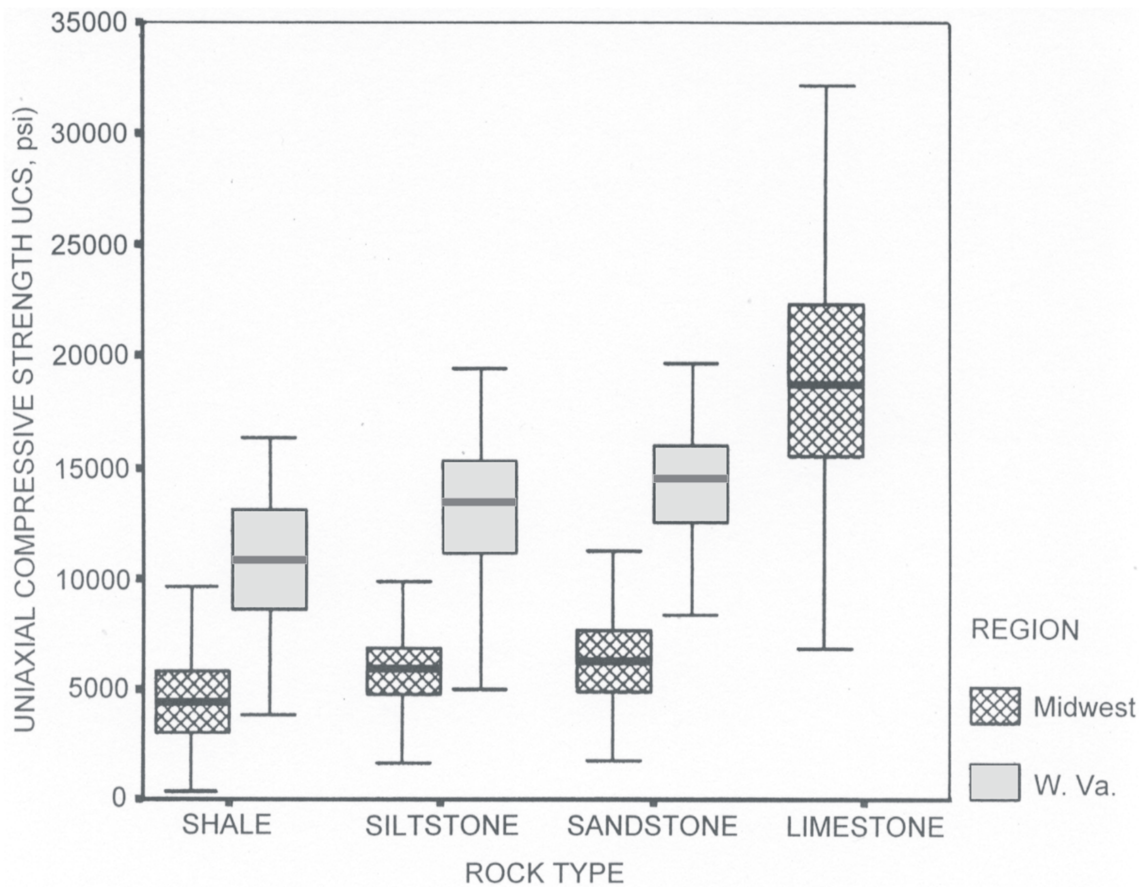


Figure 1.—Range of compressive strength for U.S. coal measure rocks [Rusnak and Mark 2000].

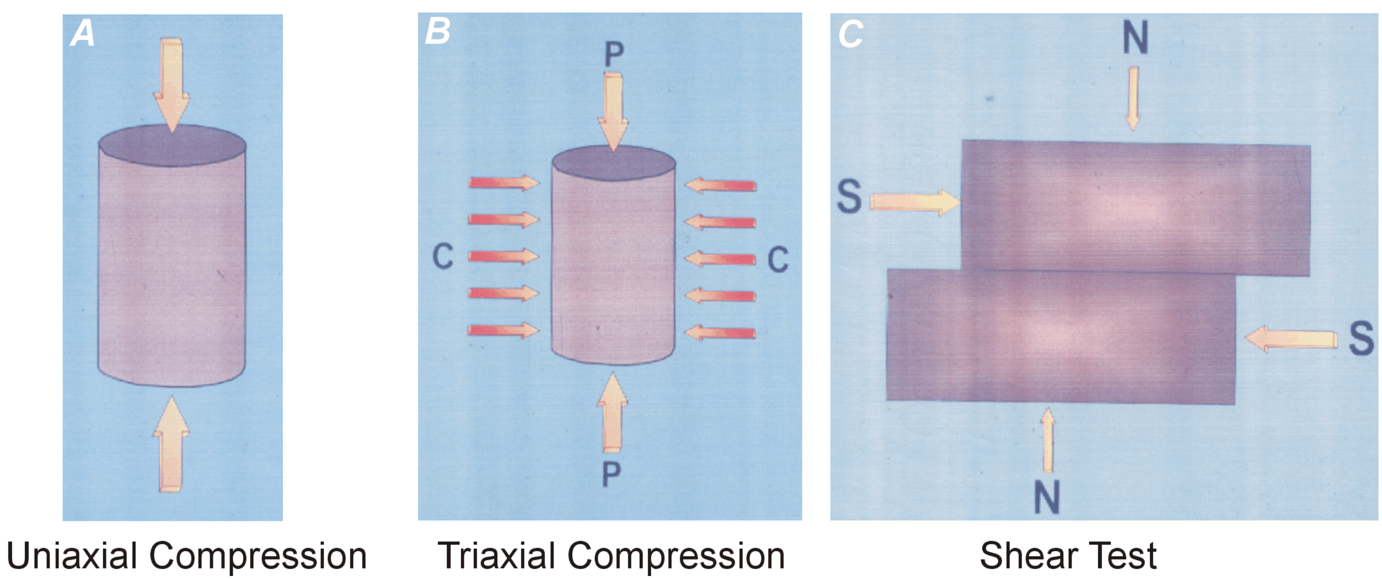


Figure 2.—Three types of laboratory strength tests. *A*, uniaxial compressive strength test; *B*, triaxial compressive strength test; *C*, bedding plane shear test.

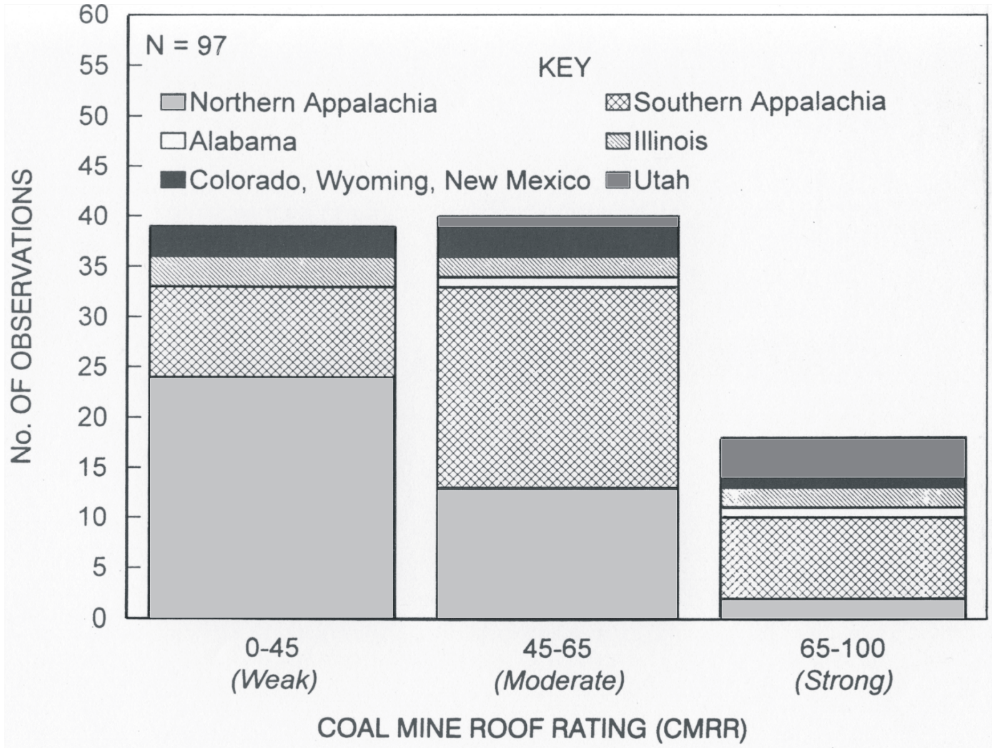


Figure 3.—Range of Coal Mine Roof Ratings (CMRR) observed in the United States [Molinda and Mark 1994].

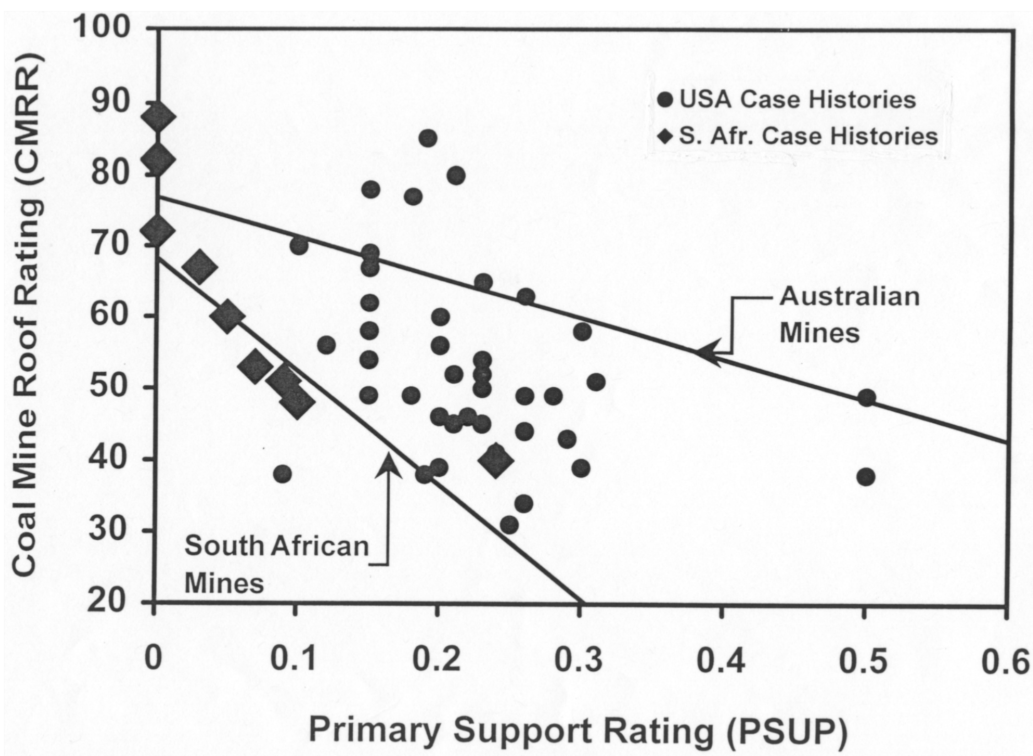


Figure 4.—Roof bolt densities observed in three coal mining countries [Mark 1999b].

- For a given rock mass, a tunnel's standup time decreases as the roof span becomes wider; and
- For a given roof span, a tunnel's standup time decreases as the rock mass quality becomes poorer.

The greatest spans in coal mines are encountered in intersections. While entries are normally limited to 6 m (20 ft), the diagonal spans of intersections are generally in the 7.5-12 m (25-40 ft) range. Approximately 70% of all roof falls occur in intersections, although intersections only account for about 20% to 25% of all drivage. Roof falls are therefore 8 to 10 times more likely to occur in intersections than in an equivalent length of entry [Molinda et al. 1991].

A study by Mark [1999a] looked at standup time during extended (deep) cut mining, where the continuous miner advances the face more than 20 ft beyond the last row of permanent supports. At 36 mines, it was found that when the CMRR was >55, the roof was stable in nearly every case. When the CMRR was <37, the roof collapsed before the cut could be completed. When the CMRR was between 38 and 55, extended cuts were feasible some times, but not others (figure 5). The data also show that extended cuts are less likely to be stable if either the entry span or the depth of cover is increased.

Many studies have documented the effect of roof span on stability. The longwall study cited earlier found a strong correlation between entry width and CMRR (figure 6) [Mark and Chase 1994]. The relationship between intersection span and the incidence of roof falls at six mines was documented by Mark et al. [1994] (figure 7). A similar correlation is reported by Molinda et al. [2000].

FORCES APPLIED TO THE COAL MINE ROOF

Stress is everywhere underground (figure 8). Usually, the external forces applied to rock are all compressive, but they are not equal in all directions. The in situ stresses are normally resolved into three components: (1) vertical stress, (2) the maximum horizontal stress, and (3) the minimum horizontal stress.

Vertical Loads

The most obvious source of loads on mine structures is the weight of the rock itself. It is convenient to analyze two types of vertical loads (figure 9):

- The *roof load*, which is due to the weight of the immediate roof strata as they sag into the mine opening; and
- The *pillar loads*, which are applied by the weight of the overburden.

The roof load is the vertical force that most directly applies to roof support. Various "dead weight" design methods are based on estimating the volume of immediate roof rock that has separated from the more stable overlying rock mass and consequently must be supported [Mark 2000].

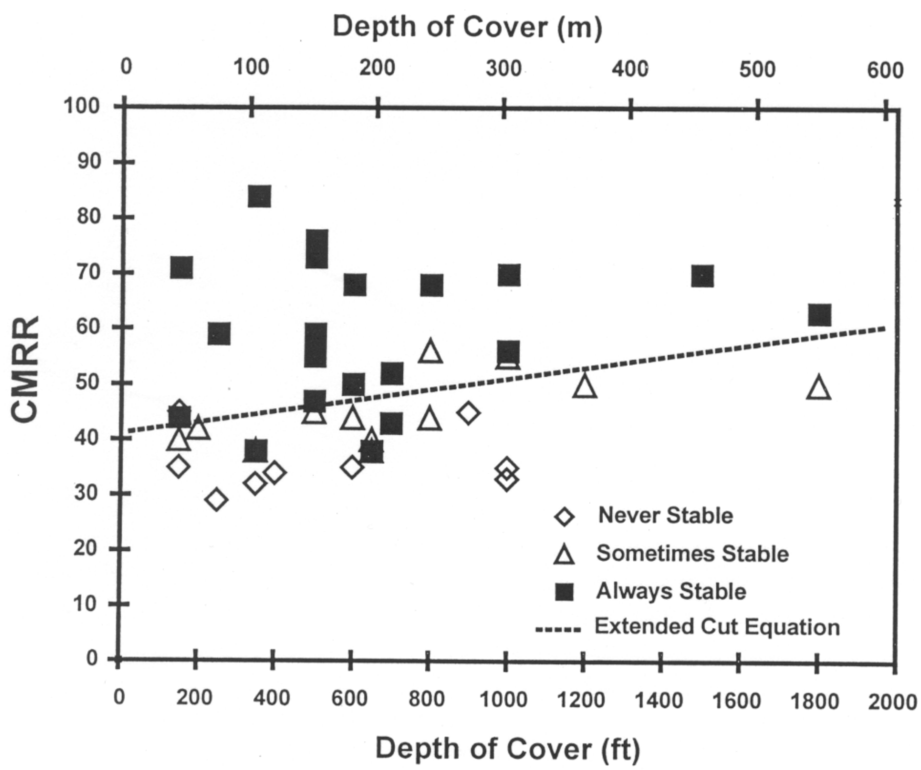


Figure 5.—Relationship between CMRR, depth of cover, and the stability of extended cuts [Mark 1999a].

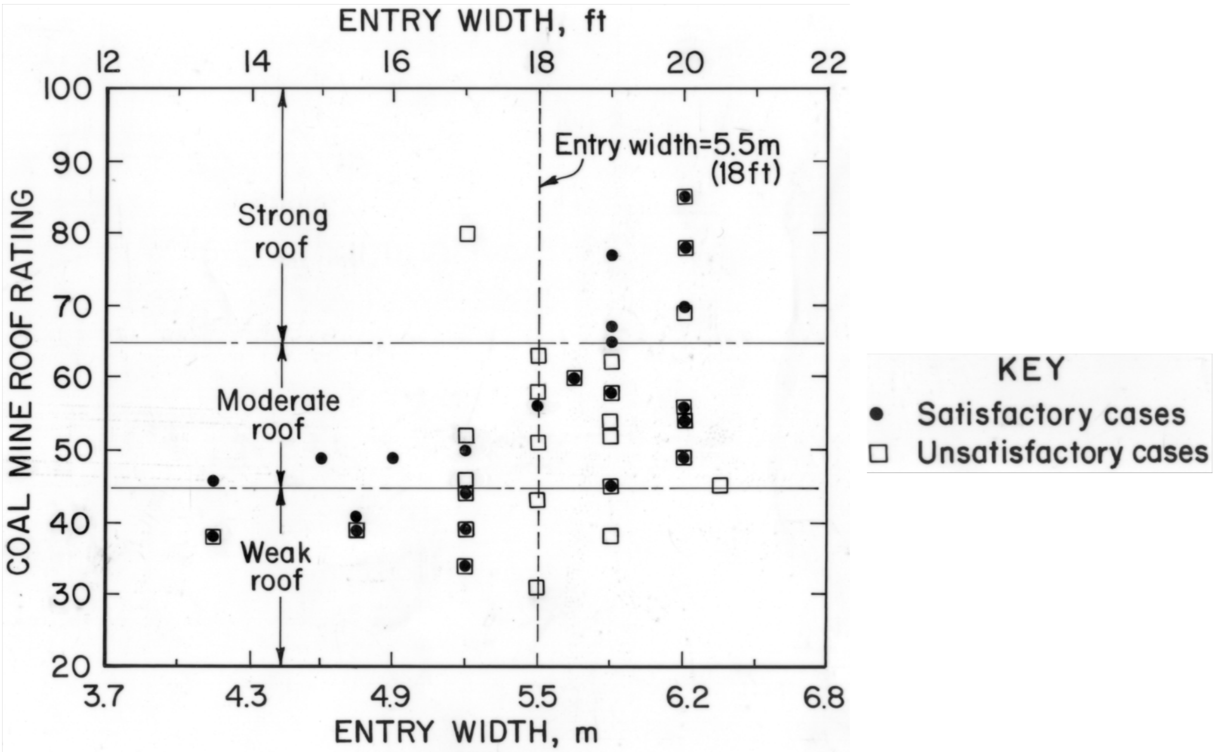


Figure 6.—Entry widths and CMRR in U.S. longwall mines [Mark and Chase 1994].

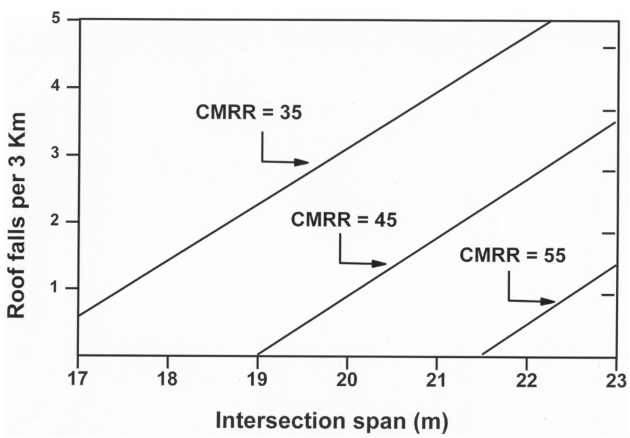


Figure 7.—Relationship between CMRR, intersection span, and roof fall rate at six U.S. mines [Mark et al. 1994].

The overburden load, on the other hand, is primarily carried by the pillars, but it can affect the immediate roof stability (and thus support loading) by—

- Causing sloughage of the pillars, thereby increasing the roof span in the mine entry;
- Excessively loading or yielding the pillars or the mine floor, resulting in differential movements that can damage the immediate roof rock;
- Stressing the pillars, which causes them to squeeze out and apply a horizontal force to the immediate roof rock above the mine entry.

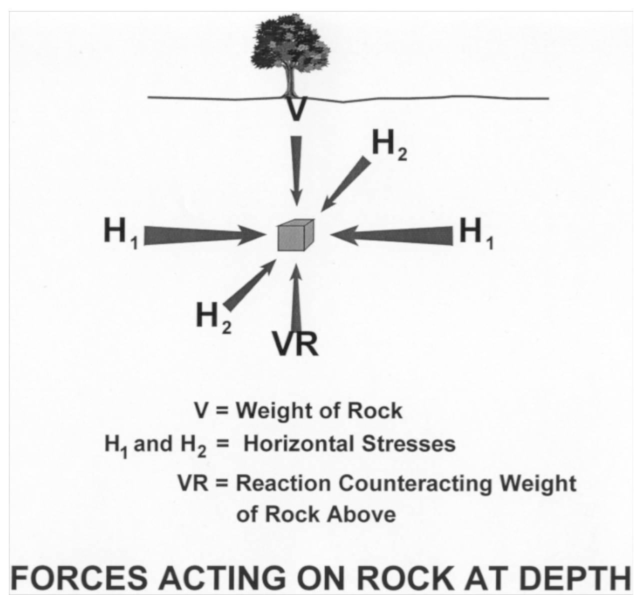


Figure 8.—Stress on a typical element of mine roof.

Horizontal Stress

The horizontal stresses are normally more important to roof stability than the vertical stresses. The reason is that most vertical stress is applied to the pillar, whereas the roof must bear the full brunt of the horizontal stress. Moreover, the magnitude of the horizontal stress is usually greater than the vertical stress. The effects of horizontal stress are—

- Compressive-type roof failures (commonly called cutter roof, guttering, shear, snap top, and pressure cutting). In thinly bedded roof, the failure develops as the progressive layer-by-layer crushing of the individual beds.
- Directional effects, because roof damage is generally much greater in entries oriented perpendicular to the maximum horizontal stress than in entries driven parallel with it.

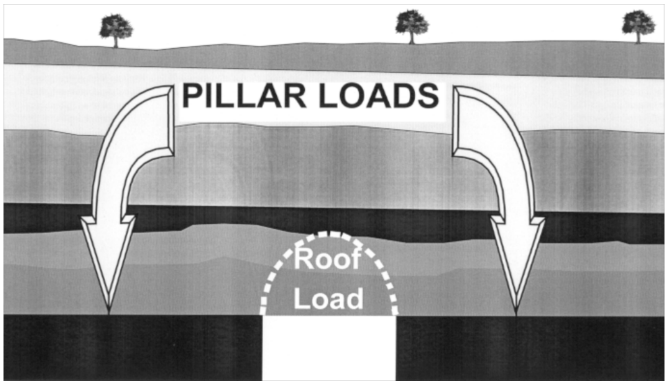


Figure 9.—Vertical loads in underground coal mines.

During the past 15 years, horizontal stress has become central to an understanding of coal mine ground control. An important breakthrough was the recognition that the stresses observed in coal mines are caused by global plate tectonic forces [Mark 1991]. The World Stress Map Project [Zoback and Zoback 1989] identified stress regimes in many parts of the world by analyzing active faults, borehole breakouts, and hydraulic fracturing stress measurements (figure 10).

An evaluation of stress measurements made in underground coal mines confirmed that the stress map applies to underground coal mines [Mark and Mucho 1994]. In the Eastern United States, 76% of the measurements fell within 25° of N. 75° E.

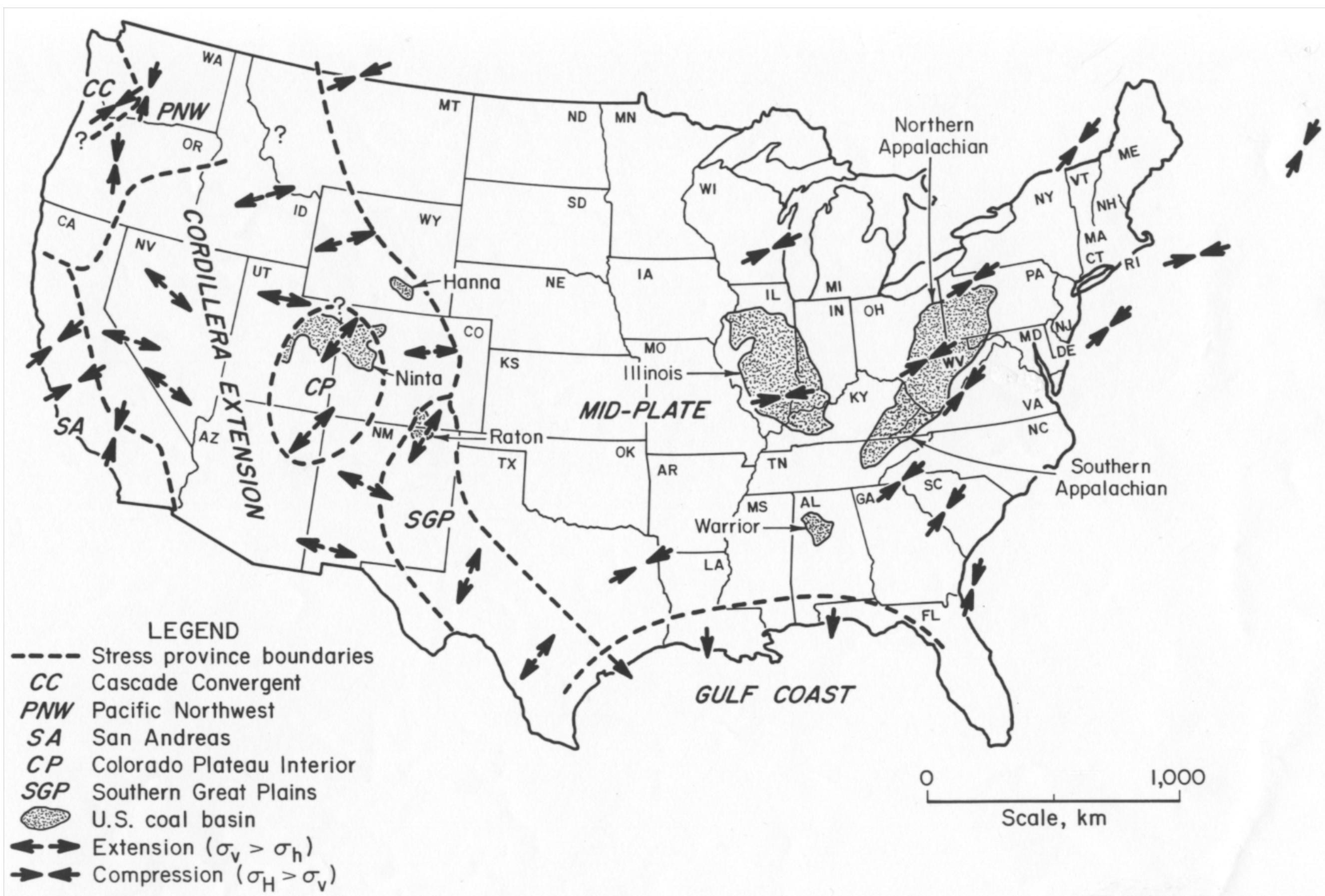


Figure 10.—Stress fields in the continental United States [Zoback and Zoback 1989].

In magnitude, the horizontal stresses were generally two to three times the vertical (figure 11). In the Western United States, there seems to be much more variation from mine to mine and even within individual mines. The horizontal stress is also approximately equal in magnitude to the vertical stress in the West (figure 12). In both sets of measurements, the maximum horizontal stress was usually about 40% greater than the minimum.

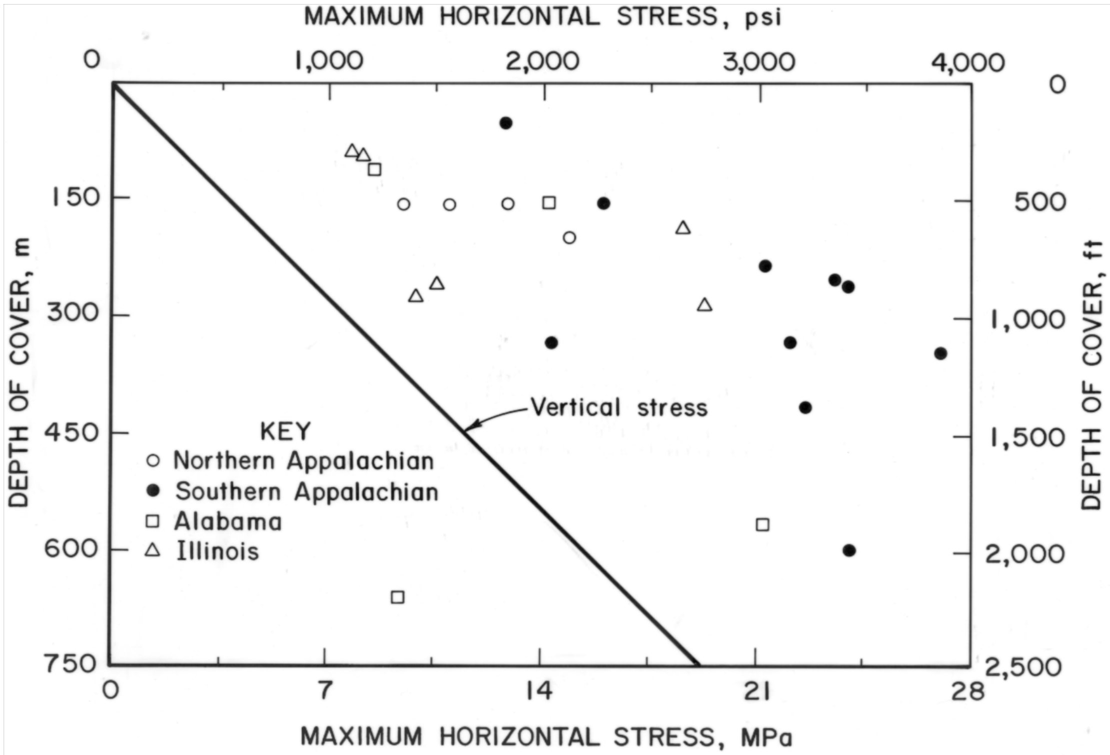


Figure 11.—Horizontal stresses in measured eastern U.S. coal mines [Mark and Mucho 1994].

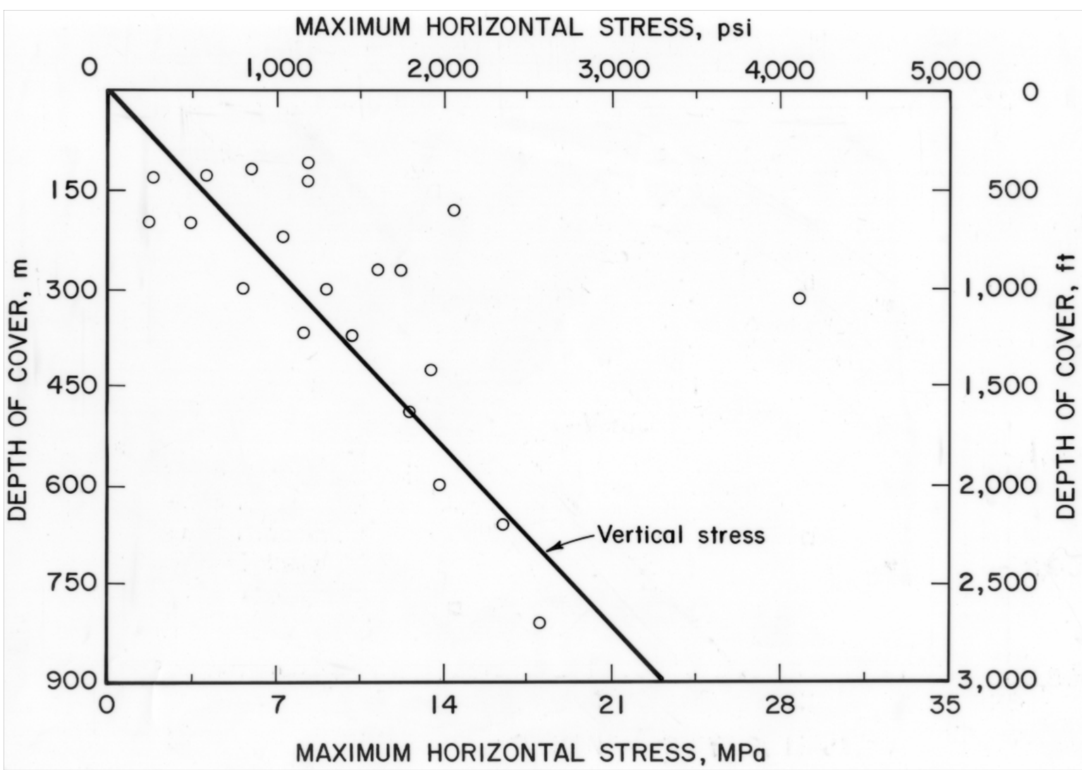


Figure 12.—Horizontal stresses measured in western U.S. coal mines [Mark and Mucho 1994].

Two other factors also determine the degree to which horizontal stress will affect ground control:

• *Roof type:* Weak roof is more likely to suffer damage than strong rock, and laminations or thin bedding (as in shales or stackrock sandstones) greatly reduce the ability of rock to resist horizontal stress.

• *Surface topography:* Stream valleys can concentrate horizontal stresses and have often been associated with particularly difficult horizontal stress conditions. Stream valleys can also reorient the maximum horizontal stress away from the regional direction [Molinda et al. 1991].

Stress measurements are too expensive for most mines to use routinely. As a substitute, procedures have been developed to estimate the orientation of the maximum principal stress [Mucho and Mark 1994; Fabjanczyk 1996]. Such features as roof "guttering" or roof "pots" are mapped underground, and the stress direction is inferred from their orientation and severity.

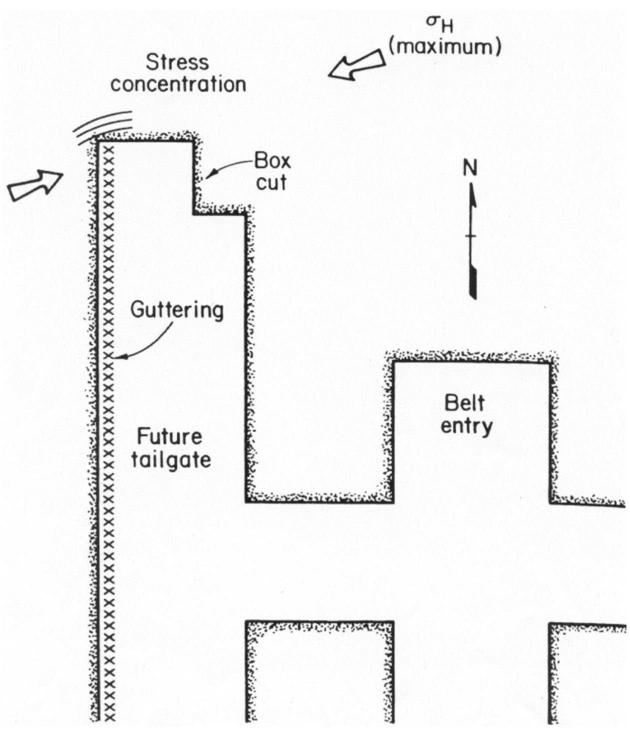


Figure 13.—Horizontal stress concentration in leading entries [Mark and Mucho 1994].

Mining-Induced Stresses

The act of mining can concentrate and reorient the original in situ stresses. Whenever coal is removed, the overburden weight that it had carried is transferred. Vertical stresses are therefore increased on the adjacent unmined coal. Horizontal stresses are similarly affected when the roof is deformed or fails. Horizontal stress cannot pass through broken ground, so it becomes concentrated where the roof is still intact.

• *Development mining:* Entry development creates pillar loads, and "transient stress abutments" have been observed [Karabin et al. 1982]. Horizontal stress creates more serious roof control problems. In some mines, "leading entries" are heavily damaged and require extensive support. Adjacent entries can be stress relieved [Mark and Mucho 1994] (figure 13). Outby the face, a roof fall can create a horizontal stress concentration, which can then propagate itself hundreds of feet.

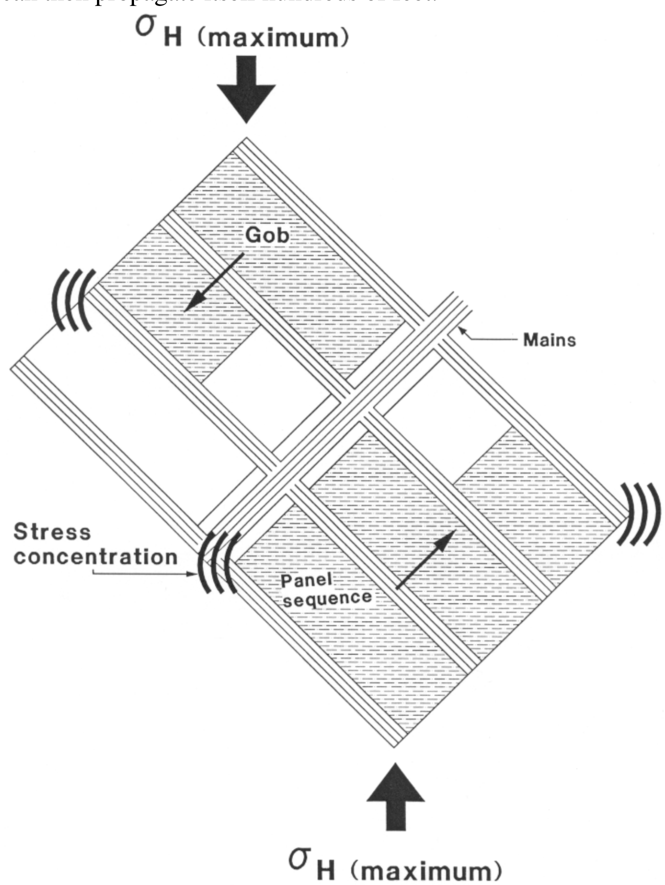


Figure 14.—Horizontal stress concentrations in longwall headgates [Mark and Mucho 1994].

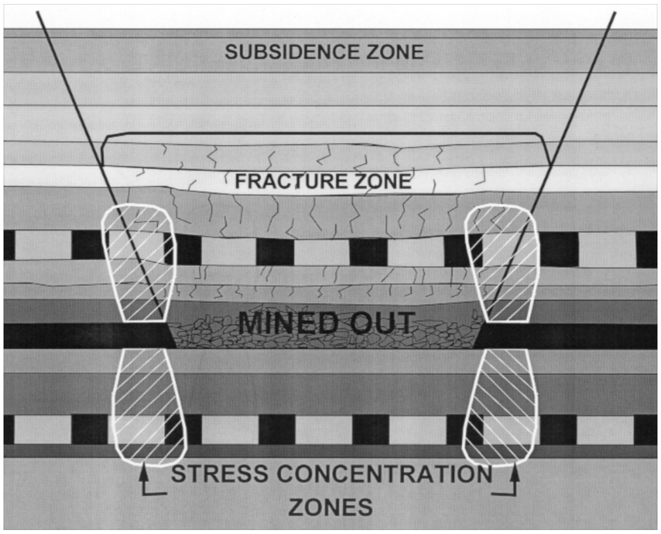


Figure 15.—Multiple-seam mining and its effects on ground control [Mark 1990].

• *Retreat mining:* Longwall mining and pillar recovery can concentrate large vertical loads on gate entries and pillar lines. Proper pillar sizing is essential for limiting the roof stresses and deformations to levels that can be handled by roof support [Mark and Chase 1994, 1997; Colwell et al. 1999]. Secondary support is generally necessary to help control the additional loads. Recently, the importance of horizontal stress abutments has also been documented [Mark et al. 1998] (figure 14). Proper panel layout can greatly reduce the loads applied to the roof.

• *Multiple-seam mining:* Overmining and undermining are responsible for some of the most severe conditions found underground. Both can concentrate vertical loads, and undermining can cause subsidence that damages the roof above overlying coalbeds (figure 15) [Chekan and Listak 1994].

THE FUNCTIONS OF ROOF SUPPORT

Support systems work best when they enhance the inherent strength of the mine roof [Hoek and Wood 1988]. They can do this by—

Providing confinement. Rock is much stronger when it is confined. Since roof rock is usually being loaded by horizontal stress, even a small amount of vertical confinement can have a big effect. The frictional strength of bedding planes may also be strengthened by confinement.

Limiting deformation and preventing unraveling. By maintaining the integrity of the roof line, supports help the upper layers maintain their strength.

Tying weaker rock units to stronger ones. Coal mine roof often consists of several layers of rock with different strengths. Roof bolts are particularly effective in tying weak or broken rock to beds that are more self-supporting.

When the rock is completely broken and has lost all of its strength, supports can also carry the dead-weight load.

PROPERTIES OF ROOF SUPPORT SYSTEMS

Roof supports can be divided into two categories:

• *Intrinsic support*, where the supporting elements are installed within the roof; and
• *Standing support*, where the supporting members are installed between the roof and floor.

Roof bolts are the best example of intrinsic supports. Roof bolts are loaded as the roof deforms, and they interact with the rock to reduce bed separation by confinement much as reinforcing steel does with concrete. Standing supports, like cribs, posts, or longwall shields, develop loads in response to the convergence between the roof and floor.

CAPACITY OF ROOF SUPPORTS

The first question usually asked regarding a support system is: "How much load can the support carry; what is its capacity?" For roof bolts, two types of capacity are normally given: the *yield* and the *ultimate* (figure 16). In general, these can be calculated from the properties of the steel and the diameter of the bolt. However, as discussed by Mark [2000], poor anchorage can substantially reduce the effective capacity of roof bolts.

The capacities of standing supports depend on several factors, including the materials, configuration, and height. In general, the capacity of each particular support type must be determined by controlled load testing. The Pittsburgh Research Laboratory of the National Institute for Occupational Safety and Health (NIOSH) has tested a large number of supports in its unique Mine Roof Simulator. From these tests, the performance characteristics of these various support systems have been determined. By matching the support characteristics to the ground behavior, an optimum support design can be achieved. To facilitate this approach, NIOSH developed the Support Technology Optimization Program (STOP). This program allows the user to determine the optimum installation parameters for any support technology and compare the installation of one support system to another in terms of installed support load density and the convergence control provided by the support [Barczak 2000c].

In many cases, however, the capacity may not be the most meaningful way to define a support. Consider the example shown in figure 17. The second support (support No. 2) has twice the ultimate capacity of the first support, but it takes four times the convergence to reach this capacity. Furthermore, at one unit of displacement, the second support has only one-half the capacity of the first support. Figure 18 is another example of the importance of defining the support capacity in relation to the displacement. Although this support has an ultimate capacity of >1,000 tons, is that really meaningful? Before this capacity is mobilized, nearly 5 ft of convergence must occur. By that time, most entries would be entirely unserviceable. Clearly, a better question is: "How much load can the support carry *at a specified amount of displacement*?" This leads directly to the issue of support *stiffness*.

STIFFNESS OF ROOF SUPPORTS

Stiffness is simply a measure of how quickly a support develops its load-carrying capacity in response to convergence. Stiffness is a measure of performance before a support reaches its maximum capacity. Stiffer supports develop capacity more quickly (with less displacement) than softer supports. The support elements can be thought of as large springs. A softer spring will compress a greater amount to provide the same resisting force as a stiffer spring. A good analogy is to think of a ½-ton and 3/4-ton pickup truck. The 3/4-ton truck has stiffer springs on the bed of the truck. Thus, if these two trucks were placed side by side and each was loaded with a cord of firewood, the bed in the ½-ton truck would be lower than the bed in the 3/4-ton truck (figure 19).

While some roof supports are installed with an initial preload, they all develop their load-carrying capacity only through movement of the roof. This creates a fundamental paradox in roof support design. The roof must deform to mobilize the support capacity, but it is this very movement that the support is trying to prevent. Thus, a critical design issue is the stiffness of the support system.

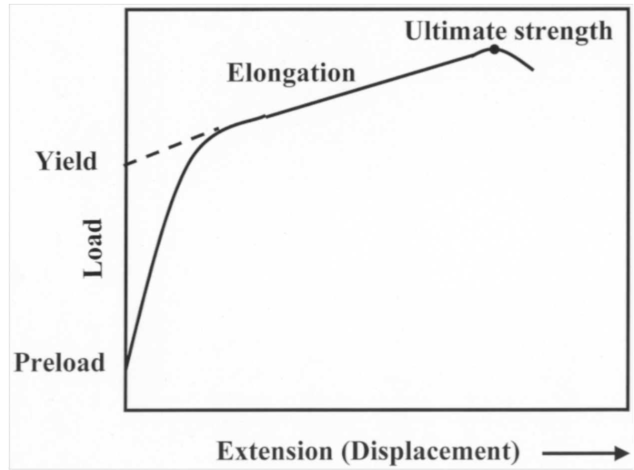


Figure 16.—Yield and ultimate strengths of a roof bolt.

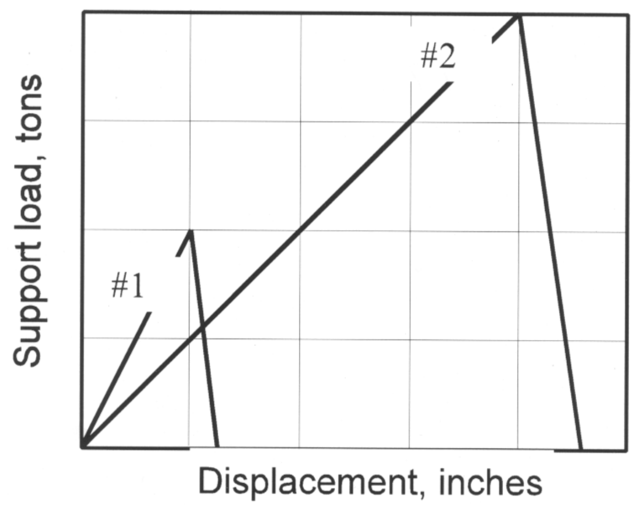


Figure 17.—Example showing that the capacity of a support should be defined in relation to its displacement.

Since stiffness is such an important design parameter for roof supports, let us examine some of the things that impact the stiffness of a support structure. Stiffness (K) is a function of the area (A), material modulus of elasticity (E), and the length or height of the support (L), as expressed in equation 1.

$$K = \frac{A * E}{L} \tag{1}$$

Thus, as seen in equation 1, stiffness increases with area and material modulus and decreases with increasing support height. The significance of these parameters can best be understood by looking at some practical examples.

Intrinsic Support

Let us first examine the implication of these parameters on roof bolt stiffness. First, since roof bolts are made from steel and the modulus of elasticity of steel varies little, the stiffness

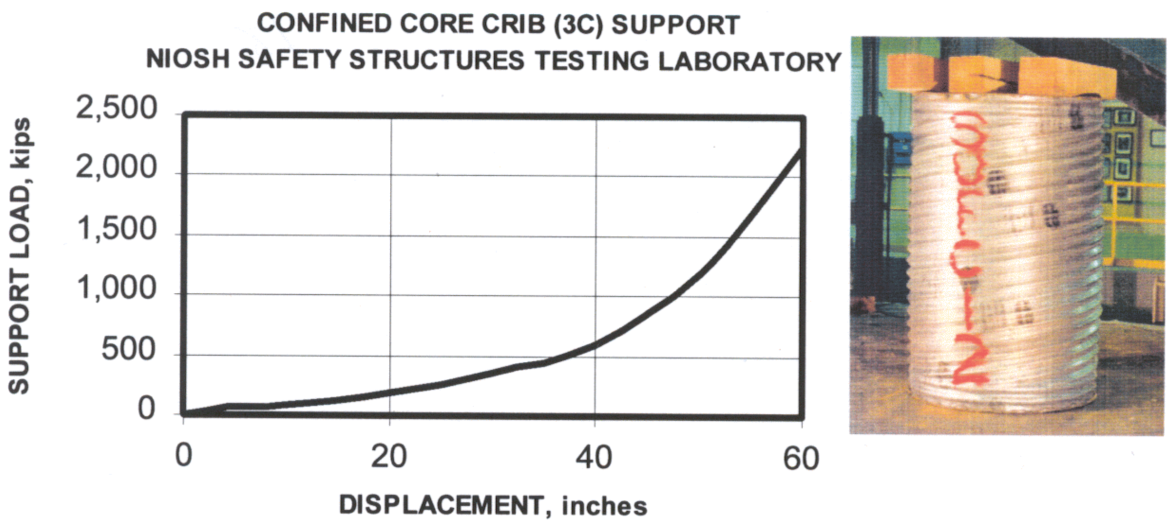


Figure 18.—A support that requires 5 ft of convergence to reach peak load.

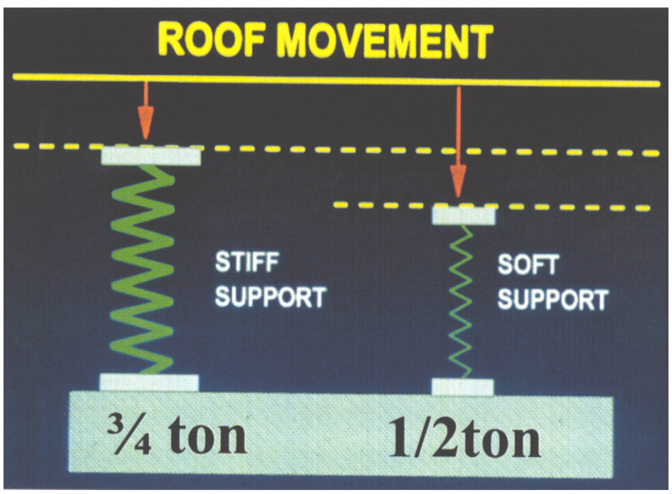


Figure 19.—Pickup truck analogy illustrating support stiffness. The heavy spring in the 3/4-ton truck deflects less than the light spring in the ½-ton truck when both are loaded with the same cord of firewood.

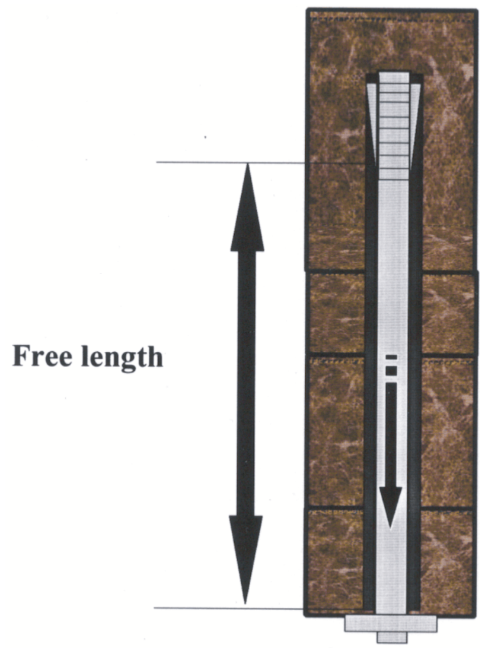


Figure 20.—The free length of a point-anchor roof bolt affects its stiffness.

of roof bolts is not affected by the grade of steel used in fabricating the bolt. However, since the stiffness increases in direct proportion to area or the square of the bolt diameter, bolt stiffness increases dramatically with increasing bolt diameter. Thus, a 7/8-in-diam bolt is twice as stiff as a 5/8-in-diam bolt, all other things being equal.

Bolt length also affects stiffness. With a conventional point-anchor mechanical roof bolt (figure 20), the bolt is anchored only at the top, and the "free length" of the bolt is defined as the length of bolt below the anchor. Thus, as the bolt length increases, the stiffness of the bolt decreases, meaning that longer bolts have a softer response and allow more roof movement to occur for the same increase in bolt load. Fully grouted bolts, on the other hand, do not initially have a "free length" and usually become highly stressed in localized areas in response to roof movements. For this reason, fully grouted bolts are normally considered to be stiffer than point-anchor bolts. Cable bolts and trusses are the least stiff of the intrinsic supports [Dolinar and Martin 2000].

Standing Support

The same principles apply to standing support. Using wood cribs as an example, 9-point cribs are stiffer than 4-point cribs because the timber contact area of a 9-point crib is 2.25 times that of a 4-point crib. Likewise, a 10-in-diam post will have a stiffer response than a 6-in-diam post. Wood cribs can be made stiffer by using different wood species. For example, the elastic modulus of oak is greater than that of poplar wood; thus, oak cribs will be stiffer supports than equivalent cribs constructed from poplar timbers. The stiffness of standing supports is also height-dependent, decreasing with increasing height. For example, a 4-point wood crib constructed from 6×6×30-in, mixed hardwood timbers in a 6-ft seam height will provide 41 tons of support capacity at 2 in of convergence, whereas the same crib design constructed in a 10-ft seam will provide only 32 tons (a 25% reduction) at 2 in of convergence.

Both intrinsic and standing support systems can made stiffer by increasing the density of the supports. An example is shown in figure 21, where two rows of wood cribs are increased to three rows, with the middle row staggered with respect to the two outer rows. Another approach to increase the system stiffness is to reduce the spacing between supports.

Supports can also be softened by adding additional material on top of the support or within the support during its construction. The rule to remember here is that of the weak-link principle—the softest material will control the initial stiffness of the support. The load-displacement response of a concrete crib topped off with a row of wood timbers is shown in figure 22. It is seen in this figure that the wood, which is the softer of the two materials, controls the initial load development of the support. The same principle applies to timber posts where cap boards and/or wedges are used on top of the post. Here, the material may be the same, but wood is much stronger and stiffer when loaded parallel to the grain as in the post section compared to perpendicular to the grain, as would be the case for the cap blocks or wedging material.

RESIDUAL STRENGTH

What happens to a support after it reaches its maximum capacity can be just as important as what happens before. Consider the concrete crib constructed from concrete block typically used in stopping walls and the 24-in-diam Can support shown in figure 23*A*. Both have approximately the same initial stiffness and capacity. However, once the concrete crib reaches its maximum load, it fails completely, leaving the roof entirely

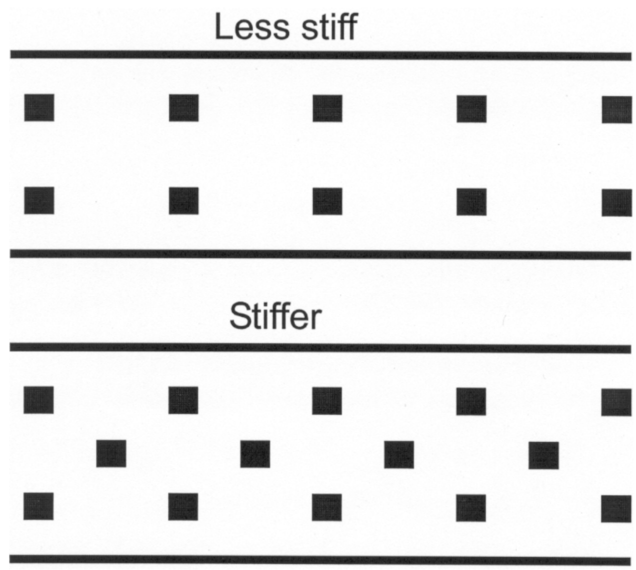
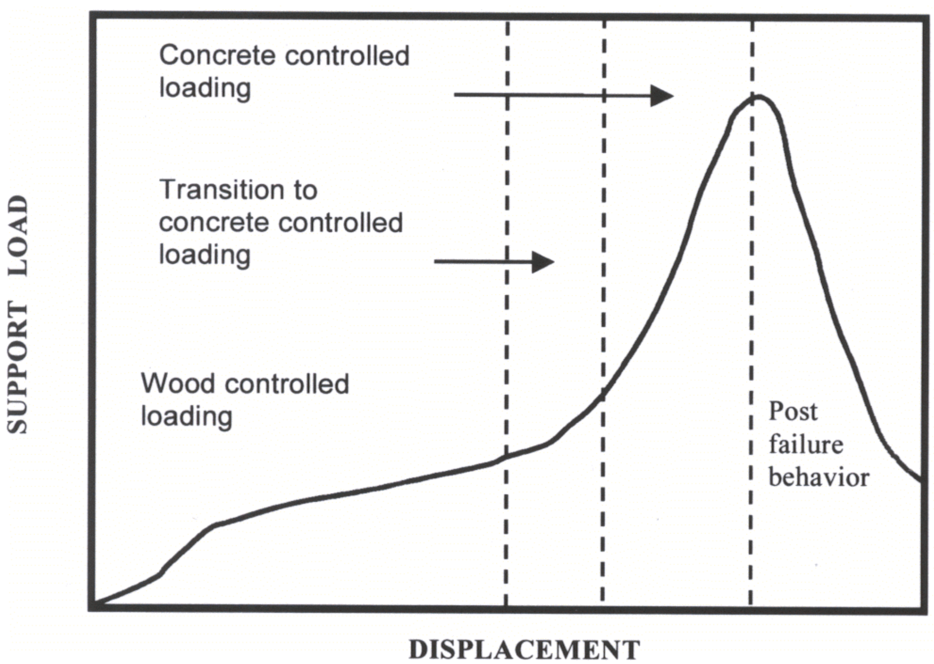


Figure 21.—The stiffness of a wood crib support system is increased by increasing the support density.



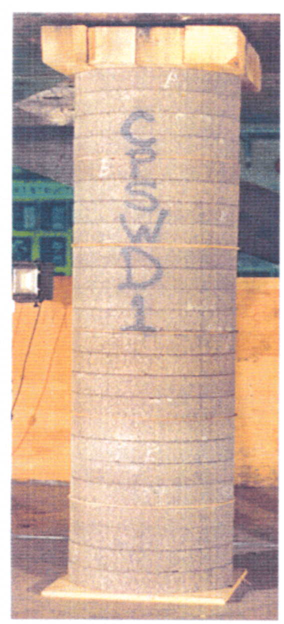


Figure 22.—The stiffness of a concrete crib is reduced by placing wood timbers on top.

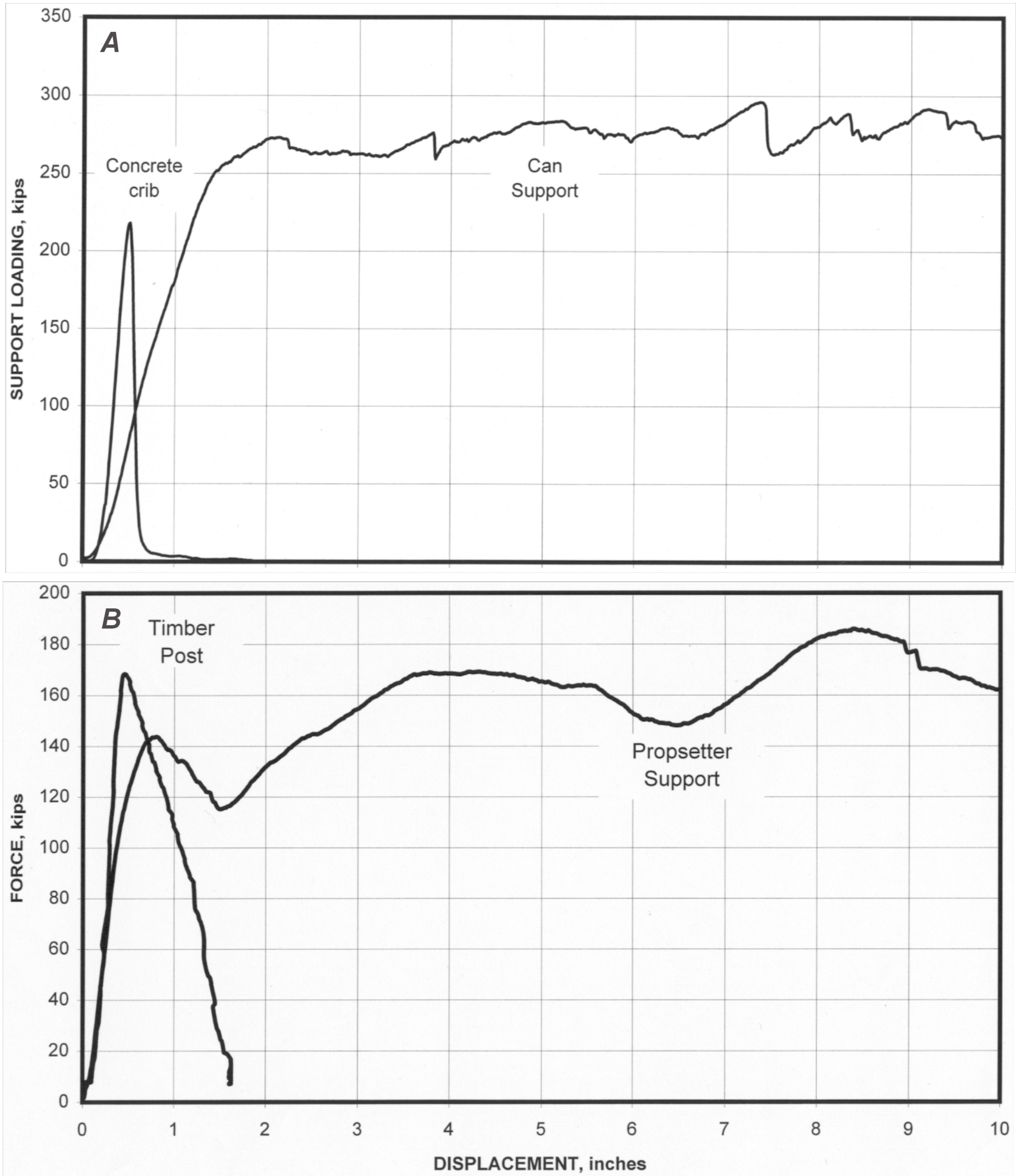


Figure 23.—Residual strength. *A*, The Can has better residual strength than the concrete block crib. *B*, The Propsetter has better residual strength than the timber post. (Note: 1 kip = 1,000 lb).

unsupported. The Can, on the other hand, continues to carry nearly all of its load as the roof continues to move down as much as 2 ft. A similar comparison can be made between a conventional timber post and a Propsetter support (figure 23*B*). The residual strength of supports like the Can and Propsetter make them much more useful in moderate to high convergence such as longwall tailgates than brittle supports like the conventional concrete crib and timber post.

OTHER SUPPORT CHARACTERISTICS

Stability

Stability can be defined as the capability of a support to sustain its load-carrying capacity through a useful range of convergence without failing prematurely. Instability that results in premature failure can be caused in several ways, the most common of which are—

- Buckling, which is common in timber posts and most prop-type supports (figure 24*A*);
- Material failure, where the load applied to the support causes the material to fail in all or part of the support such that the integrity of the support is compromised (figure 24*B*);
- Eccentric loading, which can be caused by wedging of the support in place or uneven roof and floor contact (figure 24*C*); and
- Lateral roof-to-floor loading, usually caused by differential floor heave, which causes the support to lean or tilt off axis (figure 24*D*) [Barczak 2000b].

Material Handling Requirements

Each year, 5,000 workdays are lost by workers in underground coal mines from timber handling injuries alone. In recent years, new support technologies have been developed, including engineered timber support systems, that dramatically reduce the material handling requirements for standing roof support systems [Barczak 2000a].

Installation Quality

In order to get the full benefit of the support, it must be installed properly. Improper installation of support is a major cause of premature support failure. Each support is different, thus the critical parameters for proper installation vary from support to support. Some examples are—

- *Wood cribs:* The performance of wood cribs can be degraded in several ways due to poor installation. For example, the timbers should be overhung to allow the timbers to interlock more effectively, thereby improving the crib stability during loading (see figure 25*A*). Constructing the crib with the wide side of the timber place up will reduce the capacity and degrade the stability of the support. Rounded support timbers will also reduce crib stability and capacity (figure 25*B*). If possible, these timbers should be replaced by square timbers during the construction process. Timbers should also be of consistent quality. One weak or poor-quality timber can severely degrade a 4-point wood crib, since each timber must function to provide the full support capability (figure 25*C*).
- *The Can:* The Can Support is a thin-walled steel container that is prefilled with air-entrained concrete before the unit is transported into the mine. Proper installation requires a layer of good-quality timbers that provides full coverage of the top of the Can to preserve the design load profile. If this is not done, the timbers will not have adequate strength to transfer the loading to the Can; instead, the initial load profile of the support will be unintentionally softened by the wood timber response (figure 26).
- *Roof Bolts:* Obviously, roof bolts depend on proper anchorage to achieve the rated bolt capacity. For grouted bolts, proper mixing and hold time during the bolt installation are critical. Grout performance is affected by several factors, including temperature, age, and conditions of storage.

Timing

Another way to define supports is by the time of installation. *Primary* supports are installed immediately upon development. In the United States, primary supports are almost always roof bolts. *Secondary* supports are placed in anticipation of additional loading, as in a longwall tailgate. *Supplemental* supports are used when the original supports are insufficient.

Skin Control

Skin control is the ability of a support system to prevent injuries from small pieces of falling rock. With roof bolts, skin control may be supplied by plates, headers, straps, or mesh [Bauer and Dolinar 2000]. Skin control is also the reason why many miners would prefer two rows of 4-point wood cribs to a single row of 9-point cribs, even though the load-bearing capacities are nearly the same for both support systems.

Figure 24.—Examples of support instability. *A*, buckling; *B*, material failure; *C*, eccentric loading; *D*, lateral roof-to-floor movement.

Figure 25.—Examples of poor crib construction. *A*, rollout of crib blocks due to inadequate overhang; *B*, rounded timbers degrade support; *C*, a single weak crib block causes premature failure of a "mixed hardwood" crib.

Figure 26.—Poor-quality timber on top of Can degrades support.

SUPPORT AND STRATA INTERACTION

The goal of roof support is to create a stable rock structure. The properties of the roof and the magnitude of the rock stresses determine the quantity of roof support that is required. The support must also withstand the deformation that occurs in the roof.

The concept of the "ground reaction curve" was developed to illustrate the interaction between the load and the roof movement [Scott 1989]. A ground reaction curve may be defined as "the set of possible support loads required to achieve stability for a given roof." The ground reaction curve depends on the rock mass quality, the span, the in situ stress, and the mining-induced stress. A change in any of these variables can cause the ground reaction curve to shift, thereby increasing or decreasing the support load required (figure 27).

The ground reaction curve forcefully shows that deformation, as well as load, is critical to proper roof support design. The importance of support characteristics can be illustrated using the ground reaction curve. If the support is too soft, it may not be able to develop the necessary support capacity to prevent excessive deformation from occurring (figure 28). A support with little residual strength may fail prematurely if the curve shifts because of additional mining stresses. Mucho et al. [1999] describe how a tailgate ground reaction curve can be measured and used to select the proper support density for a

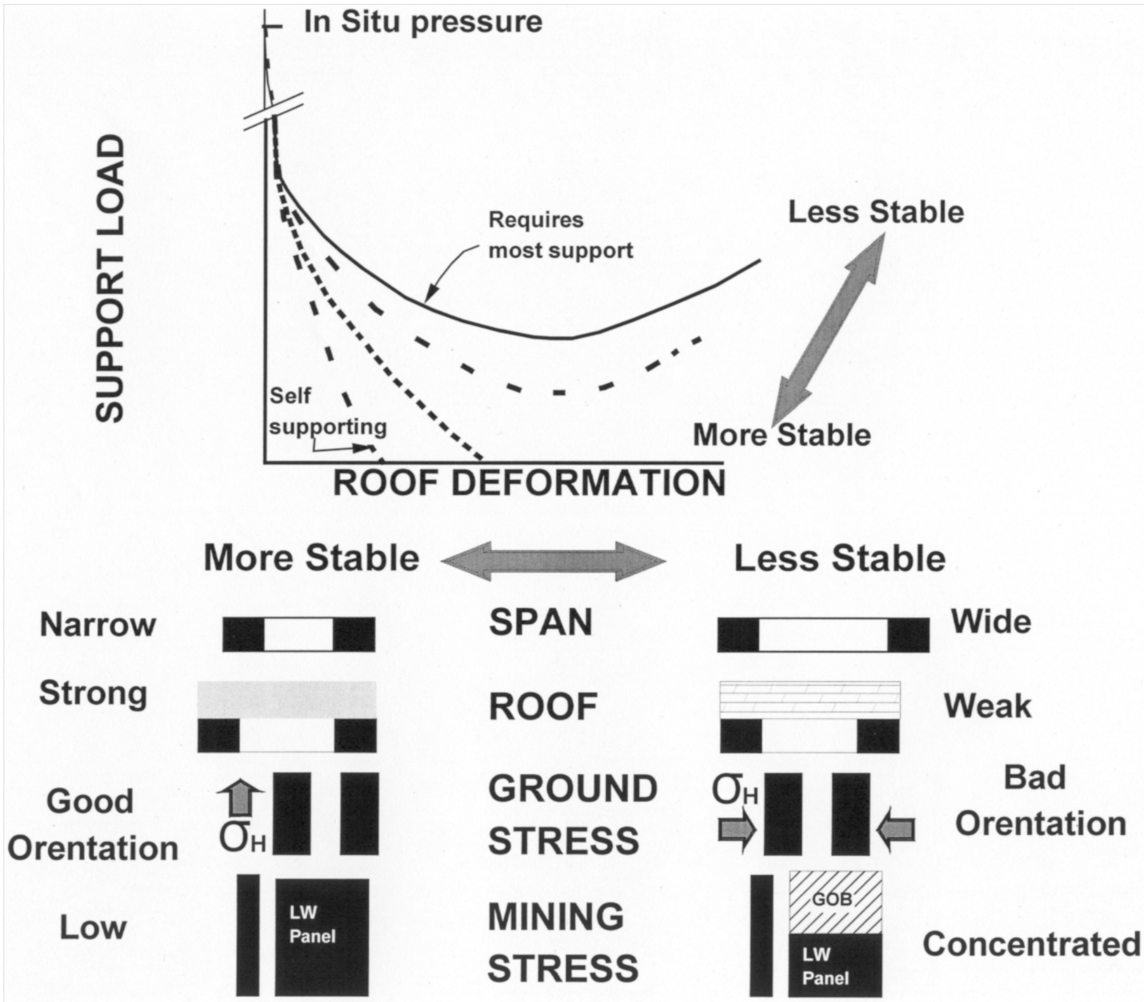


Figure 27.—Ground reaction curves and the factors that affect them.

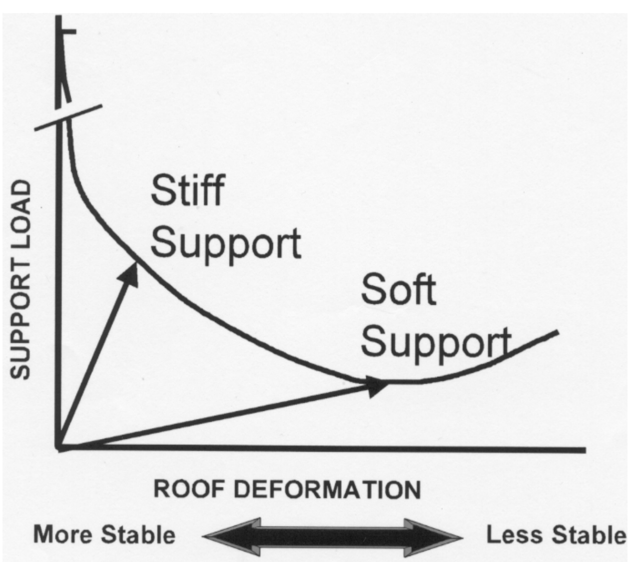


Figure 28.—Effect of support stiffness on the ground reaction behavior.

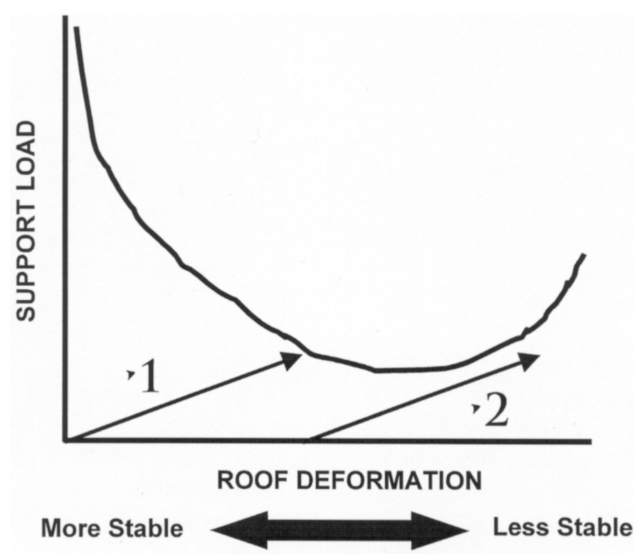


Figure 29.—Effect of installation timing.

particular support design. This capability is also provided by the Support Technology Optimization Program (STOP).

As illustrated by the ground reaction curve, the "ideal" roof support has the following properties:

- High initial stiffness, so that only small ground movements are needed to mobilize the capacity of the support;
- Large load-bearing capacity; and
- High residual strength over a large range of displacement.

Many of the engineered timber and concrete supports have largely succeeded in displaying these characteristics. Traditional wood supports have somewhat less desirable characteristics. Simple timber posts have little residual strength, while wood cribs have a low initial stiffness.

Since passive supports must be compressed to develop their load-carrying capacity, if they are installed too late, they might not develop sufficient capacity in time to put the roof into equilibrium. This is shown in figure 29. Both supports in this example have the same stiffness, but the second support was not installed in time to prevent critical roof deformation and thus could not prevent a roof fall.

CONCLUSIONS

Roof supports work best when they are matched to the ground conditions in which they are used. The performance characteristic of each support is unique. A support system may perform well in one application, but not in another. Understanding the ground, applied loads, and support characteristics are the keys to optimizing support design and application. The goal of the papers in these Proceedings is to provide the best available information and design guidelines to help mine planners in this task.

REFERENCES

Barczak TM [2000a]. Material handling considerations for secondary roof support systems. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Barczak TM [2000b]. NIOSH safety performance testing protocols for standing roof supports and longwall shields. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Barczak TM [2000c]. Optimizing secondary roof support with the NIOSH Support Technology Optimization Program. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 74-83.

Bauer ER, Dolinar DR [2000]. Skin failure of roof and rib and support techniques in underground coal mines. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Bieniawski ZT [1989]. Engineering rock mass classifications. New York, NY: John Wiley & Sons.

Chekan GJ, Listak JM [1994]. Design practices for multiple-seam room-and-pillar mines. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9403.

Colwell MG, Frith R, Mark C [1999]. Analysis of longwall tailgate serviceability (ALTS): a chain pillar design methodology for Australian conditions. In: Mark C, Heasley KA, Iannacchione AT, Tuchman RJ, eds. Proceedings of the Second International Workshop on Coal Pillar Mechanics and Design. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 99-114, IC 9448, pp. 33-48.

Dolinar DR, Martin L [2000]. Cable support in longwall gate roads. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Fabjanczyk MW [1992]. Directional drivage influences and the application of mechanistic mapping techniques. In: McNally GH, Ward CR, eds. In: Proceedings of the Symposium on Geology in Longwall Mining. Sydney, Australia: Coalfields Geology Council of New South Wales, pp. 137-142.

Hoek E, Wood DR [1988]. Rock support. Mining Magazine *Oct*:282-287.

Karabin G, Cybulski JA, Kramer JM [1982]. The formation and effects of transient abutment stresses during non-uniform face advance. In: Peng SS, ed. Proceedings of the Second International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 233-240.

Mark C [1990]. Pillar design methods for longwall mining. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9247.

Mark C [1991]. Horizontal stress and its effects on longwall ground control. Min Eng *Nov*:1356-1360.

Mark C [1999a]. Application of coal mine roof rating (CMRR) to extended cuts. Min Eng *51*(4):52-56.

Mark C [1999b]. Ground control in south African coal mines: a U.S. perspective. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 186-193.

Mark C [2000]. Design of roof bolt systems. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Mark C, Chase FE [1994]. Design of longwall gate entry systems using roof classification. In: New Technology for Longwall Ground Control; Proceedings—USBM Technology Transfer Seminar. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, SP 94-01, pp. 5-18.

Mark C, Chase FE [1997]. Analysis of retreat mining pillar stability (ARMPS). In: New Technology for Ground Control in Retreat Mining. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 97-122, IC 9446, pp. 17-34.

Mark C, Molinda GM [1996]. Rating coal mine roof strength from exploratory drill core. In: Peng SS, ed. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 415-428.

Mark C, Mucho TP [1994]. Longwall mine design for control of horizontal stress. In: New Technology for Longwall Ground Control; Proceedings—USBM Technology Transfer Seminar. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, SP 94-01, pp. 53-76.

Mark C, Molinda GM, Schissler AP, Wuest WJ [1994]. Evaluating roof control in underground coal mines using the Coal Mine Roof Rating. In: Peng SS, ed. Proceedings of the 13th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 252-260.

Mark C, Mucho TP, Dolinar DR [1998]. Horizontal stress and longwall headgate ground control. Min Eng *Jan*:61-68.

Molinda G, Mark C [1994]. The Coal Mine Roof Rating (CMRR): a practical rock mass classification for coal mines. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9387, 83 pp.

Molinda GM, Heasley KA, Oyler DC, Jones JR [1991]. Effects of surface topography on the stability of coal mine openings. In: Peng SS, ed. Proceedings of the 10th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 151-160.

Molinda GM, Mark C, Dolinar DR [2000]. Assessing coal mine roof stability through roof fall analysis. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Mucho TP, Mark C [1994]. Determining the horizontal stress direction using the stress mapping technique. In: Peng SS, ed. Proceedings of the 13th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 277-289.

Mucho TP, Barczak TM, Dolinar DR, Bower J, Bryja JJ [1999]. Design methodology for standing secondary roof support in longwall tailgates. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 136-148.

Rusnak J, Mark C [2000]. Using the point load test to determine the uniaxial compressive strength of coal measure rock. In: Peng SS, Mark C, eds. Proceedings of the 19th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 362-371.

Scott [1989]. Roof bolting: a sophisticated art. Coal *Aug*:59-69.

Zoback ML, Zoback MD [1989]. Tectonic stressfield of the United States. In: Geophysical Framework of the Continental United States. Geological Society of America, Memoir 172, pp. 523-539.

TRENDS IN ROOF BOLT APPLICATION

By Dennis R. Dolinar[1] and Suresh K. Bhatt[1]

ABSTRACT

The roof bolt system of a mine, if properly selected and installed, can allow for better roof control and reduce the potential for roof falls. Because of this potential to reduce roof falls and improve ground control conditions, the National Institute for Occupational Safety and Health (NIOSH) has an interest in the types of roof bolts that are used by the coal industry. Further, NIOSH has conducted research into how various support parameters can affect the number of roof falls that occur, including support type. Today, the five main types of roof bolts installed in U.S. coal mines are mechanical anchor bolts, point-anchor or resin-assisted mechanical anchor bolts, torque-tension bolts, combination bolts, and fully grouted resin rebar. This paper describes each support in detail. Because of the importance of resin in the functioning of the majority of bolts installed, resin grouts are also discussed. Further, trends in the types of roof bolts used are reviewed. Over the last 10 years, the significant trend has been the large reduction in the relative number of mechanical anchor bolts that are installed. These bolts have been replaced mainly by the resin-grouted rebar system. Data are also presented to show the impact of these changes in bolting on the number of roof falls.

[1]Mining engineer, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

Roof bolting in coal mines was initiated on a significant level only after World War II. At that time, the technological factor that drove the change from timber support to roof bolts was the increased mechanization in the mining industry, while the introduction of carbide-tipped drill bits made the drilling of bolt holes on a production basis feasible [Thomas 1950, 1954]. The change to roof bolts accelerated during 1949 after the steel shortage caused by the war had eased. Before 1949, only a few coal operations had experimented with roof bolting. By May 1949, there were more than 200 mines, mostly coal operations, that were installing roof bolts. Roof bolts supported over 14 million ft^2 of roof, using an estimated 800,000 to 1 million roof bolts. By the end of 1949, more than 430 coal mines used roof bolts.

Most of the bolts used a slot and wedge anchor, but some mines were already experimenting with expansion-shell or mechanical anchor bolts [Thomas et al. 1949]. The slot and wedge bolts had a threaded section at the head of the bolt and were tensioned by a nut tightened against a bearing plate. Because of the superior anchorage, the mechanical anchor bolts eventually replaced the slot and wedge bolts as the main roof support.

Even in the early days of roof bolting, it was recognized that a support in full contact with the rock along its entire length would be useful in dealing with rock shear forces, especially along hanging walls of hard-rock mines [Thomas 1954]. For the coal industry, the full contact support in the form of a fully grouted rebar was introduced in the late 1960s to early 1970s. This rebar bolt now is the predominant support used by the coal industry. Resin-assisted mechanical anchor or specialty bolts that combined the superior resin anchor with the tension of a mechanical anchor bolt were introduced in the late 1980s. Today, the five main types of roof bolts used in the U.S. coal industry are fully grouted resin rebar, mechanical anchor bolts, resin-assisted mechanical anchor bolts, torque-tension bolts, and combination bolts.

The type of bolt can be important for roof control because each support has characteristics that determine how the bolt will support the roof. The main characteristics that differentiate supports are whether the bolt is pretensioned and whether the anchorage length is full or point contact. How the bolt will interact with the site-specific rock mass properties and stress conditions and stabilize the roof must be considered when selecting a support [Mark 2000; Deere et al. 1970; Scott 1989]. The design of the support system must also consider other support properties, including anchorage capacity, anchorage load distribution, axial stiffness and toughness, shear resistance, shear stiffness, and shear toughness [Karabin et al. 1980].

This paper discusses the recent trends in the types of bolts used and the impact on roof control as the type of reinforcement has changed. The main types of bolts used today are described, and resin grouts used to anchor most of the rock bolt systems are discussed.

TRENDS

To evaluate roof bolt usage and trends in U.S. coal mines, the National Institute for Occupational Safety and Health (NIOSH) collected information from all known U.S. roof bolt manufacturers. Figure 1 shows the results of this study for 1999. However, the data were not complete in every instance, and the data are for all bolt usage, not just coal mines. Therefore, the results shown in figure 1 should be considered estimates. In the figure, five bolt types are indicated: mechanical anchor, resin-assisted mechanical anchor, torque-tension, combination, and fully grouted resin rebar bolts. Although this figure does not break down bolt usage by commodity, based on coal production, the coal industry uses an estimated 80% to 85% of the reinforcement. Therefore, the percentages of the different types of bolts used are probably very representative of the distribution of the type of bolts used in coal mines. In 1999, approximately 100 million bolts were used in the U.S. mining industry.

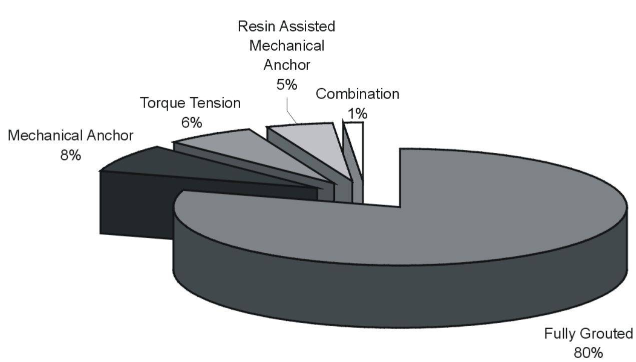


Figure 1.—Percentage of bolt types used in U.S. underground coal mines in 1999.

Fully grouted resin rebar comprises about 80% of these bolts. For the grouted rebar, approximately 80% are 0.625-in-diam #5 rebar, and nearly all of the remaining are 0.75-in-diam #6 rebar. Mechanical anchor bolts comprise about 8% of the supports, while torque-tension bolts represent 6% of the supports. Resin-assisted mechanical anchor bolts are approximately 5% and combination bolts about 1% of the market.

Surveys on bolt usage were also conducted in 1988 and 1991 [Scott 1989]. Table 1 shows a comparison between the percentages of each bolt type for these years and 1999. An estimate of the distribution of bolt types for 1976 is also given in the table. In that year, 80% of the bolts used were mechanical anchor and 20% resin-grouted rebar [Karabin et al. 1980]. From 1976 to 1991 and from 1991 to 1999, there was a substantial shift away from mechanical anchor bolts to fully grouted resin rebar. For resin-assisted mechanical anchor bolts, there was also a small decrease in the percentage used from 1991. In the 1991 survey, no distinction was made between torque-tension and combination bolts; the two systems were classified as point-anchor tension rebar. However, the combined usage of these two systems was about the same for 1988 and 1999, with a slight drop in the percentage used for 1991.

Table 1.—Bolt usage in U.S. mines by type

Bolt type	Percent			
	1976	1988	1991	1999
Mechanical anchor	80	35.3	34.1	8
Fully grouted	20	40	48.2	80
Torque-tension	—	3.5	[1] 4.6	6
Resin-assisted mechanical anchor	—	14.1	11.5	5
Combination	—	3.5	—	1
Other[1]	—	3.6	1.5	—
Total[2]	100.0	100.0	100.0	100.0

[1]Includes both torque-tension and combination bolts.
[2]Data may not add to totals shown because of independent rounding.

In general, resin-grouted rebar, despite being a passive support, may be considered a superior system to the mechanical anchor bolt because of the anchorage capacity and load transfer capabilities. Therefore, with the significant shift toward resin bolts, a corresponding improvement in roof stability might be expected that could be traced over the last 10 or even 25 years. The number of reportable roof falls that occurs each year can be used to evaluate whether a significant improvement has occurred. Table 2 shows the number of reportable roof falls, the number of mines, and the tons mined from 1989 to 1998 for longwall and room-and-pillar mining. To better compare the data for each year, the roof fall rate based on production is also shown. Figure 2 shows the roof fall rate per million tons of coal from 1989 to 1998 for both longwall and room-and-pillar mining.

For room-and-pillar mining, the roof fall rate trend from 1989 to 1998 can be evaluated by fitting a linear regression to the data. The results indicate that the coefficient of determination is <0.1, while the slope of the line is not significantly greater than zero. Essentially, there has been no change in the roof fall rate for the last 10 years in room-and-pillar mines. For 1975 and 1976, the roof fall rate per million tons was 5.667 and 6.841, respectively. Compared to the rates for 1997 and 1998 (7.067 and 7.011, respectively), there certainly has not been any decrease in the fall rate over 25 years.

For longwall mining, there was a decrease of about 1.6 roof falls per million tons, or a reduction of about 50% in the roof fall rate between 1988 and 1998. However, other factors such as face width, seam height, and number of gate road entries have caused much of this change by reducing the amount of development mining. Over this period, there was a 26%

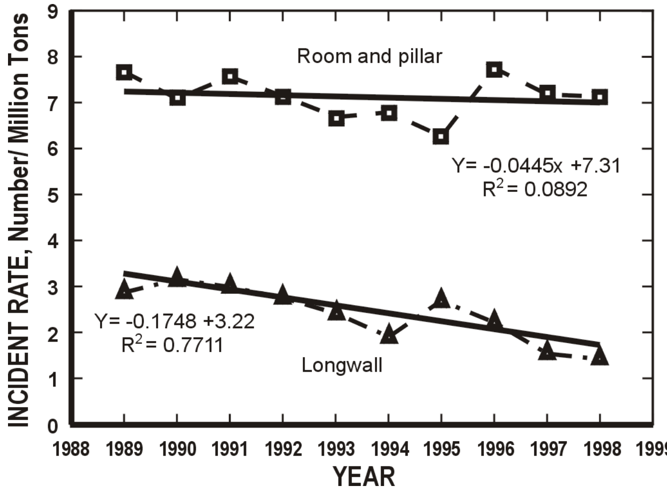


Figure 2.—Incident rate for reportable roof falls based on production from 1988 to 1998 for both longwall and room-and-pillar operations, including a regression analysis.

increase in panel width and a 7% increase in seam height [Merritt 1991; Fiscor 1999]. Further, the number of entries for a gate road has been reduced from 3.5 in 1990 to 3.1 in 1999, a decrease of 11%. Therefore, the increased panel width and seam height and the decrease in the gate road entries could account for nearly all of the decease in the roof fall rate. In 1990, about 26% of the production from a longwall was from development [Bhatt 1994]. In 1999, an estimated 15% to 18% of production came from development. Essentially, the change in roof support has had a minimal effect on the roof fall rate in longwall mines.

Therefore, despite the significant change from mechanical bolts to fully grouted rebar, there has been little change in the roof fall rate. This does not mean that the roof bolt type does not influence roof stability. There is documented evidence for specific cases where mines have changed bolt type and increased roof stability [Karabin and Hoch 1980; Peacock 1986; Stankus 1991]. However, there are many other aspects to the design of a roof support system other than the support type, and in many situations roof falls may have been prevented only with the addition of supplemental support. Further, the fully grouted bolts might have allowed mining under more difficult roof conditions, and therefore an increase in the roof fall rate could be expected that is balanced by the use of the fully grouted bolt. Also, a general analysis of all U.S. coal mines indicates that a number of factors may have changed, including the number of mines, roof conditions, and the accuracy of reporting roof falls. Further, there are reasons for changing the roof support other than for ground control, including different requirements for checking installation quality. Lastly, the mechanical anchor bolt has been replaced by a fully grouted resin bolt system using a #5 rebar in a 1-in-diam hole, and this may not be the optimum resin rebar system from a support standpoint. A more detailed discussion of the potential problems with the #5 rebar system compared to a #6 rebar bolt is presented below.

Table 2.—Roof fall rate per million tons mined for room-and-pillar and longwall mines

Year	Room-and-pillar				Longwall			
	Roof falls	Mines	Tons, million	Rate, falls per million tons	Roof falls	Mines	Tons, million	Rate, falls per million tons
1989	1,945	1,669	253	7.7	397	70	134	3.0
1990	1,875	1,659	266	7.0	470	76	154	3.1
1991	1,898	1,482	249	7.6	472	76	155	3.1
1992	1,726	1,338	242	7.1	452	77	161	2.8
1993	1,418	1,197	212	6.7	335	75	136	2.5
1994	1,496	1,136	221	6.8	348	71	175	2.0
1995	1,333	979	209	6.4	501	70	187	2.7
1996	1,653	874	214	7.7	460	66	197	2.3
1997	1,569	883	222	7.1	307	57	198	1.6
1998	1,528	828	218	7.0	285	55	201	1.4

Besides the bolt type, information was also obtained on the lengths of bolts installed. For the resin-grouted rebar lengths, 30% were under 4 ft, 47% were 4 ft, 13% were 5 ft, 8% were 6 ft, and 2% were >6 ft. The average bolt length for fully grouted rebar was 4.2 ft; for the mechanical anchor bolt, 4.1 ft; for the resin-assisted mechanical anchor bolt, 5 ft; and for the torque-tension bolt, 5.4 ft. It seems that the torque-tension and resin-assisted mechanical anchor bolts systems may be used in more difficult roof conditions where an increased length is also required to control the roof. However, no data from other years are available for comparison.

ROOF BOLTS

The five main types of roof bolts used in U.S. coal mines can be classified by two criteria: (1) anchorage length and (2) whether the support is installed with pretension [Scott 1989; Peng 1998]. Both criteria affect how the support actually functions in supporting the rock. From an anchorage standpoint, the bolts are either point- or full-contact anchors. For a point-anchor system, the anchorage lengths are usually <2 ft and include the mechanical anchor and resin-assisted mechanical anchor bolt. With the full-contact supports like the fully grouted rebar system, most, if not all, of the support is in contact with the rock. Besides anchorage, the full-contact provides reinforcement through resistance to rock movement. The full-contact support includes the fully grouted and torque-tension bolts. Although the combination bolts are only partially grouted, the anchor section can be considered a full contact support.

The other criterion is whether the system is installed with tension and therefore applies an active force to the rock. The tensioned support systems include the mechanical anchor and point-anchor bolts, which rely to a large extent on the active forces developed from bolt tensioning to provide reinforcement to the roof. The fully grouted rebar bolt is a nontensioned system. Both the combination and torque-tension bolts are active supports, yet have a full-contact anchor along a portion of their length. Therefore, along the anchorage portion, these supports resist movement similar to fully grouted rebar, with an active component to clamp the roof similar to the point-anchor systems.

MECHANICAL ANCHOR BOLTS

At one time, mechanical anchor bolts were the main roof support used in the coal industry. One advantage of mechanical anchor bolts is quick installation (usually <10 sec). Today, however, the mechanical bolt has been to a large extent displaced by other support systems, primarily fully grouted rebar. Mechanical anchor bolts consist of a smooth headed bar with a threaded anchor end. A mechanical shell anchor attached to the threaded end of the bolt is used to anchor the system. As the bolt is torqued, the force drives a plug against the outer shell, which expands and is set with a radial force against the rock (figure 3). Once the anchor is set, the bolt is then tensioned. Bolt torque is required to set the anchor and provide an active force to the rock for reinforcement.

The tension can be up to the yield of the steel or the anchorage capacity of the system. However, the anchor must be able to support high bolt loads with minimal displacement. Therefore, the anchor is critical to the functioning and capacity of this system. In general, there are two types of mechanical shell anchors: a standard and a bail anchor [Karabin et al. 1976]. The standard anchor has fixed leafs and makes only point contact with the rock. Because of this point contact, this anchor is usually better in stronger rock. The bail or free leaf anchor allows for almost total shell contact along the borehole wall and is therefore usually better in softer rock. However, because of variations in design of the shell anchors,

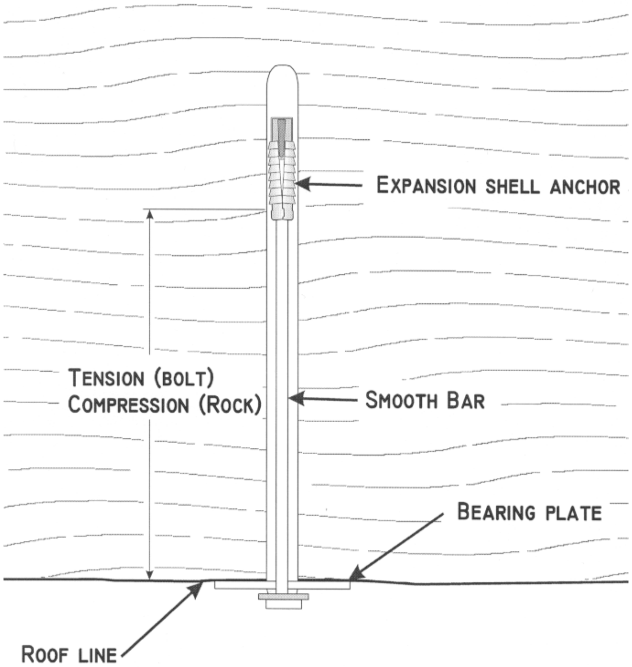


Figure 3.—Mechanical anchored bolt installed in roof.

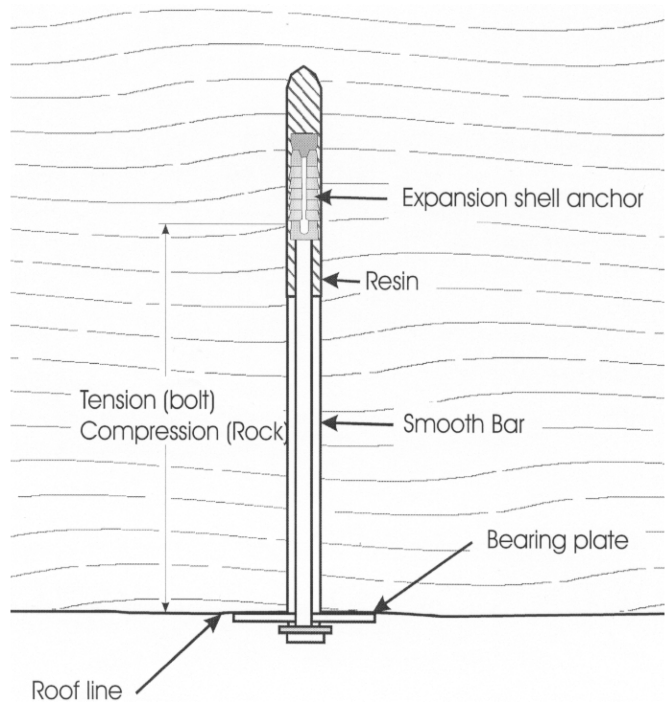


Figure 4.—Resin-assisted mechanism anchor bolt installed in roof.

underground testing is necessary to establish the best system for a particular rock.

Anchorage capacities up to 25,000 lb have been achieved, although the rock strength will always control the anchorage capacity. Therefore, the rock limits the amount of tension that can be applied by the mechanical anchor bolt and, in part, whether this load will be sustained or bleed off. Over time, the tension may be reduced because of creep or failure of the rock around the anchor and relaxation of the anchor threads. Therefore, the mechanical anchor bolt system has usually been installed in stronger roof rock or at least where the anchor is placed in a good-quality rock.

RESIN-ASSISTED MECHANICAL ANCHOR BOLT

Resin-assisted mechanical anchor bolts are essentially mechanical anchor bolts that have been transformed with the addition of a resin plug. Today, they are often called point-anchor bolts. These systems can be installed almost as fast as the mechanical anchor systems. Because they have greater anchor stability and less tension bleedoff than can be achieved with only the mechanical anchor, these bolts are used where ground control conditions are less favorable.

Although there are several varieties of resin-assisted mechanical anchor bolts, in general, the system consists of a headed bar (either deformed or smooth) with a threaded end for attachment of the mechanical shell anchor (figure 4). The mechanical anchor shells are designed with resin passages to allow for the flow of the resin around and below the anchor. The resin anchorage is usually established by a short cartridge of fast-setting resin with an anchor length of 1 to 2 ft. Mixing of the resin is achieved primarily by inserting the bolt and anchor through the cartridge. Compression washers and shells are also used with some systems to compress the resin and may result in improved anchorage [Stankus 1991]. The mechanical anchor allows for the immediate tensioning of the bolt, while the resin stabilizes the anchorage capacity. Often, the bolt tension is set at 70% or more of the yield of the bolts with torque loads up to 250 to 300 ft-lbf depending on the strength of the bolt. However, the amount of tension that can be applied is still limited by the rock strength. With the resin anchor, the tension bleedoff should be less than with a mechanical anchor system, although some bleedoff can still occur because of resin creep and anchor thread relaxation.

FULLY GROUTED REBAR BOLTS

The fully grouted bolt is now the main support used in U.S. coal mines, with about 80% of the market. The system consists of a headed rebar anchored with a full-length column of resin obtained from a cartridge (figure 5). The system is usually considered nontensioned, although plate loads of several thousand pounds may develop during installation [Karabin et al. 1976]. There are also special techniques that will allow for even higher installed loads [Tadolini et al. 1991]. The system works because of superior anchorage and stiffness that develops as a result of the full bolt length resin anchor. Pull tests show that it usually takes less than 2-ft length of a resin anchor to achieve

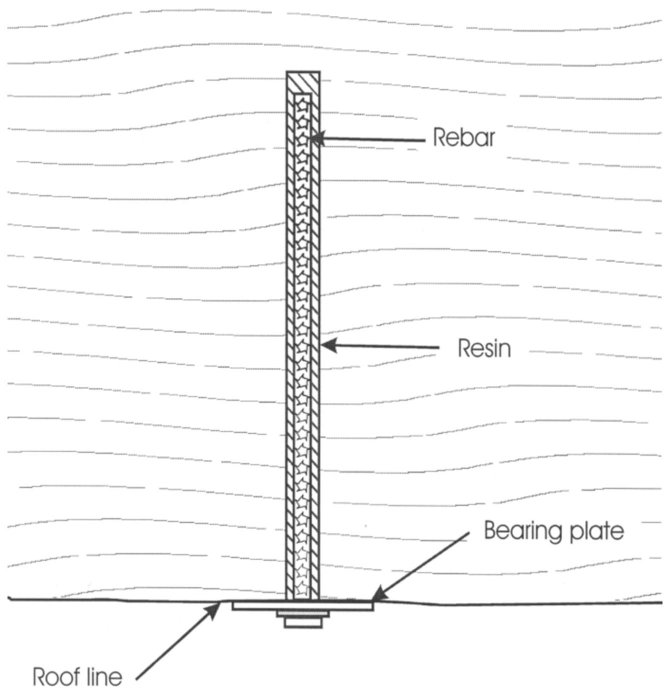


Figure 5.—Fully grouted resin rebar installed in roof.

the capacity of the support, although the "anchorage factor" depends on the rock strength and other installation parameters [Mark 2000]. The high stiffness is accomplished due to the full contact grout anchor and the ability of the resin grout annulus to quickly transfer the loads developed in the system back into the rock. Because of the superior bolt stiffness, significant resistance to the rock movement will be developed both axially and laterally. Loads developed along the bolt where roof separations occur will be quickly transferred back into the rock and the movement resisted and limited at these points.

Because the system works as a result of the load transfer that develops between the support and the rock, the annulus thickness of the resin grout, the distance between the bolt and the rock, is important to the proper installation and functioning of the system. The optimum system annulus as determined from experimental investigations was found to be about 0.125 in [Karabin 1976; Gerdeen et al. 1977]. Therefore, the optimum system for a 1-in hole is developed by using a 0.75-in-diam #6 rebar. Using an annulus smaller than 0.125 in has been limited by practical considerations, such as the thrust capacity of the bolter, the variation in the rebar diameters, and the potential resin loss [Campoli et al. 1999]. However, systems are now available that allow for only a 0.0625-in annulus that have overcome some of these problems, including controlling the rebar diameter [Tadolini 1998]. Testing has shown that effective load transfer and anchorage capacities can be achieved with this smaller annulus. Further, the installation forces, although higher with the smaller annulus, should be within the capacity of most roof bolt machines. There are two aspects to the performance of the system based on the resin annulus size, the bolt installation, and load transfer along the bolt. Both will be impacted negatively by a larger annulus. Of the rebar systems, the #5 rebar installed in a 1-in-diam hole is by far the most widely used bolt. However, the annulus for this system is 0.1875 in, which is larger than the optimum annulus thickness.

Even though fully grouted bolts are a passive system, the bearing plates are a necessary part of the support system. Without a bearing plate, a fully grouted bolt can still function and resist rock movement. However, surface control is lost where the bearing plate is the first support element in controlling surface or skin failure. About 98% of all ground fall injuries are from the failure of the surface of the roof or rib, and the plate helps protect the workers from injury due to failure of the skin of the roof. Further, loads of over 20,000 lb have been measured at the bearing plates, which indicates that the plates can add significantly to the roof support [Tadolini et al. 1986]. The plate will help the bolt resist roof movement in the lower 2 ft of the roof and is therefore an important element in roof reinforcement.

TORQUE-TENSION BOLT

A torque-tension bolt is essentially a resin-grouted rebar system that is pretensioned on installation. This system consists of a rebar with a threaded end at the head of the bolt. A nut with a torque-delay mechanism is used to torque and tension the bolt, with a resin column used to anchor the bolt (figure 6). With a full-column anchor, two different speeds of resin are used. In the upper portion of the anchor, there is a fast-setting resin; in the lower portion, a slower setting resin. The bolt is inserted, then rotated in the resin, when the upper fast resin sets, a torque-delay mechanism on the nut breaks and the nut is rotated up against the plate, tensioning the system. The applied load is distributed over the lower portion of the bolt containing the slow-set resin. When the slower resin sets, the system will resist rock movement with the stiffness of a fully grouted bolt and will further reinforce the lower roof with an active clamping force. A variation of this system is to leave the lower portion of the bolt ungrouted. If the grout column is sufficiently long, this upper part of the system will reinforce the roof similar to a full-grouted rebar, with the lower portion of the roof reinforced by the clamping action of the applied force. However, the lower portion of the bolt will have much less stiffness than the full-grout column and therefore less resistence to rock movement. In general, the fully grouted torque-tension system combines both the active force of the resin-assisted mechanical anchor bolts and the superior anchorage and stiffness of the resin rebar systems.

COMBINATION BOLTS

A combination bolt consists of a rebar anchor usually 3 to 4 ft long connected with a coupler to a smooth headed bar (figure 7). The rebar section is anchored in the hole with a resin cartridge. A shear pin in the coupler allows the resin to be mixed and set before the lower smooth headed bar is torqued and the system tensioned. Essentially, the rebar is a full-column resin bolt that provides an anchor for the system and provides reinforcement to the roof. With the tensioning of the lower section of the system, a clamping force is applied to the rock. These systems are often up to 8 ft long, and because the system consists of at least two components joined by coupler, a support system much longer that the seam height can be installed with relative ease. However, the weakness of the system is the coupler. Although the coupler is designed to withstand axial forces up to bolt failure, coupler failure can occur when there is sufficient lateral roof movement that causes coupler failure by shear.

RESIN

Grout anchors used with coal mine supports are commonly made with a polyester resin and packaged in a cartridge form. These systems can be either water- or oil-based. The trend today is toward water-based resins due to market forces. In the cartridge there are three components, but only two are active. These three components are the resin and the catalyst, which are active, and an inert filler (figure 8). Usually, the filler comprises between 65% to 75%, the resin 20% to 30%, and the catalyst 2% to 3% of the system [Eaton 1993]. However, in the United States some of the resin systems can have up to 85% filler. The resin consists of a polyester polymer solid with a liquid styrene monomer, while the catalyst is benzoyl peroxide. The filler is usually a limestone and acts not only to fill the hole, but also to help form the mechanical interlock between the rock, bolt, and grout. The cartridge is formed with mylar packaging and is set up with two compartments to keep the resin and catalyst apart. The packaging is torn and the system mixed during insertion and rotation of the bolt during installation.

Two important functional parameters of the resins are the mix time and the set or gel time. The mix time is usually between 3 to 10 sec and is limited by the gel point. However,

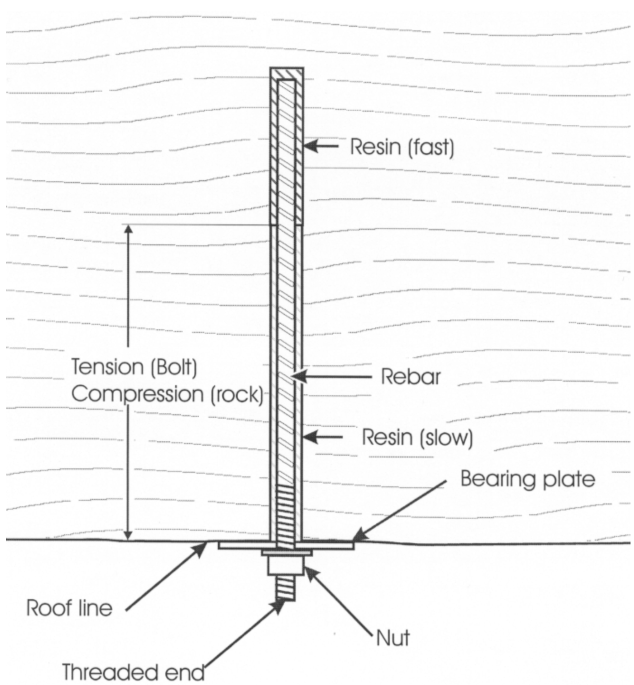


Figure 6.—Torque-tensioned bolt installed in roof.

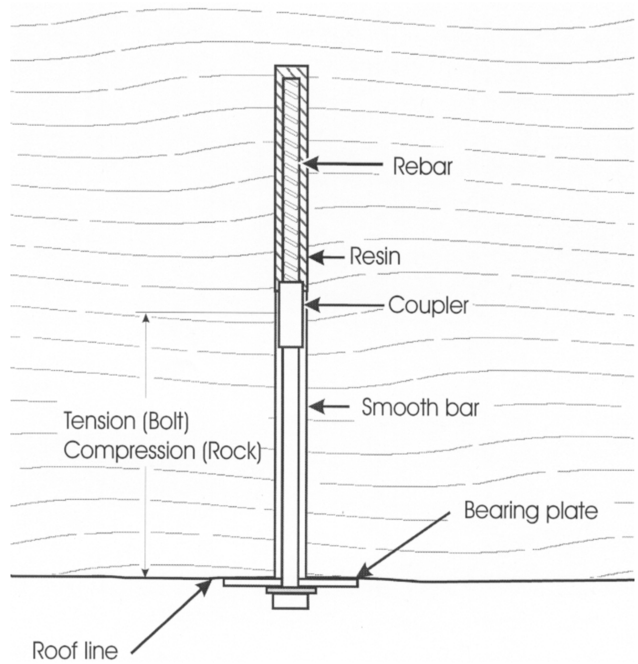


Figure 7.—Combination bolt installed in roof.

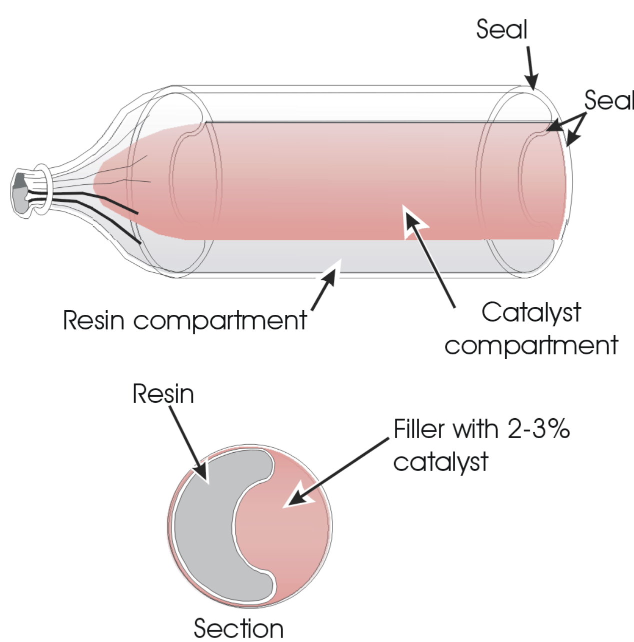


Figure 8.—Resin cartridge used to develop anchorage for roof bolts.

proper mixing of the resin is based on a certain number of rotations of the bolt that will occur during installation prior to the gel point of the resin. The gel time is controlled by inhibitors added to the resin and is the time required for the resin catalyst to change from a liquid to a solid; it will usually vary from 5 sec to 1 hr or more. The gel time must be long enough to allow for the installation of the support, but it is affected by the rock, bolt, and resin cartridge temperature. The resin systems come with manufacturer-recommended installation procedures, which, if not followed properly, could result in a substandard anchor. Also, the annulus thickness will affect how well the resin is mixed and therefore the anchor performance.

Strength and stiffness of the resin may affect the performance of the anchor system. A higher elastic modulus will allow for more efficient load transfer, while the higher strength will allow load transfer to take place over a more extended range of bolt load. The U.K. mining industry uses resins with strengths of about 11,600 psi and an elastic modulus of 1.6 million psi. These resins are designed to maximize anchor performance. In the United Kingdom, it is believed that a resin grout should be as strong as the strata that are being supported. In the United States, lower strength resins are used, normally with a strength of about 5,000 psi. With these weaker and less stiff grouts, load transfer may not be as effective and the grouts can fail sooner, resulting in less overall load transfer. However, it is uncertain whether stronger, stiffer resins would have a significant impact on roof conditions in most mining situations. In many cases, the resins used in U.S. coal mines are developing adequate anchorage for load transfer and are able to withstand strata movement and failure. The American Society for Testing and Materials (ASTM) has also developed standards for resin testing, including strength and mix time [ASTM 1998].

#5 VERSUS #6 REBAR IN A 1-IN-DIAMETER HOLE

The optimum annulus thickness based on experimental evidence and practical considerations has been found to be 0.125 in [Karabin et al. 1976; Geredeen 1977; Campoli 1999]. In a 1-in hole, this annulus is achieved with a #6 rebar. However the annulus thickness for a #5 rebar is 0.1875 in, and there is 30% less steel in the hole with this system. How and to what degree the increase in annulus and decrease in steel affects the performance of the system merits attention, especially since the #5 rebar represents about 65% of support installed.

One way to compensate for the decrease in bolt diameter is to increase the strength of the steel used for the #5 rebar. For many mines, the grade 60 #5 rebar replaced a grade 40 #6 rebar. Therefore, for the grade 60 #5 rebar the minimum yield load is 18,600 lbf compared to a grade 40 #6 rebar of 17,600 lbf. Essentially, both systems have about the same yield load. This increase in strength is accompanied by a loss of ductility in the steel. The elongation of the grade 60 rebar is 9% at failure and for the grade 40 rebar 12%. Further, the ultimate load for the #5 grade 60 rebar is 27,900 lbf, while the #6 grade 40 rebar is 30,800 lbf. The decrease in elongation and ultimate load results in a significant loss of toughness or the ability of the #5 rebar to absorb energy. Higher steel strengths such as grade 75, 90, and 100 are now used, while alloys can increase the elongation range of the steel. However, the general problem of replacing the bolt diameter with higher strength steels is still the loss of elongation and therefore toughness of the system.

Both axial and lateral stiffness of the system is affected by the annulus thickness. In tests conducted on 2-ft-long fully grouted bolts in a 1-in-hole, the axial stiffness for the #5 rebar was 142,000 lbf/in and for the #6 rebar 275,000 lbf/in [Bartels et al. 1985]. In testing these systems in shear (across the bolt axis), the shear stiffness for the #5 rebar was 30,200 lbf/in and for the #6 rebar 131,000 lbf/in. The results of these tests show a significant decrease in the stiffness both axially and laterally of the #5 rebar system. The fully grouted bolt relies on the system stiffness to resist rock movement both axially and laterally. Therefore, a decrease in stiffness should impact the support's ability to provide roof reinforcement.

With the larger annulus, more installation problems could be expected mainly in the form of increased glove fingering. To prevent this from occurring, extra care may be required to ensure that the manufacturer's recommendations on spin and rotation are followed during installation. Glove fingering will affect the bond between the grout and the rock developed by mechanical interlock and therefore the load transfer between the support and the rock. Further, glove fingering will often occur at the end of the bolt; thus, the anchorage capacity will be reduced along a critical portion of the support system. Essentially, the effective length of the support is reduced because of inadequate anchorage at the end of the bolt. Unfortunately, a pull test on a full-column grouted bolt will not normally reveal this condition.

In many situations, the #5 rebar system seems to be an effective support. However, some situations may require the increased stiffness and load transfer of the more optimum #6 rebar system. Also, the effects of inadequate installation resulting from the larger annulus have not been documented. To date, as far as the authors know, there is no published information on a direct comparison of the two systems in an underground setting or even on any extensive laboratory studies of the #5 rebar system. Therefore, the performance of the #5 rebar system needs to be evaluated in detail to determine if there is any reduction in reinforcement capabilities over those of the more optimum #6 rebar in a 1-in-diam hole. This is especially important since the #5 system is by far the most widely used support system in U.S. coal mines.

SUMMARY AND CONCLUSIONS

Today, there are five main types of roof bolts used in U.S. coal mines: fully grouted resin rebar, mechanical anchor bolts, resin-assisted mechanical anchor bolts, torque-tension bolts, and combination bolts. The trends in bolt usage indicate that the fully grouted rebar is the most widely used support, representing nearly 80% of the market. Essentially, the fully grouted rebar has replaced the mechanical anchor bolt where mechanical anchor bolts are about 8% of the market. The fully grouted bolt system is generally regarded as a superior reinforcement system compared to the mechanical anchor bolt, and there are a number of documented cases where a change from the mechanical anchor to the fully grouted bolt has increased roof stability at individual mines. However, the trend away from the mechanical anchor bolt to the fully grouted bolt has not resulted in any noticeable change in the rate of reportable roof falls in U.S. coal mines. It is possible that the fully grouted bolt has allowed for mining under more difficult roof conditions, or roof conditions in general have become less favorable in the overall mine population. Under such conditions, a higher roof fall rate would be expected where the use of the fully grouted bolt has actually offset this increase. However, such a change in the overall roof conditions, if it has occurred, might be difficult to document and prove. Further, there may be other reasons besides improved roof stability to change bolting systems, such as the different requirements to check the quality of the installation of each system. The torque-tension checks are more demanding for the mechanical anchor bolt than the installation checks for a fully grouted bolt.

For the fully grouted bolt, the #5 rebar is the most widely used and comprises nearly 65% of all support installed. However, this is not an optimum system when compared to a #6 rebar in a 1-in hole. Further, the strength and stiffness of resins used by the U.S. industry are generally less than those used in the United Kingdom and Australia. Therefore, how the #5 rebar and the property of the resins have impacted roof support performance is certainly open to discussion and further evaluation based on the reportable roof fall data. However, the selection of a specific bolt system to reinforce the roof is only one aspect of the design of primary roof support systems, although an important aspect for roof control.

REFERENCES

ASTM [1998]. Standard specifications for roof and rock bolts and accessories. West Conshohocken, PA: American Society for Testing and Materials, UF-432-98.

Bartels JR, Pappas DM [1985]. Comparative laboratory evaluation of resin-grouted roof bolt elements. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, RI 8924.

Bhatt SK [1994]. Longwall and room-and-pillar productivity: a review of U.S. coal mines. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9375.

Campoli AA, Mills P, Todd P, Dever K [1999]. Resin annulus size effects on rebar bolt pull strength and resin loss to fractured rock. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 222-231.

Deere DU, Peck RB, Monsees JE, Schmidt B [1970]. Design of tunnel lining and support systems. Washington, DC: Highway Research Board, Highway Research Record No. 339, pp. 26-33.

Eaton JC [1993]. Roofbolting-supply side technology. Colliery Guardian *Aug*:35-43.

Fiscor S [1999]. U.S. longwall census '99. Coal Age *Feb*:30-35.

Gerdeen JC, Snyder VW, Viegelahn GL, Parker J [1977]. Design criteria for roof bolting plans using fully resin-grouted nontensioned bolts to reinforce bedded mine roof. Houghton, MI: Michigan Technological University. U.S. Bureau of Mines contract No. J0366004.

Karabin GJ, Debevec WJ [1976]. Comparative evaluation of conventional and resin bolting systems. U.S. Department of Labor, Mine Enforcement and Safety Administration, IR 1033.

Karabin GJ, Hoch MT [1980]. Contemporary roof support systems provide a diversified approach to modern ground control. VPI Annual Institute on Coal Mine Health and Safety. Blacksburg, VA: Virginia Polytechnic Institute and State University, pp. 249-267.

Mark C [2000]. Design of roof bolt systems. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Merritt PC [1991]. As the time changes so do longwalls. Coal *Feb*:40-49.

Peacock MJ [1986]. Change in primary roof support system at Quarto Mining Company's Powhatan No. 4 Mine results in improved safety, productivity and costs. In: Peng SS, ed. Proceedings of the Fifth International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 80-88.

Peng SS [1998]. Roof bolting adds stability to weak strata. Coal Age *Dec*:32-38.

Scott JJ [1989]. Roof bolting: a sophisticated art. Coal, August.

Serbousek MO, Signer SP [1987]. Linear load-transfer mechanics of fully grouted roof bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9135.

Stankus JC [1991]. Tensioned point anchor resin system versus non-tensioned fully grouted rebar: two case studies. In: Peng SS, ed. Proceedings of the 10th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 65-75.

Tadolini SC [1998]. The effects of reduced annulus in roof bolting performance. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 231-236.

Tadolini SC, Dolinar DR [1991]. Thrust bolting: a new innovation in coal mine roof support. In: Peng SS, ed. Proceedings of the 10th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 76-84.

Tadolini SC, Ulrich BF [1986]. Evaluation of bearing plates installed on full-column resin-grouted bolts. Denver, CO: U.S. Department of the Interior, Bureau of Mines, RI 9044.

Thomas E [1950]. Roof bolting in the United States. Washington, DC: U.S. Department of the Interior, Bureau of Mines, IC 7583.

Thomas E [1954]. The application of rock bolting as a means of ground support is limited only by the resourcefulness and ingenuity of the mining engineer. Min Eng, November.

Thomas E, Barry AJ, Metcalfe A [1949]. Suspension roof support. Washington, DC: U.S. Department of the Interior, Bureau of Mines, IC 7533.

ASSESSING COAL MINE ROOF STABILITY THROUGH ROOF FALL ANALYSIS

By Gregory M. Molinda,[1] Christopher Mark, Ph.D.,[2] and Dennis Dolinar[3]

ABSTRACT

In 1999, 2,087 unplanned roof falls were reported from 841 mines. Nearly 55% of all mines reported at least one roof fall, and nearly 17% of the mines reported five or more falls. In order to investigate the variables that contribute to roof falls, the National Institute for Occupational Safety and Health (NIOSH) compiled a national database of roof performance from 37 coal mines. Geotechnical factors and their effect on roof fall rates were compiled from over 1,500 miles of drivage. The factor that is the best predictor of roof fall rate is the Coal Mine Roof Rating (CMRR). For a low CMRR ($\leq$30), almost all cases have high roof fall rates. Conversely, high roof fall rates are rare for strong roof rocks (CMRR $\geq$ 60). Roof fall rates were also higher in deeper mines, probably because of greater stresses. Intersections were much more likely to fall than roadways, and four-way intersections were more prone to fall than three-way intersections. In a controlled comparison of the effect of increasing bolt length on roof fall rates, it was found that longer bolts reduced the roof fall rates in 11 of 13 cases. A relationship between the roof fall rate, the intersection span, and the CMRR was also found. Finally, a systematic method for tracking roof performance and geotechnical variables was demonstrated.

[1]Research geologist.
[2]Supervisory physical scientist.
[3]Mining engineer.
Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

In 1998, a total of 2,232 unplanned reportable roof falls occurred in 884 U.S. underground coal mines. These falls resulted in 419 injuries and 13 fatalities. According to the Mine Safety and Health Administration's (MSHA) accident database, in 1999 over 55% of underground coal mines reported at least one roof fall and 17% of the mines reported more than five falls. In 1998, an estimated 12,500 miles of entry and 350,000 intersections were excavated. Falls of roof represent a very small proportion of exposed and supported ground. Nevertheless, each roof fall represents a direct threat to life and limb or an indirect threat to ventilation, escape, and equipment.

Through trial and error, operators have generally learned how to mine the coal and support the roof. After mining a certain length of time in a given coal seam, the appropriate entry width, mining height, length of cut, pillar geometry, and support can usually be determined. Roof instability occurs when conditions (usually geology, equipment, or economics) change, and the operator is uncertain how to respond or adapt.

Past studies that focused on detailed measurements at specific field sites have provided a wealth of data on specific roof stability topics, including bolt loads, bed separation, rock strength, mining influences, horizontal stress, and pillar stability [Signer 1998; Dolinar 1997; Chase 1999; Wang 1996]. This approach has been successful in increasing our understanding of the mechanics of roof instability and failure. However, site-specific instrumentation studies have the disadvantage that measurements from that site may not be entirely representative because of local variations in stress, geology, or support installation.

Roof falls, after all, are relatively infrequent events. It is difficult for deterministic rock mechanics models to explain why one intersection collapses while many others nearby remain stable. On the other hand, roof falls seem well suited for study using a probabilistic or empirical approach. The basic concept of the empirical approach is to collect a large quantity of real-world case histories and then use statistics to determine the most important factors.

The empirical approach also requires that the researcher begin with a clear hypothesis, often in the form of a simplified model of the real world that abstracts and isolates the factors that are deemed to be important. It therefore requires, as Salamon [1989] pointed out, "a reasonably clear understanding of the physical phenomenon in question." Without prudent simplification, the complexity of the problem will overwhelm the method's ability to discern relationships among the variables. But a key advantage is that critical variables may be included even if they are difficult to measure directly through the use of "rating scales."

During the past 5 years, modern empirical techniques have been applied to a variety of problems in coal mine ground control. They have resulted in some very successful design techniques, particularly in the area of pillar design, as well as some new insights into pillar and rock mass behavior [Mark 1999a, 1999b].

Much can be learned by observation of roof instability. The geometry, timing, geology, and frequency of roof falls may indicate what caused the failure. If these variables are carefully documented on a mine-wide basis, it may be possible to characterize the combination of factors that may contribute to a high incidence of roof falls. This documentation, expanded with corehole data, at a single mine is called hazard mapping. The hazard map indicates that poor ground conditions are expected. Responses can include bolt changes, narrowing the span, and supplemental support in critical intersections.

Due to highly variable geology and stress regimes, it has been difficult to transfer this knowledge to other mines. To address this problem, the National Institute for Occupational Safety and Health (NIOSH) decided to capture the experience of thousands of miles of existing mine roadways to assess the parameters that influence roof stability.

NATIONAL ROOF FALL DATABASE

Several geotechnical variables are known to influence roof stability. These include geology, mine opening geometry, horizontal and vertical stress regime, abutment load, and support. Through extensive interviews and underground reconnaissance with mine operators, NIOSH documented many of these variables. A national database of roof falls was created from data obtained during visits to U.S. coal mines (table 1). Ultimately, 41 mines in 10 States were visited, representing over 1,500 miles of drivage in most of the major coal basins where underground mining occurs (figure 1). Study mines were selected by computing the roof fall rate from the MSHA accident database. Drivage was estimated by converting annual production (excluding longwall production) into linear feet of advance, assuming an average seam height. Reportable roof falls were then divided by drivage to arrive at the roof fall rate (figure 2). Mines were then selected for study from this distribution to represent the entire range of roof stability from high, to medium, to low roof fall rates. Mines were also selected to represent a wide range of roof geologies, as well as varying size, ranging from large (>1 million tons per year) to small mines (<200,000 tons per year).

Table 1.—Summary of geotechnical data gathered for each case in the study

Mine No.	CMRR	Bolt length, ft	Bolt tension	Bolt grout column	Bolt capacity, kips	Bolts per row	Row spacing, ft	Entry width, ft	PRSUP	Inter-section diagonal, ft	Depth of cover, ft	Mining height, ft	Drivage, 10,000 ft	No. of 3-way	No. of 4-way	No. of segments	Falls 3W	Falls 4W	Segment falls	Roof fall rate
1	59	6.0	0	3	20.50	4	5.0	18.5	5.32	65.90	150.00	6.0	3.50	94	282	705.0	0	0	.0	.00
1	59	6.0	1	3	20.50	4	5.0	18.5	5.32	65.90	150.00	6.0	3.50	95	300	742.5	0	0	.0	.00
2	50	4.0	0	3	17.70	4	4.0	18.0	3.93	60.00	400.00	7.0	3.02	82	175	473.0	0	1	1.0	.66
2	50	6.0	0	3	17.70	4	4.0	18.0	5.90	60.00	400.00	7.0	.93	22	55	143.0	0	1	.0	1.08
2	75	4.0	0	3	17.70	4	4.0	18.0	3.93	60.00	400.00	7.0	2.34	58	116	319.0	0	0	.0	.00
2	75	2.5	1	1	17.70	4	4.0	18.0	2.46	60.00	400.00	7.0	2.32	51	140	356.5	0	0	.0	.00
3	44	6.0	0	3	18.60	4	4.0	18.0	6.20	62.00	350.00	3.5	1.40	28	128	298.0	0	2	3.0	3.57
3	44	6.0	1	2	26.40	4	4.0	18.0	8.80	62.00	350.00	3.5	.44	15	39	100.5	0	0	.0	.00
3	44	4.0	0	3	19.60	4	4.0	18.0	4.36	62.00	350.00	3.5	4.75	122	461	1,105.0	0	0	.0	.00
4	45	6.0	1	1	18.00	4	5.0	19.0	4.55	65.00	350.00	5.5	5.80	148	434	1,090.0	0	24	9.0	5.69
4	41	5.0	0	3	18.00	4	5.0	19.0	3.79	59.00	350.00	5.5	8.60	220	562	1,454.0	1	6	4.0	1.28
4	45	6.0	1	3	18.00	4	5.0	19.0	4.55	59.00	350.00	5.5	6.90	150	444	0	0	2	.0	.29
4	41	4.0	0	3	18.00	4	5.0	19.0	3.03	59.00	350.00	5.5	6.90	0	0	1,113.0	0	0	4.0	.58
4	45	6.0	0	3	18.00	4	5.0	19.0	4.55	59.00	350.00	5.5	21.90	700	1,250	3,550.0	0	2	3.0	.23
5	41	4.0	0	3	20.00	4	4.0	20.0	4.00	61.00	300.00	8.0	8.52	265	690	1,777.5	1	1	.0	.23
5	78	4.0	0	3	20.00	4	4.0	20.0	4.00	61.00	300.00	8.0	4.94	155	445	1,122.5	0	0	.0	.00
6	70	6.0	0	3	20.50	4	4.0	18.0	6.83	60.00	500.00	7.0	2.93	53	203	485.5	0	0	.0	.00
6	50	6.0	0	3	20.50	4	4.0	18.0	6.83	60.00	500.00	7.0	5.24	96	489	1,122.0	1	2	3.0	1.15
6	30	6.0	0	3	20.50	4	4.0	18.0	6.83	60.00	500.00	7.0	5.10	720	3,667	8,414.0	5	17	19.0	8.04
7	40	5.0	0	3	20.00	4	4.0	16.0	6.25	61.00	750.00	7.0	176.70	4,604	5,528	17,962.0	23	23	46.0	.52
8	38	4.0	0	3	20.50	4	4.0	19.0	4.32	60.00	600.00	4.0	39.84	1,000	3,079	7,658.0	3	11	12.0	.65
9	40	5.0	0	3	33.00	4	4.0	16.0	10.31	61.00	500.00	6.5	46.27	1,207	1,684	5,178.5	4	7	7.0	.39
9	40	6.0	1	2	33.00	3	4.0	16.0	9.28	61.00	725.00	6.5	17.12	483	385	1,494.5	3	0	1.5	.26
10 ...	55	6.0	0	3	26.00	4	4.0	19.0	8.21	75.20	1,050.00	10.0	3.79	149	89	401.5	1	2	.0	.79
10 ...	55	5.0	0	3	26.00	4	4.0	19.0	6.84	75.20	1,050.00	10.0	9.77	207	288	886.5	0	0	.0	.00
10 ...	55	4.0	0	3	26.00	4	4.0	19.0	5.47	75.20	1,050.00	10.0	.74	18	36	99.0	0	3	.0	4.05
10 ...	55	5.0	1	2	30.00	4	4.0	19.0	7.89	75.20	1,050.00	10.0	11.34	540	270	1,350.0	0	4	.0	.35
11 ...	42	4.0	0	3	18.40	4	5.0	18.0	3.27	61.00	200.00	5.0	5.90	0	654	1,308.0	0	7	7.0	2.37
11 ...	42	4.0	0	3	18.40	4	5.0	20.0	2.94	70.50	200.00	5.0	.65	0	109	104.0	0	2	3.0	7.69
11 ...	42	5.0	0	3	18.40	4	5.0	18.0	4.09	61.00	200.00	5.0	.49	0	60	120.0	0	0	.0	.00
11 ...	42	5.0	0	3	18.40	4	5.0	20.0	3.68	70.50	200.00	5.0	.05	0	10	10.0	0	0	.0	.00
11 ...	42	4.5	0	3	18.40	4	5.0	18.0	3.68	61.00	200.00	5.0	1.76	0	180	372.0	0	6	5.0	6.25
11 ...	42	4.5	0	3	18.40	4	5.0	20.0	3.31	54.00	200.00	5.0	.20	62	0	.0	0	0	.0	.00
12 ...	45	5.0	1	3	18.60	4	4.0	20.0	4.65	66.00	300.00	6.0	9.53	236	599	1,552.0	11	8	11.0	3.15
13 ...	55	4.0	0	3	17.60	4	4.0	19.0	3.71	62.70	500.00	6.5	1.44	30	58	161.0	0	0	.0	.00
13 ...	55	4.0	0	3	17.60	4	4.0	19.0	3.71	62.70	1,000.00	6.5	14.37	252	647	1,672.0	0	2	2.0	.28
14 ...	45	5.0	0	3	18.40	4	4.0	20.0	4.60	59.00	300.00	10.0	13.00	346	1,036	2,591.0	4	16	9.0	2.23
15 ...	37	6.0	0	3	18.40	4	4.0	20.0	5.52	58.00	400.00	10.0	4.00	114	285	741.0	0	8	1.0	2.25
15 ...	37	6.0	1	2	33.10	4	4.0	20.0	9.93	58.00	400.00	10.0	.58	15	36	94.5	0	4	3.0	12.07
15 ...	37	8.0	1	2	33.10	4	4.0	20.0	13.24	58.00	250.00	10.0	.58	17	46	117.5	0	3	2.0	8.62
16 ...	44	4.0	0	3	17.60	4	4.0	19.0	3.71	64.00	300.00	4.0	5.88	129	423	1,039.5	0	3	1.0	.68
16 ...	46	4.0	0	3	18.40	5	4.0	19.0	4.84	64.50	300.00	3.5	1.90	68	101	304.0	1	4	.0	2.63

Table 1.—Summary of geotechnical data gathered for each case in the study—Continued

Mine No.	CMRR	Bolt length, ft	Bolt tension	Bolt grout column	Bolt capacity, kips	Bolts per row	Row spacing, ft	Entry width, ft	PRSUP	Inter-section diagonal, ft	Depth of cover, ft	Mining height, ft	Drivage, 10,000 ft	No. of 3-way	No. of 4-way	No. of segments	Falls 3W	Falls 4W	Segment falls	Roof fall rate
16 . . .	53	4.0	0	3	18.40	4	4.0	19.0	3.87	62.00	300.00	3.0	.67	22	51	135.0	0	0	.0	.00
17 . . .	58	3.5	0	3	17.70	4	4.0	20.0	3.10	68.60	800.00	6.0	10.20	274	485	1,481.0	0	6	3.0	.88
17 . . .	58	3.5	0	3	17.70	4	4.0	20.0	3.10	75.70	800.00	6.0	1.20	0	137	137.0	0	4	.0	3.33
17 . . .	58	5.0	0	3	17.70	4	4.0	20.0	4.43	68.60	800.00	6.0	8.80	220	330	1,100.0	1	0	1.0	.23
17 . . .	58	5.0	0	3	17.70	4	4.0	20.0	4.43	75.70	800.00	6.0	1.20	0	110	110.0	0	0	.0	.00
17 . . .	58	3.5	0	3	17.70	4	4.0	20.0	3.10	64.80	800.00	6.0	4.80	112	254	681.0	0	0	.0	.00
17 . . .	58	5.0	0	3	17.70	4	4.0	20.0	4.43	64.80	800.00	6.0	8.90	188	418	1,108.0	0	0	.0	.00
18 . . .	42	4.0	0	3	31.00	4	4.0	20.0	6.20	59.00	500.00	6.5	3.10	55	95	272.5	3	2	4.0	2.90
18 . . .	42	6.0	1	2	37.00	4	4.0	20.0	11.10	62.00	500.00	6.5	4.50	47	130	330.5	1	2	2.0	1.11
19 . . .	40	4.0	0	3	17.70	4	4.0	18.0	3.93	63.00	350.00	7.0	4.45	93	236	611.5	3	6	6.0	3.37
19 . . .	40	4.0	0	3	17.70	4	6.0	18.0	2.62	63.00	350.00	7.0	.96	24	60	156.0	3	4	3.0	10.42
19 . . .	50	6.0	0	3	17.70	4	4.0	18.0	5.90	63.00	350.00	7.0	1.28	32	80	208.0	0	0	.0	.00
19 . . .	50	5.0	0	3	17.70	4	4.0	18.0	4.92	63.00	350.00	7.0	.64	16	40	104.0	0	0	.0	.00
19 . . .	75	2.5	1	1	17.70	4	4.0	18.0	2.46	63.00	350.00	7.0	3.74	96	234	612.0	0	0	.0	.00
20 . . .	40	8.0	1	2	33.00	4	4.0	18.0	14.67	65.00	600.00	7.5	17.20	340	820	2,150.0	9	43	32.0	4.88
21 . . .	63	6.0	0	3	26.50	4	4.0	18.0	8.83	69.00	500.00	7.5	.17	21	74	.0	0	1	.0	5.88
21 . . .	63	4.0	0	3	26.50	4	4.0	18.0	5.89	69.00	500.00	7.5	.90	0	0	179.0	0	0	1.0	1.11
21 . . .	45	6.0	0	3	26.50	4	4.0	18.0	8.83	69.00	500.00	7.5	.28	62	96	.0	0	5	.0	17.86
21 . . .	45	4.0	0	3	26.50	4	4.0	18.0	5.89	69.00	500.00	7.5	1.43	0	0	285.0	0	0	5.0	3.51
21 . . .	28	4.0	0	3	26.50	4	4.0	18.0	5.89	69.00	500.00	7.5	2.76	0	0	552.0	0	0	5.0	1.81
21 . . .	28	6.0	0	3	26.50	4	4.0	18.0	8.83	69.00	500.00	7.5	.52	59	232	.0	0	9	.0	17.18
21 . . .	39	4.0	0	3	26.50	4	4.0	18.0	5.89	69.00	500.00	7.5	6.13	0	0	1,225.0	0	0	1.0	.16
21 . . .	39	6.0	0	3	26.50	4	4.0	18.0	8.83	69.00	500.00	7.5	1.14	93	543	.0	0	1	.0	.87
21 . . .	76	4.0	0	3	26.50	4	4.0	18.0	5.89	69.00	500.00	7.5	8.15	0	0	16,292.0	0	0	4.0	.49
21 . . .	76	6.0	0	3	26.50	4	4.0	18.0	8.83	69.00	500.00	7.5	15.26	1,340	7,141	.0	1	10	.0	.72
22 . . .	40	6.0	0	3	17.70	4	4.0	18.0	5.90	63.00	400.00	7.0	3.02	75	185	482.5	0	0	.0	.00
22 . . .	40	6.0	1	2	17.70	4	4.0	18.0	5.90	63.00	400.00	7.0	1.00	24	60	156.0	0	0	.0	.00
22 . . .	40	6.0	1	1	17.70	4	4.0	18.0	5.90	63.00	400.00	7.0	1.10	25	65	167.5	0	3	.0	2.73
23 . . .	50	6.0	0	3	17.70	4	4.0	18.0	5.90	63.00	400.00	7.0	1.28	30	75	195.0	0	0	.0	.00
23 . . .	40	6.0	1	2	17.70	4	4.0	18.0	5.90	63.00	400.00	7.0	.76	18	45	117.0	0	0	.0	.00
24 . . .	42	6.0	1	2	33.00	4	5.0	19.0	8.34	61.00	650.00	6.0	73.20	1,967	2,825	8,600.5	5	34	16.5	.76
24 . . .	42	4.0	1	2	45.00	4	5.0	19.0	7.58	61.00	650.00	6.0	1.04	23	47	128.5	2	11	6.0	18.27
25 . . .	66	4.0	0	3	26.50	4	4.0	20.0	4.71	74.60	1,100.00	7.1	17.90	346	777	2,073.0	0	0	.0	.00
26 . . .	55	6.0	0	3	26.00	4	4.0	20.0	7.80	73.50	500.00	10.0	2.02	48	72	216.0	0	0	.0	.00
26 . . .	30	6.0	0	3	26.00	4	4.0	20.0	7.80	73.50	500.00	10.0	.76	18	27	81.0	3	5	.0	10.53
27 . . .	50	6.0	0	3	18.40	4	4.0	20.0	5.52	60.00	300.00	6.0	.94	80	48	168.0	0	0	3.0	3.19
27 . . .	50	6.0	0	3	18.40	4	4.0	16.0	6.90	50.00	300.00	6.0	4.94	254	221	871.0	0	0	2.0	.40
27 . . .	50	6.0	1	3	26.50	4	4.0	20.0	7.95	60.00	300.00	6.0	.10	26	3	42.0	0	0	2.0	20.00
27 . . .	50	6.0	1	3	26.50	4	4.0	16.0	9.94	50.00	300.00	6.0	.24	40	19	101.0	3	1	.0	16.67
27 . . .	50	4.0	1	2	26.50	4	4.0	20.0	5.30	60.00	300.00	6.0	.04	2	4	11.0	0	0	.0	.00
27 . . .	50	4.0	1	2	26.50	4	4.0	16.0	6.63	50.00	300.00	6.0	.22	13	15	49.5	0	1	2.5	15.91
28 . . .	50	6.0	0	3	26.50	4	4.0	22.0	7.23	78.40	1,000.00	5.0	14.30	396	736	1,914.0	12	24	14.0	3.50

Table 1.—Summary of geotechnical data gathered for each case in the study—Continued

Mine No.	CMRR	Bolt length, ft	Bolt tension	Bolt grout column	Bolt capacity, kips	Bolts per row	Row spacing, ft	Entry width, ft	PRSUP	Inter-section diagonal, ft	Depth of cover, ft	Mining height, ft	Drivage, 10,000 ft	No. of 3-way	No. of 4-way	No. of segments	Falls 3W	Falls 4W	Segment falls	Roof fall rate
29 . . .	35	6.0	0	3	18.00	4	4.0	18.0	6.00	64.00	200.00	7.5	2.90	39	227	512.5	0	0	.0	.00
29 . . .	35	6.0	1	2	18.00	4	4.0	18.0	6.00	64.00	200.00	7.5	.98	11	52	120.5	0	0	.0	.00
29 . . .	35	4.0	0	3	18.00	4	4.0	18.0	4.00	64.00	200.00	7.5	.51	9	52	117.5	0	0	.0	.00
30 . . .	52	4.0	0	3	17.70	4	4.0	18.0	3.93	63.00	400.00	7.0	8.20	179	516	1,300.5	0	21	5.0	3.17
30 . . .	40	4.0	0	3	17.70	4	4.0	18.0	3.93	63.00	400.00	7.0	5.30	117	216	607.5	12	4	10.0	4.91
30 . . .	75	2.5	1	1	17.70	4	4.0	18.0	2.46	63.00	400.00	7.0	.61	27	73	186.5	0	0	.0	.00
31 . . .	32	6.0	0	3	26.00	4	4.0	18.0	8.67	54.00	300.00	10.0	13.50	526	429	1,647.0	4	1	4.0	.67
31 . . .	32	6.0	0	3	26.00	4	4.0	18.0	8.67	54.00	800.00	10.0	2.50	62	93	279.0	1	1	3.0	2.00
32 . . .	51	3.5	0	3	18.50	4	4.0	20.0	3.24	66.00	500.00	3.5	11.80	261	763	1,917.5	0	0	.0	.00
33 . . .	30	5.0	1	2	37.00	4	4.5	18.0	9.14	56.00	300.00	6.0	3.22	64	258	612.0	0	8	1.0	2.80
33 . . .	30	6.0	0	3	20.50	4	4.5	18.0	6.07	57.00	300.00	6.0	12.50	236	873	2,100.0	4	21	25.0	4.00
34 . . .	44	3.5	0	3	20.00	4	4.0	20.0	3.50	68.00	250.00	3.5	6.40	296	506	1,456.0	0	4	6.0	1.56
35 . . .	47	3.0	0	3	26.50	4	4.0	20.0	3.98	61.70	300.00	3.0	7.90	275	701	1,814.5	0	0	.0	.00
35 . . .	47	3.0	0	3	26.50	4	4.0	20.0	3.98	66.30	300.00	3.0	1.20	0	201	402.0	0	0	.0	.00
35 . . .	47	3.0	0	3	26.50	4	4.0	20.0	3.98	61.70	500.00	3.0	3.90	144	402	1,020.0	0	0	.0	.00
35 . . .	47	3.0	0	3	26.50	4	4.0	20.0	3.98	66.30	500.00	3.0	.66	0	127	254.0	0	11	10.0	31.82
35 . . .	47	4.0	0	3	26.50	4	4.0	20.0	5.30	62.90	500.00	3.0	2.80	74	153	417.0	0	0	.0	.00
35 . . .	47	4.0	0	3	26.50	4	4.0	20.0	5.30	67.20	500.00	3.0	.30	0	43	86.0	0	0	.0	.00
36 . . .	49	3.5	0	3	26.50	4	4.0	20.0	4.64	63.00	150.00	3.0	1.00	41	72	205.5	0	0	.0	.00
36 . . .	49	3.5	0	3	26.50	4	4.0	20.0	4.64	57.00	150.00	3.0	.10	0	19	38.0	0	0	.0	.00
36 . . .	49	3.0	0	3	26.50	4	4.0	20.0	3.98	57.00	200.00	3.0	.10	0	16	32.0	0	0	.0	.00
36 . . .	49	3.0	0	3	26.50	4	4.0	20.0	3.98	63.00	200.00	3.0	.80	23	43	120.5	0	0	.0	.00
37 . . .	60	4.0	0	3	18.60	4	4.0	16.0	4.65	60.00	400.00	4.3	17.40	216	1,215	2,754.0	0	0	1.0	.06
37 . . .	39	6.0	0	3	18.60	4	4.0	16.0	6.98	60.00	400.00	4.3	5.50	81	294	709.5	0	1	1.0	.36
37 . . .	40	5.0	0	3	18.60	4	4.0	16.0	5.81	60.00	400.00	4.3	16.80	186	1,601	3,481.0	3	16	11.0	1.79

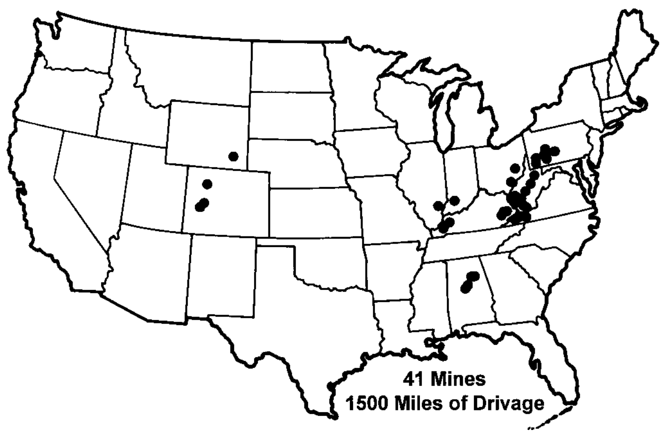


Figure 1.—Location of study mines.

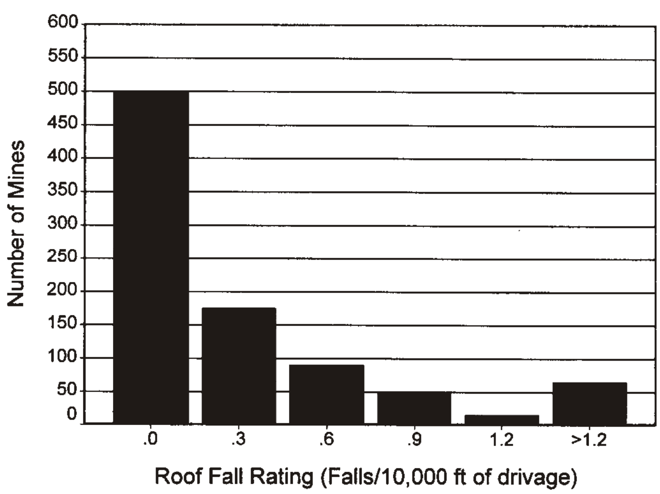


Figure 2.—Distribution of roof fall rates in U.S. coal mines.

At each mine, one or more "case histories" were collected. A case history was a portion of the mine that could be defined by a number of descriptive parameters and an outcome parameter (roof fall rate, falls/10,000 ft of drivage). The outcome parameter was based on the number of reportable roof falls that occurred in that portion of the mine.

According to MSHA regulations at 30 CFR 75.223, a fall of roof is reportable when it—

- Causes injury;
- Falls above anchorage;
- Blocks ventilation;
- Stops production for 30 min; or
- Blocks escape.

It was recognized that not all roof falls should be treated equally because their causes and impacts vary widely. A protocol for filtering roof falls for the study was developed. Tabulated roof falls were restricted to falls less than 18 months old in order to reduce time-dependent effects. Additionally, some mined areas are only accessible for short times (retreat panels or gate roads). To ensure equal treatment, mined areas had to be open a minimum of 18 months for use in the study.

Falls that were associated with longwall recovery, pillaring, multiple-seam effects, or other abutment pressures were also excluded. Falls associated with large-scale geologic dis-continuities, such as faults or sandstone channel margins, were excluded because they represent anomalous conditions and require specialized primary or supplemental support. In any study of roof safety or support performance, falls due to these factors must be treated separately because roof stability will not be achieved by standard support practices.

Because geotechnical parameters vary within mines, it was often not possible to characterize a whole mine by one set of variables. As a result, it was possible to have two or more "cases" within a mine representing a combination of geotechnical variables. The database ultimately included information from 37 of the 41 mines, but actually contained 109 "cases." The changing geotechnical environment of a mine roof was characterized by partitioning sections of a mine into zones with common variables. For example, a single mine might be broken up into three zones, or cases, if three roof bolt lengths were used. If two different roof geologies with different CMRR values were encountered within each of the three bolt length zones, then six cases were created.

ROOF FALL RATE

The roof fall rate was calculated as the outcome variable for each case. It was calculated by dividing the total number of roof falls that qualified for the study by the drivage. In order to quantify the percentage of drivage affected by a roof fall, roof falls were counted not as single entities but by the number of intersections and entry segments involved. A single roof fall covering two intersections and the crosscut between would count as three falls. Figure 3 shows the distribution of roof fall rates for the database. Nearly 60% of the cases in the data set had no roof falls. The other outcome variable was the four-way intersection rate. This number is calculated by dividing the number of four-way falls by the total number of four-way intersections in each case.

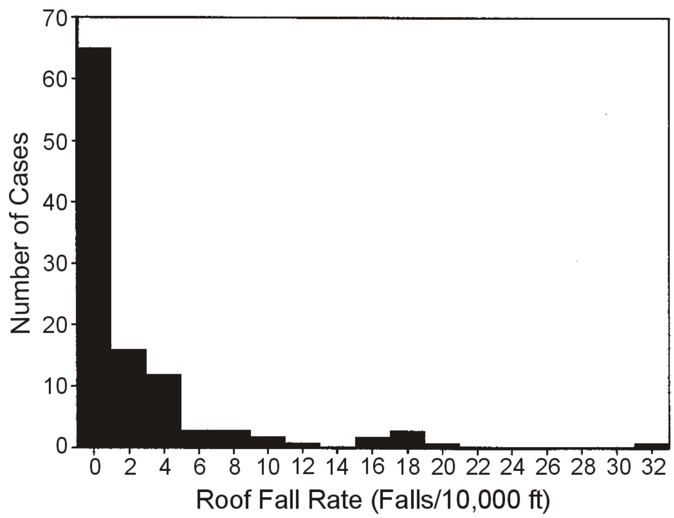


Figure 3.—Distribution of roof fall rates for cases in the study sample.

PRIMARY ROOF SUPPORT

For this study, a careful effort was made to characterize the roof bolts used in each case history. Operators were asked to report what bolts were installed historically through the mine. Where underground access permitted, NIOSH checked the accuracy of the information by reading the roof bolt heads, using a wire brush to clean them as necessary. This was done routinely where access permitted. After underground verification, roof bolt maps were compiled for the entire mine (figure 4). Six bolt variables were documented for each type of bolt used:

- Bolt length.
- Tension.
- Length of grout column.
- Yield capacity (grade of steel times cross-sectional area of the roof bolt).
- Bolts per row.
- Row spacing.

The most common bolt length used at the mines in our study was 5 ft (figure 5). Over 3.2 million feet of drivage was supported by 5-ft bolts (38%). Six-foot bolts were the next common length used (2.4 million feet of drivage, 30%), followed by 4-ft bolts (1.83 million feet of drivage, 22%).

Bolt tension was defined as either tensioned or untensioned. Eighty percent of the drivage was untensioned bolts (6.5 million feet), and 20% of the bolts were tensioned (1.6 million feet) (figure 6). All untensioned bolts were fully grouted, while nearly all tensioned bolts were ungrouted or partially grouted.

Figure 7 shows the distribution of roof bolts in the study by grout column and roof fall rate. The percentage of fully grouted bolts far outweighs the other grout column types, mirroring the national trend [Dolinar and Bhatt 2000]. There seems to be no correlation between roof fall rate and grout column length (roof fall rates are evenly distributed between variables), indicating that other factors are involved in the roof fall rate. Figure 8 shows the distribution of tensioned bolts as related by roof fall rate. Again, there is no correlation between roof fall rate and tension. Yield capacity ranged from 8.8 to 22.5 tons, with 9.5 tons of capacity occurring most frequently. The pattern of bolting in the United States varies little, with four bolts per row across the entry and 4 to 5 ft spacing between rows standard in nearly every case.

A summary variable, PRSUP, was calculated as a rough measure of roof bolt density:

$$PRSUP = \frac{Lb * Nb * C}{Sb * We}\,, \tag{1}$$

where
- Lb = length of the bolt, ft;
- Nb = number of bolts per row;
- C = capacity, kips;
- Sb = spacing between rows of bolts, ft; and
- We = entry width, ft.

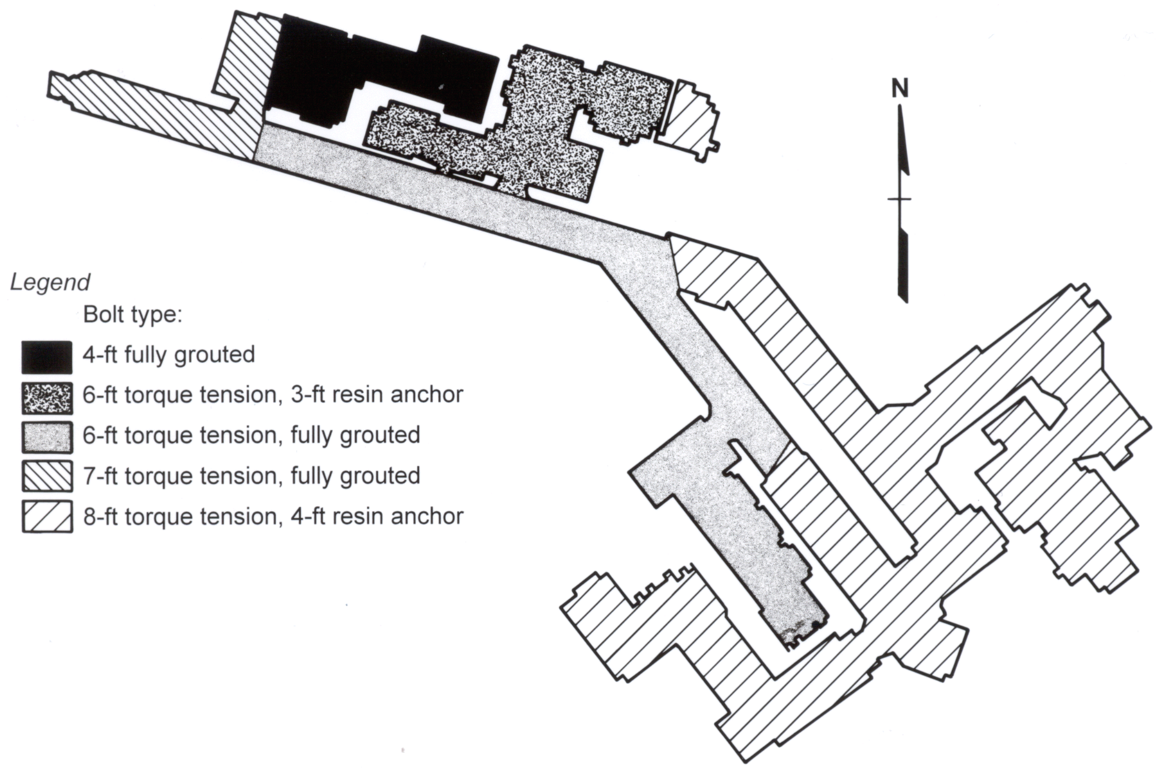


Figure 4.—Roof bolt map for study mine.

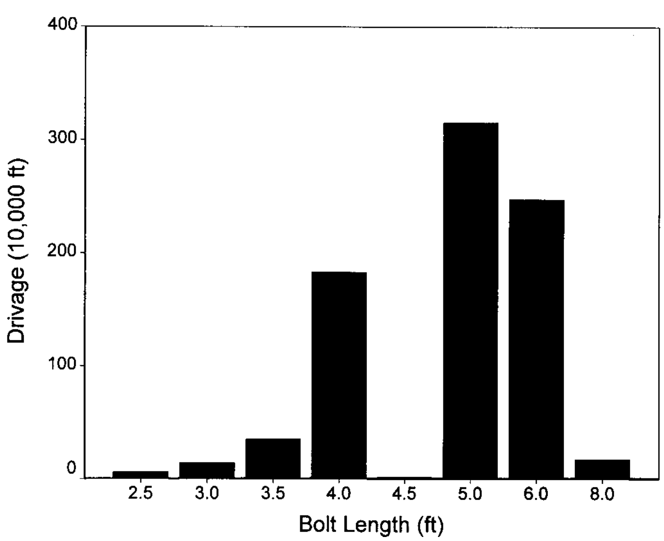


Figure 5.—Bolt length distribution in database as normalized by drivage.

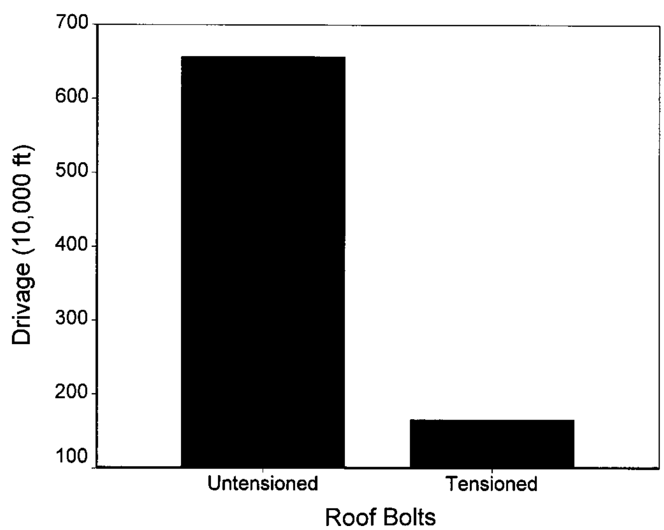


Figure 6.—Distribution of tensioned and untensioned bolts in the study.

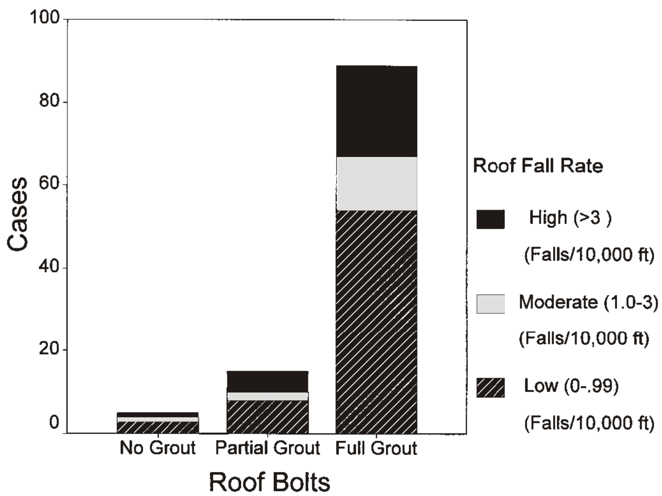


Figure 7.—Relationship between bolt grout column length and roof fall rate.

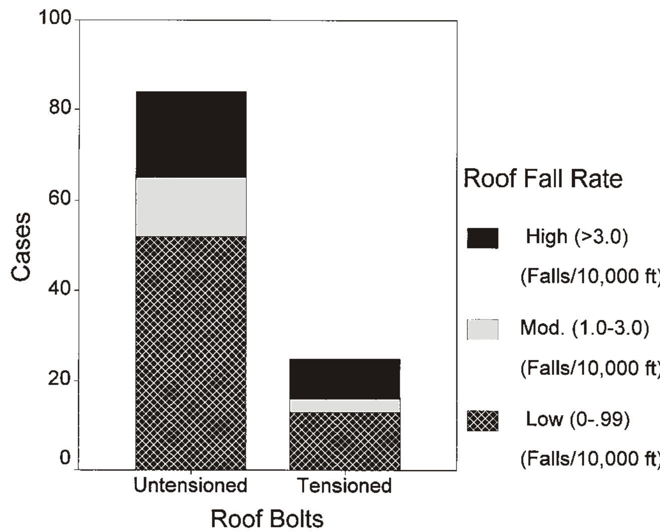


Figure 8.—Relationship between bolt tension and roof fall rate.

Figure 9 shows the distribution of PRSUP of all cases as grouped by roof fall rate. PRSUP is a rough measure of the "intensity" of the support. The more "steel" in the roof, the higher the PRSUP. It does not consider the type of bolt. PRSUP differs from the PSUP used in past studies [Mark et al. 1994] in that the bolt capacity has been substituted for the bolt diameter. The proportion of cases with high roof fall rates increases with increasing PRSUP. Additionally, the average PRSUP for cases with a roof fall rate equal to 0 is 4.4. The average PRSUP for cases with roof fall rate >1 is 6.1 This difference is significant at the $\propto$ = 0.05 level. This is an indication that operators are responding to poor roof conditions (higher roof fall rates) by adding more roof bolt support (increasing PRSUP) and that they are being only partially successful. The correlation between higher support densities and higher fall rates also presented a problem for the statistical study.

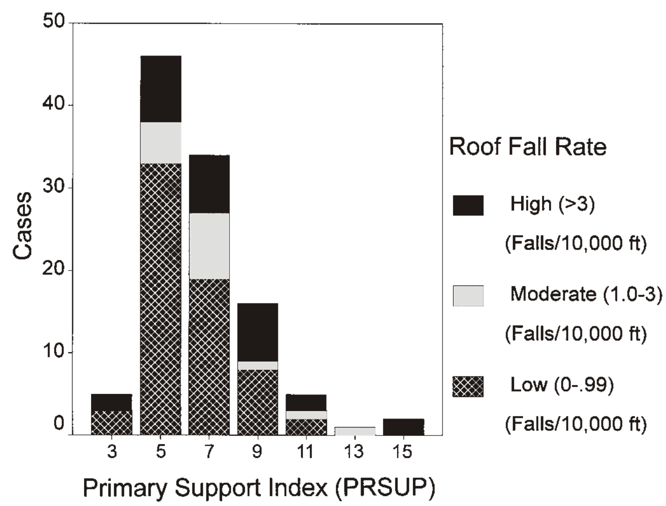


Figure 9.—Relationship between PRSUP and roof fall rate.

ROOF GEOLOGY

Roof geology has historically been difficult to quantify in ground control studies because of the many factors that comprise it. The Coal Mine Roof Rating (CMRR) was designed to quantify the geotechnical elements of the roof and return a number from 0 to 100 that reflects the competence of the roof [Molinda 1994]. The CMRR for each case was determined primarily by underground observation of roof falls, supplemented by drill core when it was available. A roof geology-CMRR map was constructed for each mine (figure 10). Figure 11 shows the distribution of the CMRR in the database. There is a strong correlation between CMRR and roof fall rate, with higher roof fall rates in the weaker roofs (CMRR $\leq$ 50). For cases with a CMRR $\leq$ 30, all have high or moderate roof fall rates. Conversely, high roof fall rates are rare for roof rocks with CMRR $\geq$ 60. If just the cases with no roof falls at all are considered (n = 41), the average CMRR is 52.3. For cases with a roof fall rate $\geq$ 2.0 (n = 36), the average CMRR is significantly lower at 42.8.

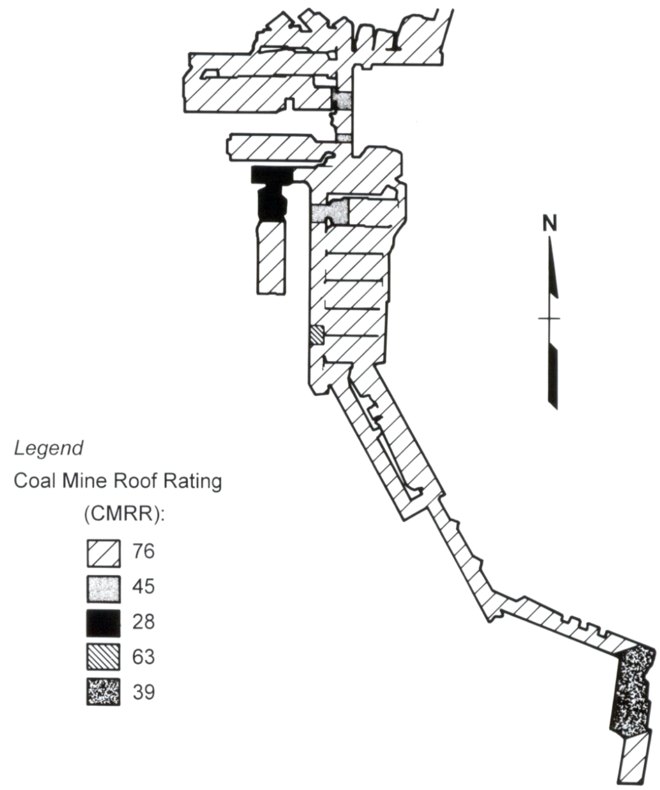


Figure 10.—CMRR map of study mine.

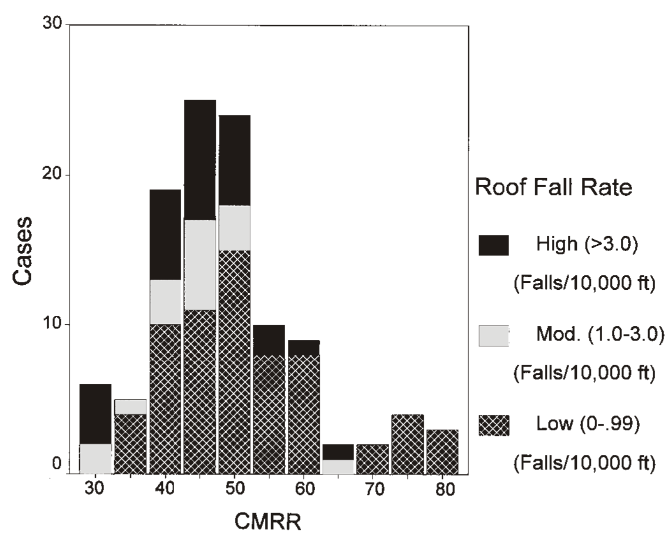


Figure 11.—Relationship between roof fall rate and CMRR.

INTERSECTION SPAN

The intersection diagonals were measured for a sampling of intersections in each mine, and in cleaned-up roof falls when possible. Figure 12 shows the method of measurement of intersections. Measurements were averaged to represent the typical span for each case. Figure 13 shows the distribution of intersection spans and roof fall rate. There is no obvious correlation between roof fall rate and intersection span.

While most (62%) of the falls in the total database occur in intersections, the intersection *fall rate* shows that intersections are much more likely to fall than entry or crosscut segments between intersections (figure 14). Segments are defined as any mined room that is not an intersection. Segments usually are two to three times as long as intersections. Of the intersections, four-way intersections are more likely to fall than three-way intersections (figure 15).

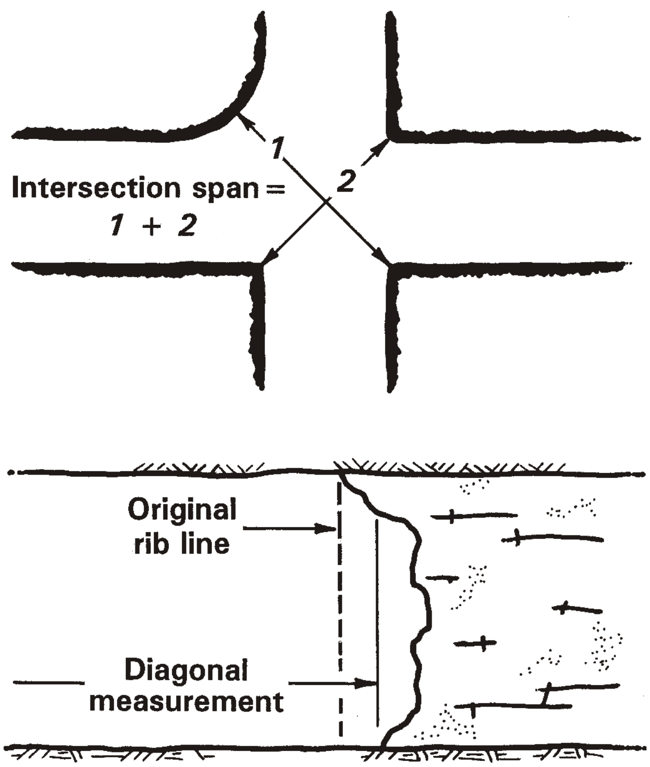


Figure 12.—Method of measuring intersection diagonals.

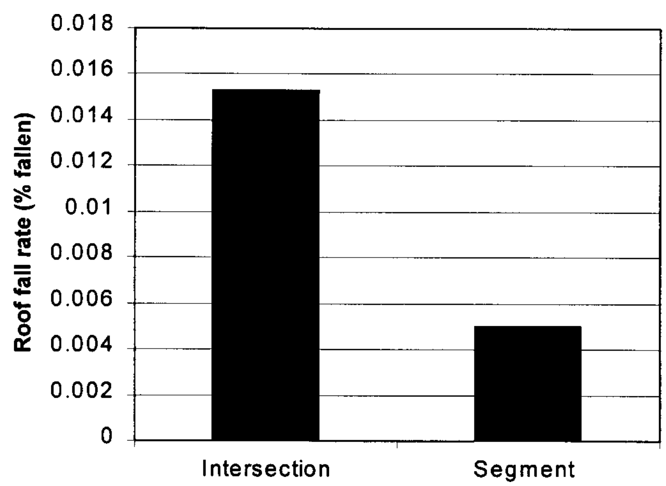


Figure 14.—Comparison of the roof fall rate between intersections and entry segments.

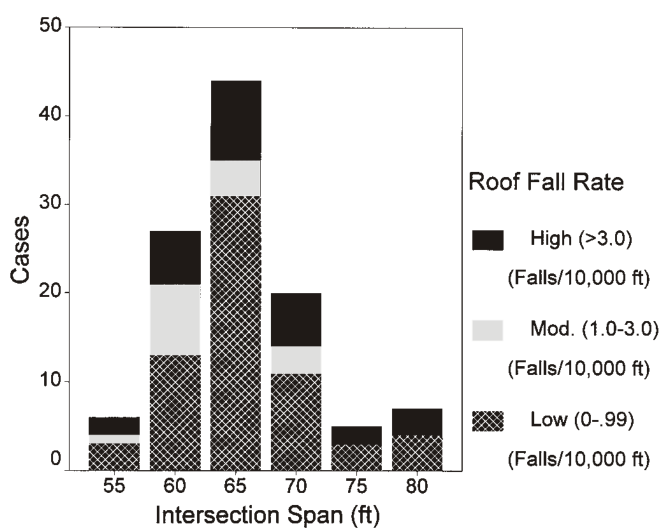


Figure 13.—Relationship between roof fall rate and intersection span.

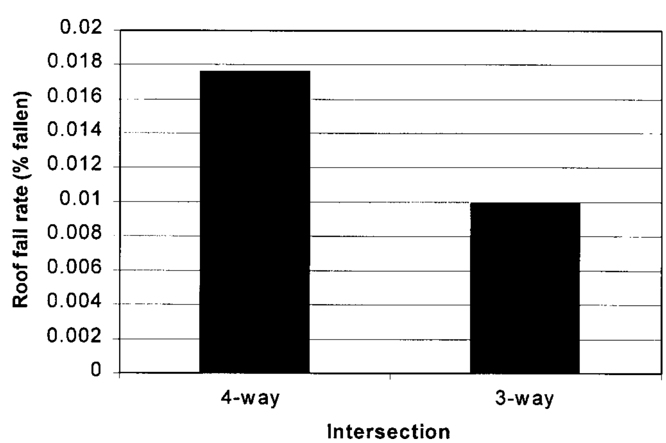


Figure 15.—Comparison between the roof fall rate for four-way and three-way intersections.

DEPTH OF COVER

The depth of cover over the mines in the study ranged from 0 to 1,600 ft. For analysis purposes, the average cover was recorded into three categories: shallow (0 to 400 ft), moderate (400 to 800 ft), and deep (>800 ft). Figure 16 shows the distribution of depth of cover for study cases.

STATISTICAL ANALYSIS

In order to determine the influence of the collected geotechnical variables on roof instability, the database was standardized and prepared for analysis. SPSS was the statistical package used for the analysis.

One goal of the study was to determine if there was a universal design equation that would use all or some of the geotechnical variables to predict the roof fall rate. A linear regression was performed that included all the significant geotechnical variables, including overburden, bolt length, grout length, density, entry width, CMRR, intersection span, tension, and bolt capacity. The regression technique progressively removes variables that are not significant in a stepwise procedure. The resultant regression equation can explain only 29.9% of the variation of the four-way intersection fall rate. More importantly, there is a positive relationship between bolt capacity and the roof fall rate. In other words, when bolt capacity goes up, the roof fall rate goes up. This defies logic, but the explanation is that when roof conditions deteriorate (roof fall rate goes up), higher capacity bolts are generally installed.

There are several explanations for the low overall correlation of the regression equation to the data. A test for intercorrelation of the variables revealed that a number of the variables were correlated to each other. This interdependence reduces the overall correlation of the design equation. Table 2 (Pearson correlation) is a test of the codependence of the geotechnical variables. A value of 1.0 is a perfect correlation, and 0.0 shows no correlation at all. Several bolt parameters—tension and grout indices, capacity and density, and bolt tension and capacity—are related. As roof conditions worsen, operators generally move toward tensioned bolts as well as increased capacity, apparently with only partial success. Intersection span and entry width are naturally related. The CMRR and the bolt length are also related. As expected, as the roof gets stronger (higher CMRR), operators are installing shorter bolts. These intercorrelations of variables confound the overall effect of any one variable on the outcome, which is the roof fall rate. Therein lies the difficulty in producing a reliable roof bolt design equation.

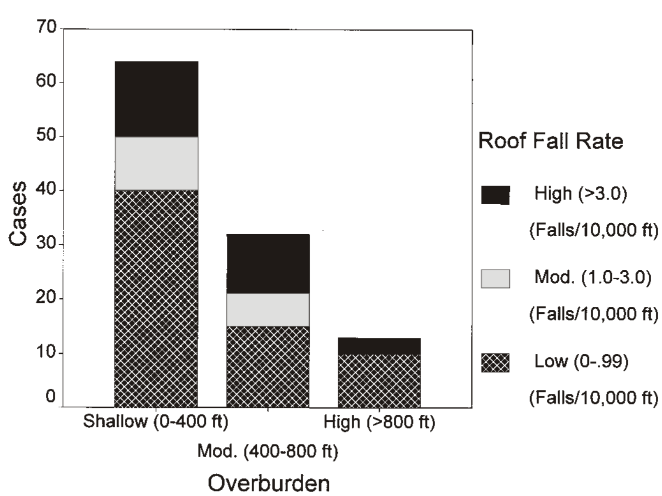


Figure 16.—Relationship between roof fall rate and overburden.

Although a universal design equation was not possible, the data analysis produced other interesting results. Other studies show significant evidence of increasing horizontal stress with depth [Mark 1994]. In this study, there is indirect evidence of the relationship. The data show that there is a statistically significant correlation (Pearson correlation = 0.253, statistically significant at 0.01 level) between CMRR and overburden. It seems that stronger roof rocks are encountered as overburden increases. There is no geologic reason for this, but it seems that operators are unable to mine weak roof at great depth. As overburden increases, stronger roof is encountered. In our database, 10 cases are mining at depths below 800 ft of cover, and 9 are >50 CMRR.

Figure 17 shows the relationship between CMRR and depth of cover for the study data. The individual cases have been divided into three roof fall rate categories; high, borderline, and zero. A line has been drawn on the graph that roughly separates lower roof fall rates from higher roof fall rates. Sixteen of twenty-two cases of zero roof falls fell above the classification line and were correctly classified. Nineteen of twenty-three cases with high roof fall rate (>2.0 falls per 10,000 ft of drivage) fell below the classification line and were correctly classified. The overall correct classification rate was 77%.

It seems likely that in our data, depth of cover is an indirect measure, or surrogate, for horizontal stress level. Horizontal stresses are seldom measured directly because of the difficulty and expense.

Using this assumption, the case histories were divided into two groups by depth of cover. The shallow-cover group included depths <400 ft, and the deeper-cover groups included depths ≥400 ft. Figure 18 shows the relationship between CMRR and PRSUP at high cover. Ten of sixteen "high" roof

Table 2.—Correlation between geotechnical variables in the study

Geotechnical variable	CMRR	Bolt length selected, ft	Tension index	Grout index	Capacity, kips	Bolts per row	Row spacing, ft	Entry width, ft	Density	PRSUP	Intersection span, ft	Overburden index	4-ways rate	Roof fall rate
CMRR	1.000	-.329	-.020	-.081	-.166	.037	-.147	.089	-.134	-.282	[1].233	[2].249	-.257	-.215
Bolt length selected, ft	[1]-.329	1.000	.251	-.059	.134	-.116	.038	-.215	.161	.738	-.067	-.039	.091	.105
Tension index	-.020	.251	1.000	[1]-.811	.326	-.161	.070	-.107	.271	.356	-.204	-.188	.217	.187
Grout index	-.081	-.059	[1]-.811	1.000	-.196	.131	-.012	.123	-.169	-.176	.124	.127	-.192	-.044
Capacity, kips	-.166	.134	[1].326	[2]-.196	1.000	-.178	-.084	.086	[1].907	.686	.043	[1].234	.442	.322
Bolts per row	.037	-.116	-.161	.131	-.178	1.000	.000	.167	-.076	-.132	.042	-.097	.079	.031
Row spacing, ft	-.147	.038	.070	-.012	-.084	.000	1.000	.041	-.375	-.232	-.089	-.248	.182	.075
Entry width, ft	.089	[2]-.215	-.107	.123	.086	.167	.041	1.000	-.172	-.245	[1].359	[1].124	.063	-.011
Density	-.134	.161	.271	-.169	[1].907	-.076	-.375	-.172	1.000	.767	-.016	[2].248	.341	.294
PRSUP	-.282	.738	.356	-.176	.686	-.132	-.232	-.245	.767	1.000	-.049	.154	.273	.266
Intersection span, ft	[1].233	-.067	-.204	.124	.043	.042	-.089	[1].359	-.016	-.049	1.000	[1].600	.060	-.053
Overburden index	[2].249	-.039	-.188	.127	[1].234	-.097	-.248	[1].124	[2].248	.154	[1].600	1.000	.089	.004
4-ways rate	-.257	.091	.217	-.192	.442	.079	.182	.063	.341	.273	.060	.089	1.000	[1].661
Roof fall rate	-.215	.105	.187	-.044	.322	.031	.075	-.011	.294	.266	-.053	.004	[1].661	1.000

[1]Correlation is significant at the 0.01 level (2-tailed).
[2]Correlation is significant at the 0.05 level (2-tailed).

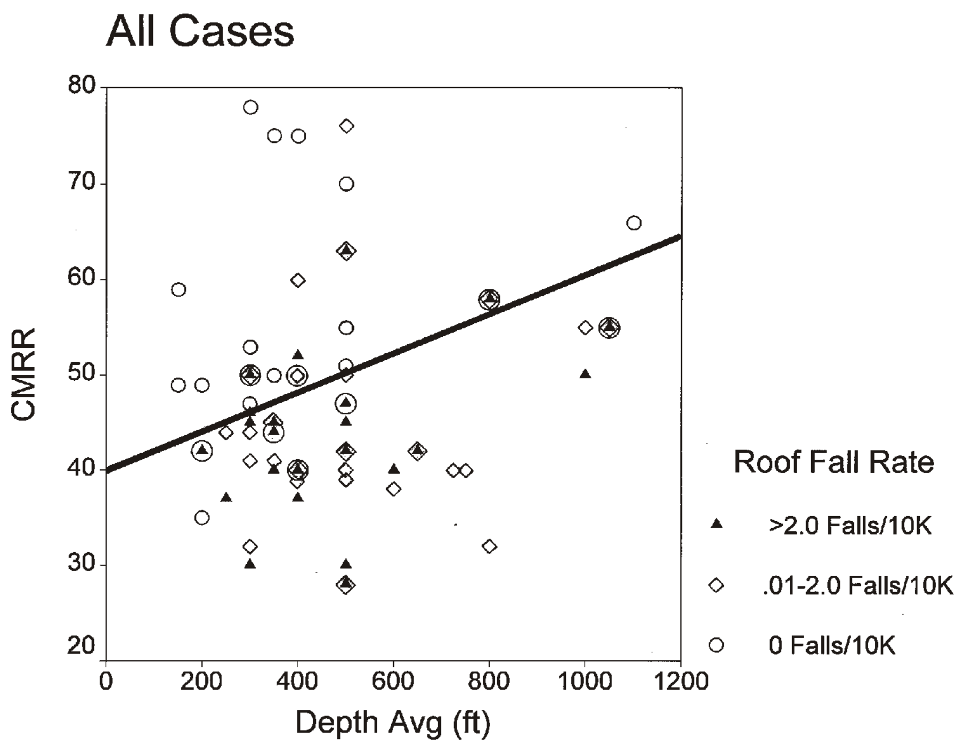


Figure 17.—Relationship between CMRR, depth of cover, and roof fall rate.

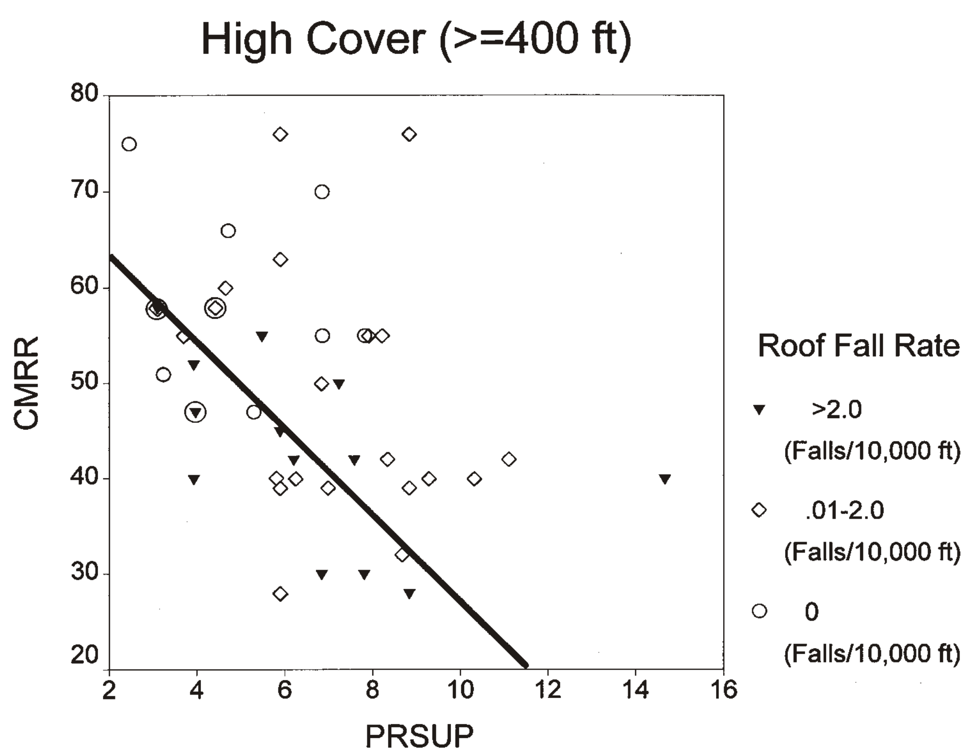


Figure 18.—Relationship between PRSUP, CMRR, and roof fall rate for cases under cover ≥400 ft.

fall rates fell below the classification line and were correctly classified. Eleven of 17 zero roof fall rate cases fell above the classification and were correctly classified. Looking at the "low" cover group, 16 of 17 cases with zero roof falls fell above the classification line, whereas the high roof fall rate cases were approximately evenly split (figure 19).

In both groups, most of the misclassified high-rate cases and the borderline cases plotted fairly close to the classification line. There were also high-roof-fall-rate cases that were misclassified that were also low CMRR. However, it seems that the relationship may break down for the weakest roof.

The relationship between CMRR and PRSUP was used to develop design equations, which are described by Mark [2000].

OTHER VARIABLES AFFECTING ROOF FALL RATE

The data collected during this study showed a considerable amount of scatter, as evidenced in many of the figures presented thus far. The explanation for the scatter is that the mining environment is far from a controlled experiment where all variables may be held constant and varied individually. If this were the case, the change in outcome variable can be observed and attributed to one variable. Moreover, some variables are difficult to measure, particularly over large areas. As a result, the observed roof fall rates may be affected by a number of factors that could not be included in the analysis, including—

Geologic variation: Typically, in an underground coal mine, parameters like geology (CMRR) can vary rapidly. Without systematic roof exposure (test holes), it may be difficult to assign the CMRR accurately to large sections of mine roof. By underground observation of roof falls and other exposures and drill core data, a CMRR was calculated that best represents the case area.

Horizontal stress: The presence of high biaxial horizontal stress is known to affect roof quality adversely [Mucho 1995]. The study used overburden depth as a surrogate for horizontal

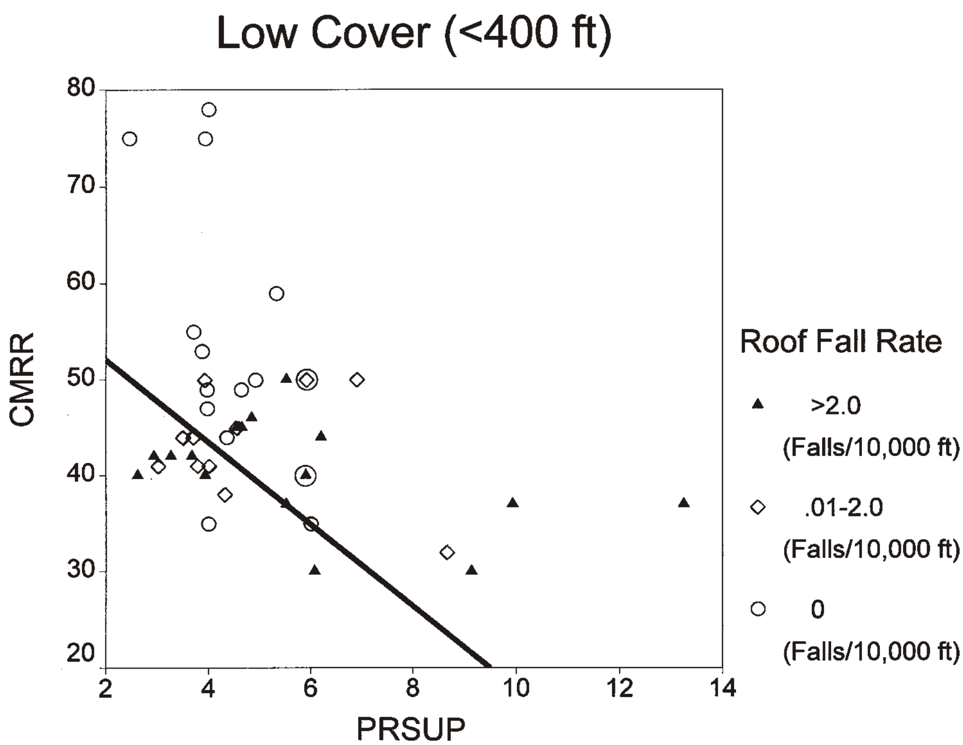


Figure 19.—Relationship between PRSUP, CMRR, and roof fall rate for cases under <400 ft.

stress, but actual stress levels vary by region, direction of mining, and surface topography.

Overwide intersections: It is suspected that in weak rock, small increases in intersection span (3 to 8 ft) can significantly weaken the roof. Many roof falls in the study were either inaccessible or not cleaned up, making it difficult to document accurately any overspans that may have contributed to the fall.

Quality of roof bolt installations: One of the most difficult parameters to measure is the quality of roof bolt installation. It is suspected that some failures can be caused by deficient bolting practices, including loss of tension, large bolt annulus, bent or notched bolts, overdrilled holes, or long lag times before bolting. All of these factors may mask the performance of bolt systems by increasing the roof fall rate.

OTHER RESULTS

As described above, the statistical analysis of data becomes more complicated with increasing numbers of variables. Interdependence of variables and errors in measurement are compounded with large numbers of variables. An alternative was to conduct analyses using "paired data" from individual mines. In these cases, only a single variable changes.

The most successful of these analyses was on roof bolt length. From the large data set, 13 pairs of data where two different lengths of roof bolts were used at the same mine were extracted. The roof bolt lengths differed by at least 1 ft in the pairs. Table 3 shows the bolt lengths, along with the CMRR, the bolt type, the roof fall rate, and the percentage of difference in roof fall rate between the two lengths.

The data show that in 11 of 13 cases, the four-way intersection roof fall rate was less with the longer bolt (figure 20). The roof fall rates for the paired data range from 0.0 to 18.3 and the

Table 3.–Test cases showing the effect of bolt length on roof fall rate

Mine	CMRR	Bolt length, ft	Roof fall rate (falls per 10,000 ft)	% change	Bolt type
1	50	4	1.08		Fully grouted.
		6	.66	-39	Fully grouted.
2	37	6	12.07		Tension.
		8	8.62	-29	Tension.
3	41	5	1.28		Fully grouted.
		6	.23	-82	Fully grouted.
4	55	4	4.05		Fully grouted.
		6	.79	-80	Fully grouted.
5	58	3.5	.88		Fully grouted.
		5	.23	-74	Fully grouted.
6	39	5	1.79		Fully grouted.
		6	.36	-80	Fully grouted.
7	42	4	2.9		Tension.
		6	1.11	-61	Fully grouted.
8	42	4	18.3		Tension.
		6	.76	-96	Fully grouted.
9	44	4	0		Tension.
		6	3.57	+100	Tension.
10	40	5	.52		Fully grouted.
		8	0	-100	Tension.
11	40	5	.39		Fully grouted.
		6	.26	-34	Tension.
12	30	5	2.8		Tension.
		6	4.0	+40	Fully grouted.
13	50	4	2.45		Tension.
		5	1.77	-28	Fully grouted.

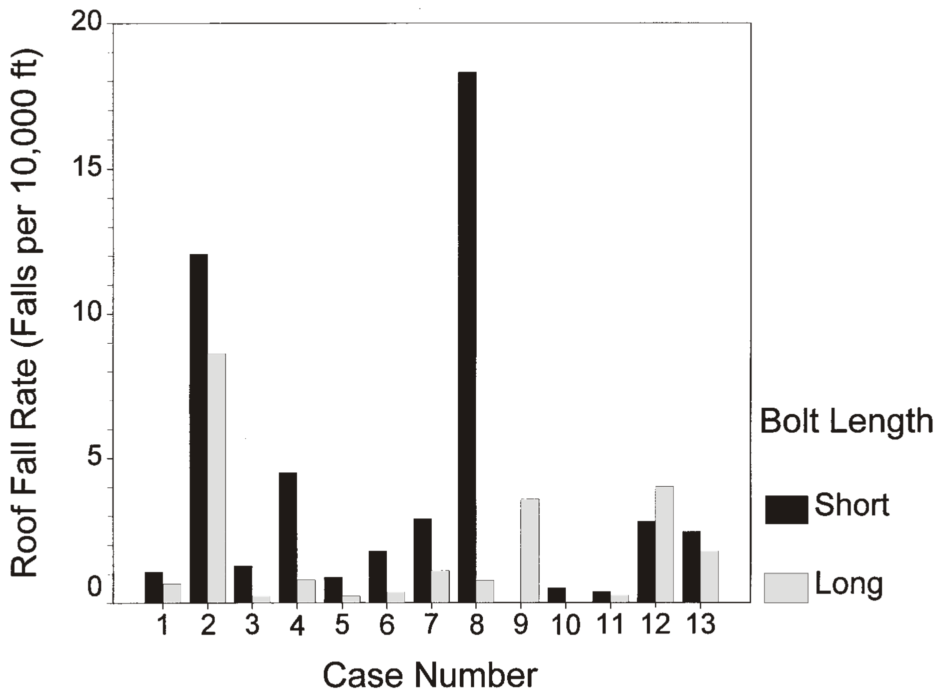


Figure 20.—Paired cases of long and short bolts showing benefits of long bolts and decreasing roof fall rates.

decrease in roof fall rate (average 65%, n = 13) with increasing bolt length holds true even in the high roof fall rate range. Six of the 13 pairs of bolt lengths mixed tensioned bolts with fully grouted bolts and the relationship holds true with both types of bolts. Thus, through a wide range of CMRR (30-58), an increase of at least 1 ft in bolt length can be expected to result in a decrease in four-way intersection roof fall rate.

The paired bolt length data contain four cases comparing longer fully grouted bolts with shorter tensioned bolts. The roof fall rate was lower in just one case and higher in the three other cases when the mine used the shorter tensioned bolts.

The relationship between CMRR and intersection span was also analyzed. The data were partitioned by four-way intersection fall rate into low (0-0.001 falls per 100 four-way intersections), moderate fall rate (0.001-0.05 falls per 100 four-way intersections), and high fall rate (>.05 falls per 100 four-way intersections). Additionally, only fully grouted bolts were used in the analysis. By logistic regression, a line was fitted to the data and presented in figure 21. No cases with high roof fall rates fell to the right side of the regression line. This line can be used to indicate whether smaller spans might be helpful in relieving the incidence of roof falls. It is rare for high roof fall rates to occur in roof with intersection spans less than the equation. The less conservative regression line (span = 31 + 0.66 CMRR) might be an appropriate first approximation design equation. Based on our data, there is also a likelihood that intersection falls will be reduced by a decrease in intersection span. The intersection span measured for the study was taken at the midpoint between the original rib corner and the subsequent sloughage point. This differs from MSHA's measurement point, which is the original rib corner. For this reason, the projected intersection spans will be somewhat conservative.

Intersection diagonals are usually related to entry width. The data were studied to determine the typical intersection spans that are encountered underground. Figure 22 shows the mean of the sum of the diagonals for 16-, 18-, and 20-ft entries. It also shows that in deeper mines, the sum of the diagonals was 3-4 ft wider than in the shallow mines with the same entry width, probably because of greater rib sloughage.

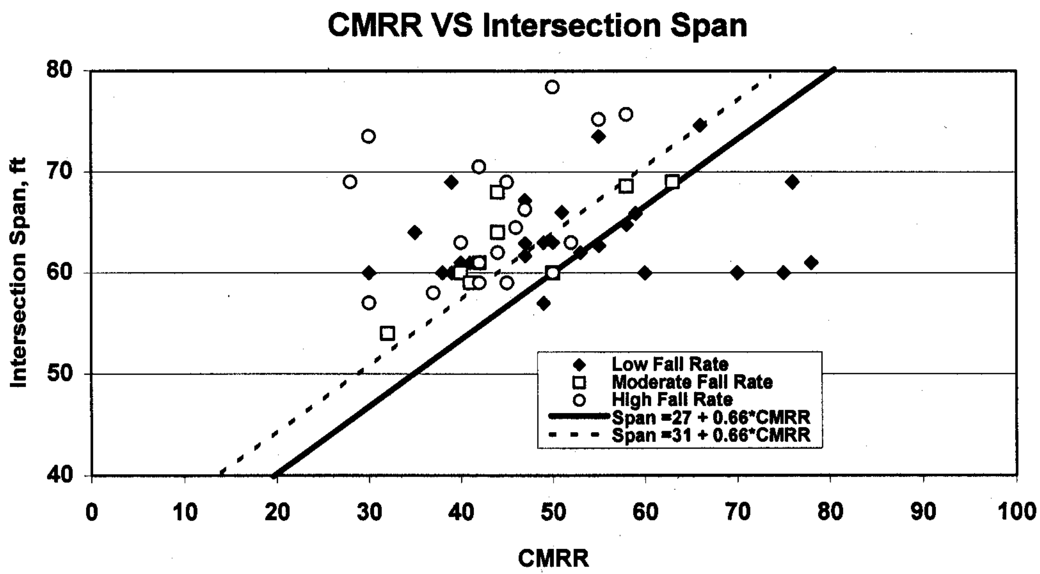


Figure 21.—Relationship between CMRR, intersection span, and roof fall rate.

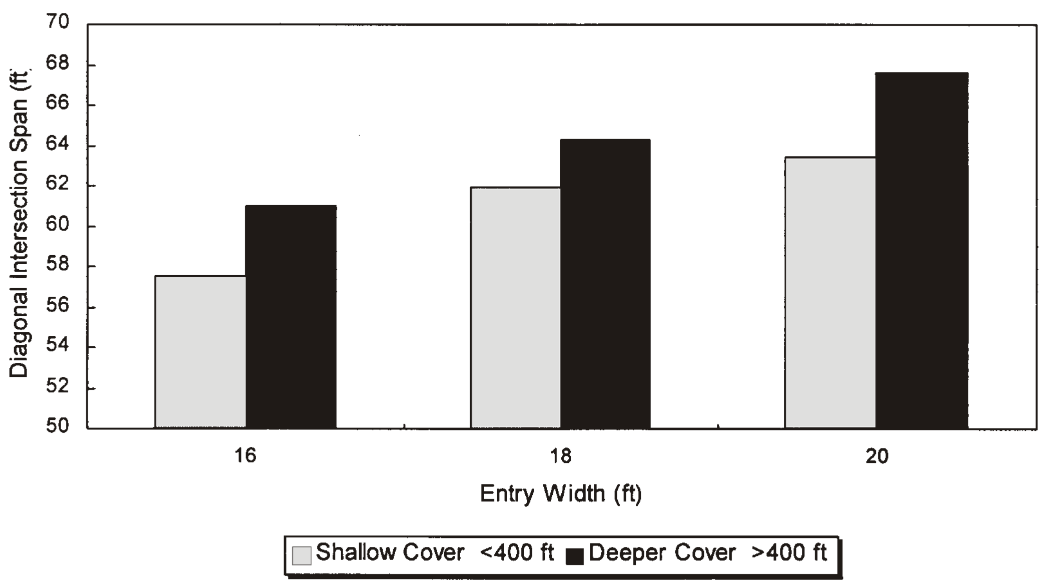


Figure 22.—The effect of overburden on intersection diagonal.

CONCLUSIONS

Data on roof quality and roof bolt performance were collected from interviews and underground reconnaissance at 41 U.S. coal mines. The roof fall rate (falls per 10,000 ft of drivage) and the four-way intersection fall rate (roof falls in four-ways/total number of four-ways) were developed as the outcome variables for analyzing the influence of numerous geotechnical variables on stability. From the data it was determined that higher roof fall rates were more common in the lower CMRR range (CMRR $\leq$ 50). Intersections were much more likely to fall than entry segments, and four-way intersections were more likely to fall than three-way intersections. When the data were divided into two groups by depth of cover, a relationship between PRSUP and the CMRR was determined that could be used in design. The study determined that overburden depth could be used as a surrogate for stress level. Paired data extracted from the database show that increasing bolt length decreased the roof fall rate in 11 of 13 cases over a wide range of roof fall rate and bolt types. A useful relationship between the intersection span and the CMRR was also found. The data showed considerable scatter, which was attributed to variations in roof geology, horizontal stress, and bolt installation quality, none of which could be measured.

The method for constructing historic roof bolt maps and hazard maps using the CMRR was described. These methods of tracking roof quality and support performance will be valuable for individual mine operators.

REFERENCES

Chase FE, Mark C, Mucho TP, Campbell PL, Holbrook AD [1999]. The advance and relieve mining method: a horizontal stress control technique. In: Peng SS, ed. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: University of West Virginia, pp. 300-308.

Dolinar DR, Bhatt SK [2000]. Trends in roof bolt applications. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Dolinar DR, Oyler DC, Compton CS [1997]. Analysis of extensometer data from a room widening experiment designed to induce a roof fall. In: Peng SS, ed. Proceedings of the 16th International Conference on Ground Control in Mining. Morgantown, WV: University of West Virginia, pp. 289-295.

Mark C [1999a]. Application of the Coal Mine Roof Rating (CMRR) to extended cuts. Min Eng *51*(4):52-56.

Mark C [1999b]. Empirical methods for coal pillar design. Proceedings of the Second International Workshop on Coal Pillar Mechanics and Design. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 99-114, IC 9448, pp. 145-154.

Mark C [2000]. Design of roof bolt systems. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Mark C, Mucho TP [1994]. Longwall mine design for control of horizontal stress. In: New Technology for Longwall Ground Control; Proceedings: U.S. Bureau of Mines Technology Transfer Seminar. Pittsburgh, PA: U.S. Department of the Interior, U.S. Bureau of Mines, SP 01-94, pp. 53-76.

Mark C, Chase FE, Molinda GM [1994]. Design of longwall gate entry systems using roof classification. In: New Technology for Longwall Ground Control; Proceedings: U.S. Bureau of Mines Technology Transfer Seminar. Pittsburgh, PA: U.S. Department of the Interior, U.S. Bureau of Mines, SP 01-94, pp. 5-18.

Molinda GM, Mark C [1994]. Coal Mine Roof Rating (CMRR): a practical rock mass classification for coal mines. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9387.

Mucho TP, Mark C, Zelanko JC, Compton CS [1995]. Roof support performance in high stress conditions. In: Peng SS, ed. Proceedings of the 14th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 30-38.

Parker J [1973]. Practical rock mechanics for the miner. Part 5: how to design better mine openings. Eng Min *Dec*:76-80.

Salamon MDG [1989]. Some applications of the frictionless laminated model. In: Proceedings of the 30th U.S. Symposium on Rock Mechanics. Morgantown, WV: West Virginia University, pp. 503-510.

Signer SP, Lewis JL [1998]. A case study of bolt performance in a two-entry gateroad. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: University of West Virginia, pp. 249-256.

Wang Y, Peng SS [1996]. High horizontal stress effects on longwall gate entry stability. In: Ozdemir L, Hanna K, Haramy KY, Peng SS, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 179-191.

LOAD BEHAVIOR OF GROUTED BOLTS IN SEDIMENTARY ROCK

By Stephen P. Signer[1]

ABSTRACT

This paper presents an overview of laboratory and field tests on approximately 250 fully grouted roof bolts instrumented with strain gauges in order to study loading behavior. Laboratory work included pull tests, time-dependent tests, and shear tests. The field tests were conducted in 14 different mines extracting different commodities. However, because of the focus of this publication, only the results of tests in 9 coal mines are reported here. In the field tests, all but 14 bolts were loaded by rock movement; these 14 bolts were tested using pull gear. The variables studied included anchorage length, mechanical interlock, time-dependent behavior, shear, installation load, loading rate, and maximum load level. Such information will improve the understanding of the behavior of resin-grouted bolts, which in return will enhance the safety of miners by reducing the occurrence of rock falls. The influences of geology, overburden depth, entry width, mining-induced stresses, and bolt spacing are not described.

[1] Mining engineer, Spokane Research Laboratory, National Institute for Occupational Safety and Health, Spokane, WA.

INTRODUCTION

Millions of roof bolts are installed in U.S. coal mines each year to prevent ground falls. Despite the importance of entry stability, there is a lack of understanding of how roof bolts provide reinforcement to the mine rock. Bolt loads are affected by many variables, such as changes in mining conditions (geology, geometry, and in situ stress fields) and bolt properties (diameter, length, spacing, and stiffness). This lack of understanding has contributed to our inability to prevent rock falls, which remains one of the most significant safety hazards in underground mines. The Spokane Research Laboratory (SRL) of the National Institute for Occupational Safety and Health (NIOSH) has conducted research to study the behavior of grouted roof bolts and how they reduce support failure and subsequent ground falls. The purpose of this paper is to summarize important facts and principles obtained from the completed studies. Because of space limitations, we will refer to previously published material that contains detailed information on each individual test setup and variables.

The first investigation [Serbousek and Signer 1987] was a study of the axial elastic behavior of grouted bolts installed in concrete blocks. The purpose was to find out how load was transferred both between the bolt and the grout and the grout and the rock by using a known loading condition uninfluenced by geologic variations. Strain gauges were installed to measure load changes along the length of the bolt. More than 50 pull tests were performed in which applied loads were restricted to the elastic range of the bolt steel. Variations were made in hole size, bolt length, grout type, and grout strength. Results of an axisymmetric, finite-element numerical model were compared with these test results.

The second phase of research [Signer 1990] was performed at four different coal mines in Colorado, Illinois, and Pennsylvania. Fourteen instrumented bolts were installed in sedimentary rock and tested using the same procedures as employed in the laboratory tests. In addition, these bolts were loaded to failure to study their nonlinear behavior. The purpose of this investigation was to verify the results of the axial elastic laboratory studies in the field where geology became a variable. A second goal was to study load transfer mechanisms when bolts were loaded past their elastic limit.

A third study [Signer 1988] involved laboratory tests in concrete blocks to study the creep behavior of grouted bolts instrumented with strain gauges. Variations were made in bolt length and grout type. The loads were applied with pull gear and held constant for long periods of time. The purpose of this investigation was to see if a constant load applied to the bolt head would propagate along the length of the bolt over time.

Laboratory tests [McHugh and Signer 1999] were also conducted to study the axial and bending behavior of grouted bolts subjected to horizontal joint movements. These tests were conducted on 17 instrumented bolts installed perpendicular to a joint surface in 0.6- by 0.6-m (2- by 2-ft) square concrete blocks. A shear force was applied until the bolts failed.

The last phase of research consisted of field studies in which fully grouted bolts instrumented with strain gauges were installed as supports and loaded by mining-induced stress changes [Signer and Jones 1990; Signer et al. 1993; Maleki et al. 1994; Larson et al. 1995; Signer and Lewis 1998]. These bolts were placed in 14 mines (9 coal, 2 trona, 2 gold, and 1 platinum). Variations in bolt length, bolt spacing, bolt diameter, geology, geometry, stress fields, and mining methods were recorded to study the response of grouted bolts under actual field conditions. These investigations have just been completed, and a complete data analysis is ongoing.

BEHAVIOR OF GROUTED ROOF BOLTS

BOLT LOADING

Roof bolts are loaded in four ways: axial, bending, shear, and torsion (figure 1). Torsional loading applies only to bolts that are actively tensioned by rotation and are not covered in this paper. Steel bolts are weakest in shear and strongest in axial. Although axial bolt loads result from by both vertical and horizontal rock movement, most of the axial load is produced by vertical rock movement. Because there is a large difference in strength between the steel and the rock, actual shear failures of the steel are not common in sedimentary deposits. If a shear plane develops in the bolted rock mass and horizontal movement occurs, then the bolt will be subjected to a combination of axial and bending loads. A commonly observed phenomenon is the S-shaped failure shown in figure 2. This characteristic is actually a combination of axial and bending forces.

VERTICAL ROCK MOVEMENT

A grouted bolt is a passive support system, which means that bolt loads are caused by rock movements. Additional rock movement increases load on the bolt and reduces and/or stabilizes movement in the mine rock. Vertical rock movements produce axial loading in the bolt. Axial loads are transferred between the bolt and the rock by shear resistance in the grout. This resistance is the result of mechanical interlock in which load is transferred between the steel bolt and the grout and the grout and the rock via contact surfaces. Bolt hole walls have voids and irregularities created by the drilling process. Steel bolts are rolled with ribs to provide anchorage. Grout fills these irregularities and voids in the walls and bolts if the bolt has been properly installed. Localized deformation and crushing will occur in the grout at the contact points before the system

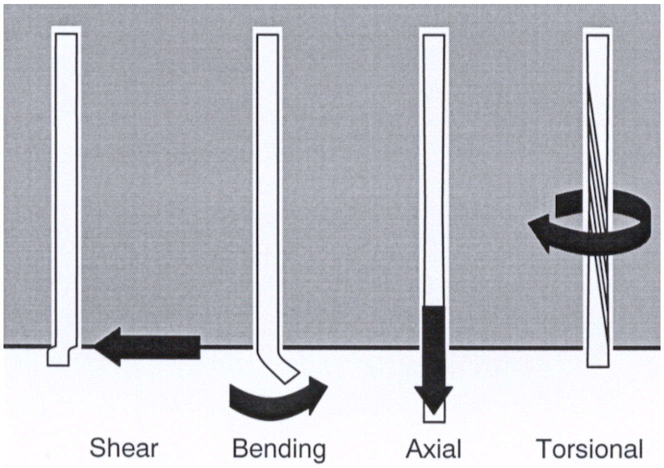


Figure 1.—Types of roof bolt loads.

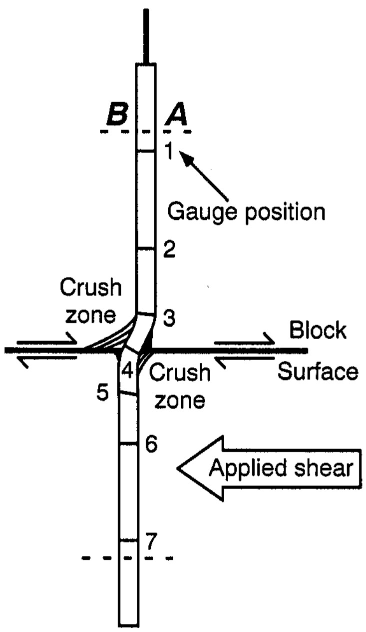


Figure 2.—Bolt response to shear movement.

tightens up, which allows additional deflection in both the bolt and the rock. Mechanical interlock will cause shear forces to be transferred from one medium to another until the maximum shear strength of the grout and/or rock is reached. At that point, the weakest material will fail, and friction will control load transfer. The rate of load transfer is similar to an exponential decay curve and is dependent on the material properties of the bolt, the grout, the rock, and the respective interfaces.

The amount of bolt length necessary to transfer the ultimate axial load capacity of the bolt to the rock is called anchorage length or critical embedment length and is a function of the strength of both the rock and the grout. If anchorage length is not long enough, then the grout or rock interfaces will fail, and the bolt will pull out of the hole.

HORIZONTAL ROCK MOVEMENT

Horizontal movement of a rock joint will compress the surrounding rock against the grout. This produces shear loads in the bolt. If the forces exceed the compressive strength of the rock, the rock will fracture. When this occurs, shear loads will be reduced and will be replaced by a combination of both bending and axial loads. Bending loads are highest near the joint and will dissipate quickly along the length of the bolt. Axial loads will travel farther along the length of the bolt than bending loads and will interact with the grout and rock as noted above.

FAILURE

Various types of failure can occur when using grouted bolts. Failure can take place in the bolt, the grout, the rock, or at the bolt-grout or grout-rock interfaces. The type of failure depends on the characteristics of the system and the material properties of individual elements.

(1) If the rock is weaker than the grout and if anchorage length is inadequate, then failure of the bolt system will occur at the grout-rock interface, which is the weakest point. As the shear strength of this interface is exceeded, then failure will progress from the point of maximum load in the bolt down the length of the bolt.

(2) If the grout is weaker than the rock, then shear failure will occur in the grout at the bolt-grout interface. If the anchorage length is inadequate, then failure will progress along the length of the bolt. Shear stress is greater at the bolt-grout interface than it is at the grout-rock interface simply because there is less area at the bolt-grout interface. The bolt-grout interface is also more prone to failure because of Poisson's effect on the steel bolt, which causes the bolt to pull away from the grout as the bolt is loaded.

(3) If there is adequate anchorage length to develop the full capacity of the steel bolt, regardless of the properties of the grout and the bolt, then the bolt will fail if loading on the bolt exceeds the ultimate strength of the steel. However, prior to bolt failure, localized grout or rock shear failure will occur. This is because the bolt, which has a greater ductility, will take larger deflections than the rock or the grout.

RESULTS

Approximately 250 bolts instrumented with strain gauges were installed in both the laboratory and the field to study loading behavior on fully grouted roof bolts. Laboratory tests included pull tests, time-dependent tests, and shear tests. Although the field tests were conducted in 14 different mines extracting different commodities, because of the focus of this publication, only the results of tests in 9 coal mines are reported here. In the field tests, all but 14 bolts were loaded by rock movement; these 14 bolts were tested using pull gear. The influences of geology, overburden depth, entry width, mining-induced stresses, and bolt spacing are not described. The following discussion covers some of the major factors related to the performance of fully grouted bolts.

BOLTS LOADED UNDER ARTIFICIAL CONTROL

Anchorage Length and Mechanical Interlock

Elastic tests in which grout type, hole size, and bolt length were varied were conducted in the laboratory on 50 bolts [Serbousek and Signer 1987]. Results on the 1.2- and 0.6-m (4- and 2-ft) long bolts indicated that 56 cm (22 in) of bolt length were required to transfer 90% of the load from the bolt to the rock (figure 3). Polyester resin and gypsum grout were used with a 19-mm (0.75-in) bolt and installed in 25-mm (1-in) holes. Nineteen-millimeter (0.75-in) bolts were also tested in 35-mm (1-3/8-in) holes using gypsum grout. The variations in grout type and hole size had no statistically significant effect.

The results from the axial elastic test conducted on grouted bolts installed in shale compared well with the results from previous laboratory work. The average anchorage length for these bolts was slightly longer than for bolts installed in concrete blocks, even though the field test results showed more variability. The roof at the first mine site contained layers of weaker rock. Test results from this mine reflected the presence of these weaker layers as changes in the rate of load transfer. A weaker layer requires a longer anchorage length compared to that needed in stronger rock. The stiffness of the bolting system decreases in weaker zones due to slip at the grout-rock interface.

Laboratory and field studies of fully grouted bolts indicate that the average anchorage length for bolts in competent rock is 56 cm (22 in). Anchorage length in weak and broken rock must be determined by field pull tests of bolts with grout lengths less than 30 cm (1 ft). Anchorage length can also be affected if the bolt hole is smooth, which reduces the effect of mechanical interlock, so that the resisting force is mostly friction. The effect of a pull test on a 30-cm (1-ft) long bolt in a smooth hole is shown in figure 4.

Anchorage length depends on the material properties of the bolt, the grout, and the rock; the quality of the installation; the smoothness of the drill hole; and possibly other factors. Weaker grout and/or rock will require longer bolt anchorage lengths.

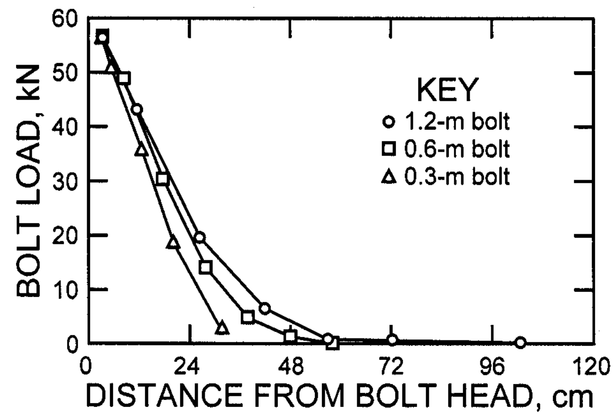


Figure 3.—Load transfer of No. 6 bolts in concrete blocks.

Figure 4.—A smooth hole lowers anchorage strength, as seen with this bolt as it pulls out of the hole.

Proper installation of the bolt is critical to the performance of the bolt. If the grout is inadequately mixed, is overspun, or is glove-fingered, then the capacity of the grout to provide mechanical interlock is severely impaired. Glove-fingering occurs when the plastic casing of the resin remains intact and causes a plastic interface between the grout and the rock. The bolt hole must be drilled with bits of appropriate sizes to produce holes of the proper diameter [Pettibone 1987].

Readings from seven bolts were averaged, and the results are shown in figure 5. Each curve represents load decay along the bolt length. The curve was established from readings of the applied load to the bolt and strain gauges. The length necessary to transfer all the load from the bolt to the rock was the same at different load levels. The slope of each curve is an indication of the stiffness of the system. Increasing the applied load resulted in higher stiffnesses, but the load transfer length remained the same, indicating that mechanical interlock between the bolt, the grout, and the rock was the primary mechanism for transference of load. If adhesion were the mechanism of load transfer, then stiffness would be the same for all elastic loads and the anchorage length would increase as a function of applied load. Friction could not be the load transfer mechanism because bolt deflections were elastic.

Fifteen instrumented bolts were tested past the yield point of the steel to study load transfer mechanisms prior to bolt failure. The results show that yielding of the steel will translate down the length of the bolt from 23 to 51 cm (9 to 20 in), depending on rock strength, bolt hole properties, and bolt installation quality. When the bolt has yielded, load is estimated using a load strain curve that was determined experimentally. These bolts were pulled an average of 5 cm (2 in) before failure.

Time-Dependent Behavior

Laboratory tests were conducted on six bolts to determine the time-dependent properties of grouted bolts. These bolts were instrumented with strain gauges and installed in 2.5-cm (1-in) holes with both gypsum and resin grout. Bolts that were 1.2, 0.6, and 0.3 m (4, 2, and 1 ft) long were tested at applied loads of 40, 58, 80, and 102 kN (9,000, 13,000, 18,000, and 23,000 lb). The strain gauges were monitored constantly to detect load changes along the bolt. Load was applied with hydraulic rams and maintained with hydraulic accumulators for a period of at least 1 month, or until the bolt stabilized.

Figure 6 shows the results for the 1.2-m (4-ft) long bolts. To determine the rate of load change per day, a linear fit was done at each strain gauge location beginning at day 10 and continuing to the end of the test. The resulting values were normalized by dividing the rate of load change per day by the load on the bolt at a given location. When the load on the bolt approached zero, the data were deleted because of problems caused by dividing by zero. Bolts installed with gypsum grout showed three to five times more creep than bolts installed with resin grout.

Shear Tests

Laboratory tests on 17 bolts were conducted to study the behavior of roof bolts subjected to shear loading over a range of axial bolt loads. Fourteen strain gauges were attached to each bolt to measure both axial and bending loads. The instrumented bolts were grouted through two high-strength concrete blocks, and axial tension up to 75% of the yield strength was applied to the bolts. The block interface was smooth and acted as the failure plane. Shear loads were applied to the blocks until the bolts failed due to shear movement of the joint. The tests characterized the relationship of axial and bending loads on the bolt to shear forces across a rock bedding plane.

The results showed that (1) axial bolt loading had little to no effect on the capacity to resist shear loading of a joint, (2) the bolts failed as a combination of axial and bending loads rather than shear loads, (3) the bending loads caused by joint movement dissipated within a few inches of the joint, but axial loads were transferred along the bolt length for the distance of the anchorage length, and (4) all but one bolt failed within 5 cm (2 in) of horizontal movement of the joint.

BOLTS LOADED BY ROCK MOVEMENT

Installation Loads

When a fully grouted bolt is installed, the upward thrust of the roof bolter will compress the weakest rock layer in the immediate roof. The amount of compression will depend on the distance from the roof line, the strength of the surrounding rock layers, and the amount of upward thrust of the roof bolter. The bolt will be held in place until the grout hardens. When the force from the roof bolter is removed, the compressed roof rock will rebound, which produces a resisting force in the grouted bolt.

Installation loads on 40 strain-gauged roof bolts from 4 different coal mines are shown in figure 7. The data represent an average of 420 strain gauges. Some bolts had 10 strain gauges per bolt, and some bolts had 12 strain gauges per bolt. The average installation load was 11 kN (2,400 lb) with a standard deviation of 8.5 kN (1,900 lb). This means that installation loads tended to vary from zero to 20 kN (4,500 lb).

The highest initial load was usually near the roof line and decreased as the distance from the roof line increased. The amount of force developed depended on the properties of the immediate roof, the ability to apply the upward thrust of the roof bolter, and the behavior of the roof bolter operator. That is, often a bolter operator would not apply the same amount of thrust to the specially instrumented roof bolts as to a normal bolt. The amount of time elapsed from installation to the first measurement could affect the readings if the mine roof were active. If the hole was not drilled deep enough, the bolt could bottom out.

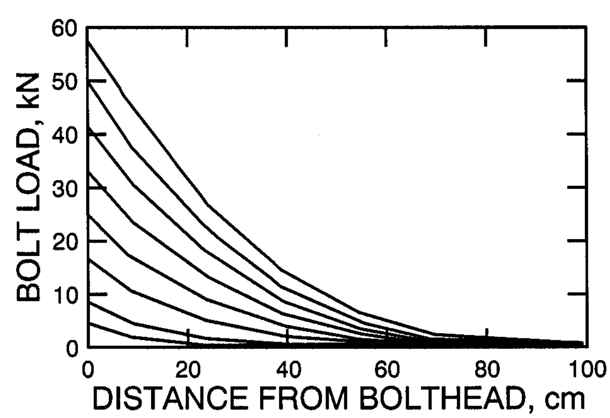


Figure 5.—Results of pull tests on instrumented bolts.

Loading Profiles

Figure 8 shows axial loading behavior of a typical fully grouted roof bolt. When the bolt is first installed, there is an initial load that varies along the length of the bolt. As mining progresses, redistribution of rock stresses change the stress pattern in the immediate mine roof. A fully grouted bolt responds to the stress changes by increasing in load. The distribution of load along the length of the bolt varies in proportion to the stress pattern. After primary mining has reached a sufficient distance away from the test area, the load on the bolt increases at a slower rate, stabilizes, and in some cases decreases. If secondary mining occurs and the immediate roof is within the zone of influence from abutment stress changes, then the bolts will go through the same cycle of load increase and stabilization. This sequence assumes that there are enough bolts installed to cause the immediate roof to stabilize.

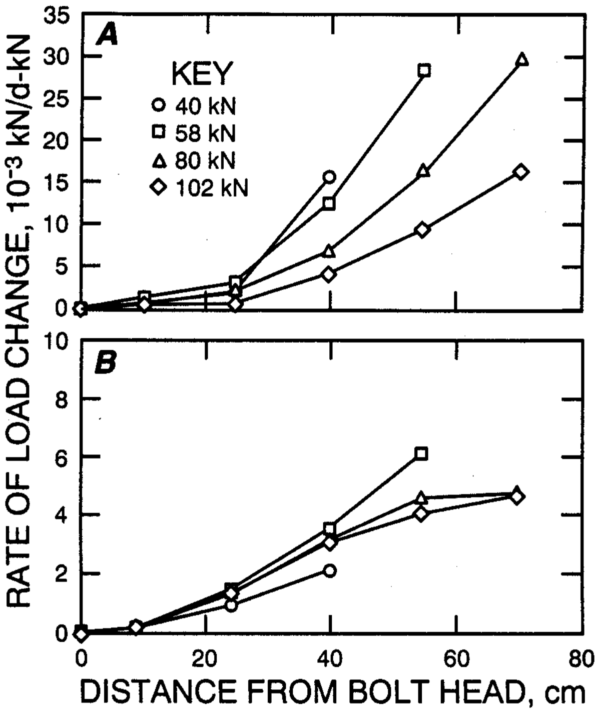


Figure 6.—Time-dependent behavior of (*A*) gypsum-grouted bolts and *(B)* resin-grouted bolts.

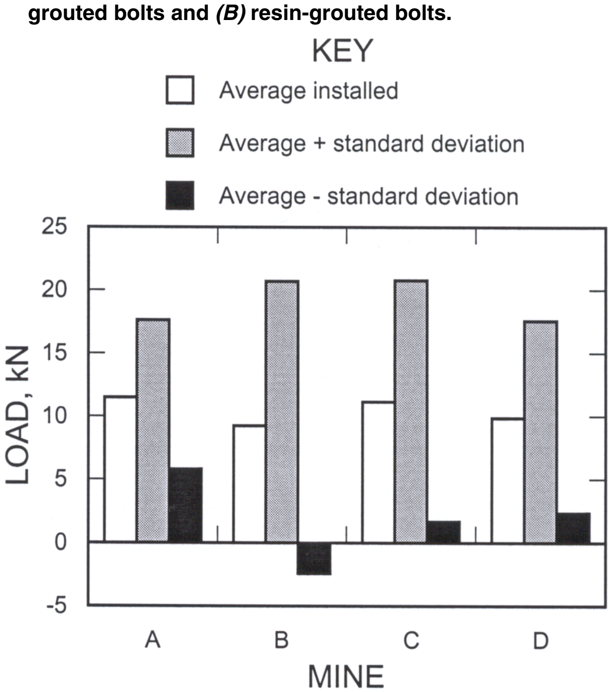


Figure 7.—Installation loads on instrumented bolts.

Loading Rates

The rate at which grouted bolts increase in load depends on the rate of rock movement in the immediate roof. Stress changes in the rock result in movement. The amount of movement depends on the rock and joint modulus and strengths. Strain gauges on bolts can measure rock movement several orders of magnitude finer than can other deflection measurement systems. For example, 0.05 mm (0.002 in) of movement in a bolt section 2.5 cm (1 in) long will result in 2,000 microstrain, which is the yield strain for a No. 6 slotted bolt.

Initial loading rates on 62 strain-gauged roof bolts installed in 6 different coal mines are shown in figure 9. The initial loading rate was calculated by a linear fit of the change in axial loads right after installation. The duration of this loading rate varied significantly from one site to another, as did the load level attained during initial loading.

The average rate of change of bolt load resulting from entry development varied from 1 to 16 kN/d (250 to 3,500 lb/d). Each bar on the graph represents an average of all the strain gauges at each mine site. Each site had significant variations in loading rates among bolts and even among gauges on each bolt. The maximum loading rate at each site is significant because highly

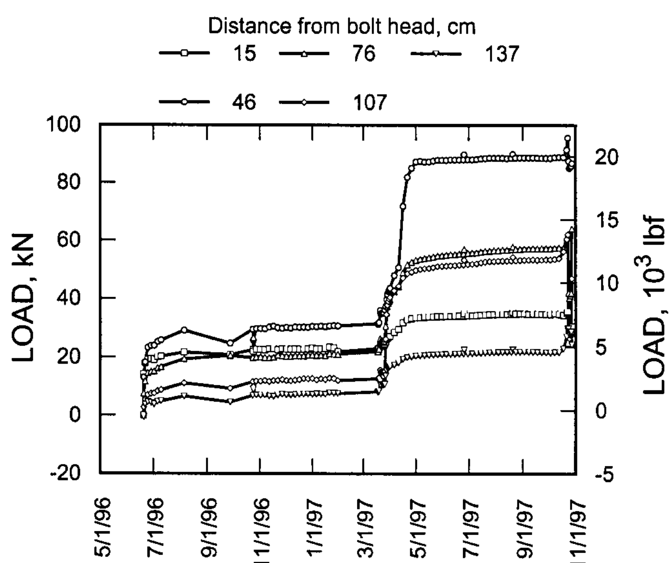


Figure 8.—Typical bolt load response to mining-induced stresses.

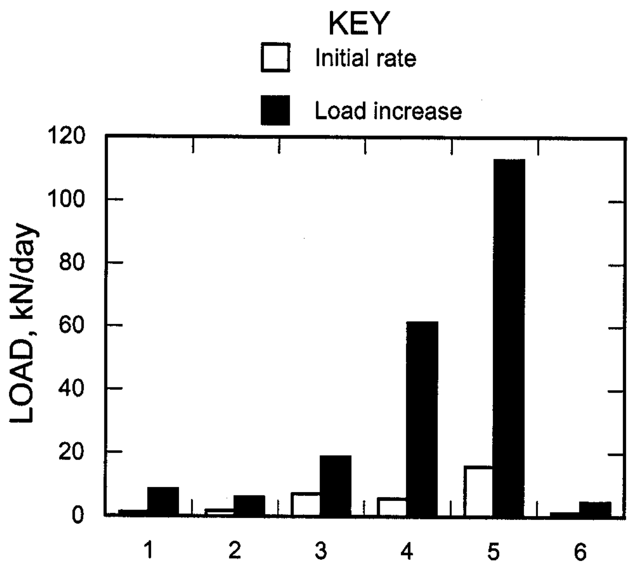


Figure 9.—Initial load rate and rate of bolt load increase after installation.

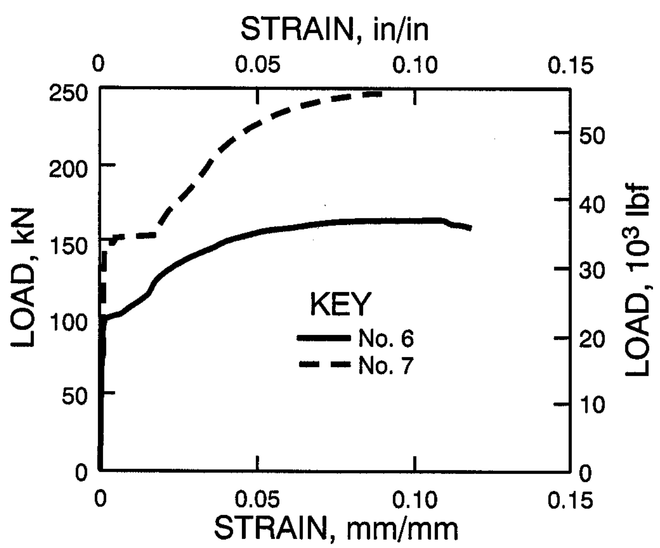


Figure 10.—Strain-load curve for No. 6 and No. 7 slotted bolts.

Table 1.—Properties of bolts

Bolt type	Yield load		Ultimate load		Cross-sectional area	
	kN	lb	kN	lb	cm^2	in^2
No. 6	107	24,000	176	39,500	2.58	0.40
No. 6, slotted . .	96	21,500	160	36,000	2.38	0.37
No. 7	160	36,000	270	60,500	3.87	0.60
No. 7, slotted . .	149	33,500	249	55,900	3.61	0.56

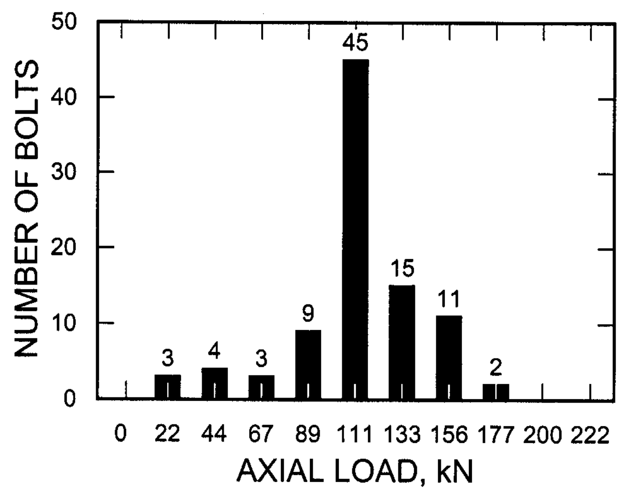


Figure 11.—Maximum axial loads on bolts in all coal mines.

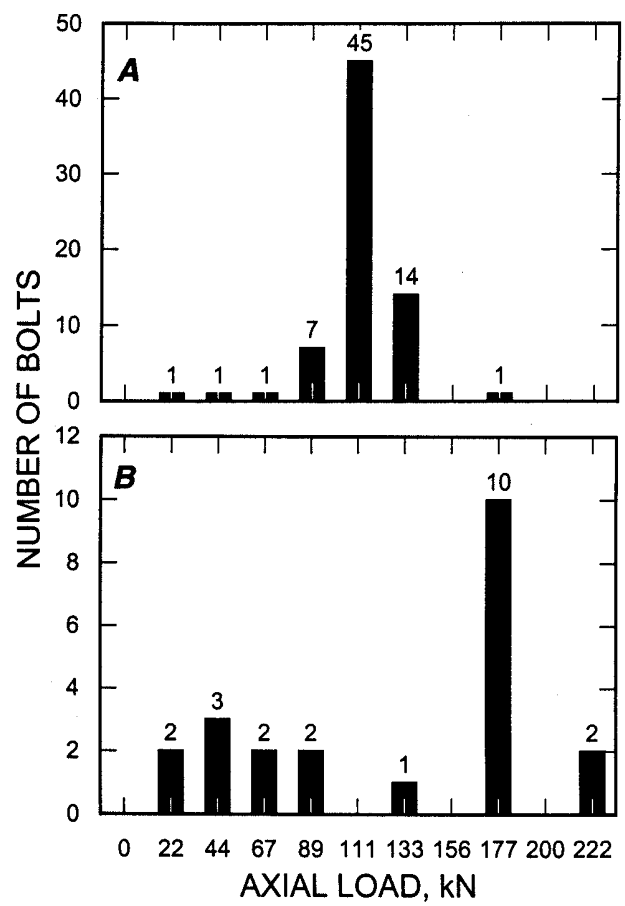


Figure 12.—Maximum axial loads on *(A)* No. 6 bolts and *(B)* No. 7 bolts.

loaded bolts fail first. The maximum loading rate after bolt installation varied from 4.4 to 105 kN/d (1,000 to 23,500 lb/d). There are many factors that cause these variations, including geology, seam height, entry width, pillar size, and stress field.

MAXIMUM BOLT LOADS

Figure 10 shows the load-strain behavior of slotted bolts, and table 1 compares the yield and ultimate strengths of slotted bolts and regular rebar bolts. Bolt row spacing for the instrumented bolts was reduced to compensate for the 10% reduction in strength. In some cases, the instrumented bolts were installed as supplementary support. Also, No. 7 strain-gauged bolts were sometimes installed where No. 6 bolts were used as the support system.

The distribution of maximum axial loads, i.e., the maximum load on each instrumented bolt, is shown for all bolts installed in coal mines in figure 11 and for No. 6 bolts and No. 7 bolts in figure 12*A* and 12*B*, respectively. This value, rather than average load, is what would cause a bolt to break. Maximum axial load on the majority of the bolts exceeded the yield point of the steel, but was less than ultimate load. Maximum load on the instrumented bolts can be used as an estimate for selecting bolt size and spacings.

These data represent loading on 92 instrumented bolts at eight different mine sites. Seventy-five percent of the instrumented bolts reached the yield point of the steel on at least one gauge location, and 50% of the bolts exceeded the yield point of the steel. These values represent bolt loading resulting from a variety of loading conditions. Most were installed in longwall gate road entries; loading represents both passes of the longwall. Fourteen bolts were installed in a longwall recovery room where the roof support failed.

CONCLUSIONS

The anchorage length for grouted bolts installed in competent rock is 56 cm (22 in) and is established by mechanical interlock of the grout. Bolts loaded to failure show that yield will translate down the length of the bolt from 23 to 51 cm (9 to 20 in) and will be deflected an average of 5 cm (2 in) before failure. Load creep on bolts installed with polyester resin grout was shown to be minimal compared with creep on bolts installed with gypsum grout. Results of shear tests showed that the bolts failed from a combination of axial and bending loads rather than shear loads and that axial bolt loading had little effect on the capacity to resist shear loading of a joint.

The average load on fully grouted bolts just after installation was 11 kN (2,400 lb) and increased anywhere from 1 to 105 kN/d (250 to 23,500 lb/d). The data representing 92 instrumented bolts at 8 different coal mine sites showed that 75% of the bolts reached the yield point of the steel (0.2% strain) and 50% of the bolts exceeded the yield point of the steel.

The results of this study will increase understanding of the behavior of roof bolts in underground mines, which in turn will enable miners to select the appropriate bolts for reducing roof falls, thereby increasing workplace safety.

REFERENCES

Larson MK, Stewart CL, Stevenson MA, King ME, Signer SP [1995]. A case study of a deformation mechanism around a two-entry gate-road system involving probable time-dependent behavior. In, Peng SS, ed. Proceedings of the 14th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 295-304.

Maleki H, Signer SP, King ME, Edminster PA [1994]. Evaluation of support performance in a highly stressed mine. In: Peng SS, ed. Proceedings of the 13th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 9-17.

McHugh E, Signer SP [1999]. Roof bolt response to shear stress: laboratory analysis. In: Peng SS, ed. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 232-238.

Pettibone HC [1987]. Avoiding anchorage problems with resin-grouted roof bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9129.

Serbousek MO, Signer SP [1987]. Linear load transfer of fully grouted roof bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9135.

Signer SP [1988]. Comparative studies in the mechanics of grouted roof bolts. In: Peng SS, ed. Proceedings of the Seventh International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 282-288.

Signer SP [1990]. Field verification of load transfer mechanics of fully grouted roof bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9301.

Signer SP, Jones SD [1990]. A case study of grouted roof bolt loading in a two-entry gate road. In: Peng SS, ed. Proceedings of the Ninth International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 35-41.

Signer SP, Lewis JL [1998]. A case study of bolt performance in a two-entry gateroad. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 249-256.

Signer SP, Mark C, Franklin G, Hendon G [1993]. In: Peng SS, ed. Comparisons of active versus passive bolts in a bedded mine roof. In: Peng SS, ed. Proceedings of the 12th Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 16-23.

SUMMARY OF FIELD MEASUREMENTS OF ROOF BOLT PERFORMANCE

By Christopher Mark, Ph.D.,[1] Dennis Dolinar,[2] and Thomas P. Mucho[3]

ABSTRACT

During the 1990s, the former U.S. Bureau of Mines conducted a number of field studies in which the performance of different types of roof bolts were evaluated in different geologic environments. The studies used a standard suite of measurements, including multipoint extensometers, strain-gauged roof bolts, and roof bolt load cells. The sites were chosen to investigate the effect of a variety of parameters, including installed tension, bolt capacity, grout annulus, and horizontal stress orientation. Although not fully successful, the measurements provided valuable insights into each of these issues. They also showed that instrumentation and monitoring have important advantages over observational methods for comparing the performance of different roof bolting systems.

[1]Supervisory physical scientist.
[2]Mining engineer.
[3]Supervisory physical scientist.
Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

Roof bolts interact with the ground to create a reinforced rock structure. The mechanics of this interaction are difficult, if not impossible, to replicate in the laboratory. Field studies are essential to developing an understanding of how factors such as bolt tension, bolt length, bolt capacity, and resin annulus contribute to the support of real rock masses. Detailed measurements of bolt loads and roof movements can provide the information necessary to build conceptual and numerical models of supported mine roof.

Field studies are also the only way to compare the overall effectiveness of different roof bolt systems. In the United States, such comparisons are usually made by visual observations rather than by measurements. If an area supported by one type of bolt experiences less roof degradation or fewer roof falls than an area supported by another type, then the first bolt is deemed superior [Stankus 1991]. This observational approach, however, has limitations. Often, significant roof movements can occur without visual evidence at the roof line. Waiting to see how many roof falls occur can be expensive, particularly if large areas of the mine were supported with a particular bolt before its inadequacies became apparent. Again, instrumentation can provide an alternative. Measurements can show that bolts are overloaded or that the roof is becoming unstable long before there is any visual evidence.

Studies of roof bolt behavior have a long history in the United States. Some of the classic early work with strain-gauged resin bolts was performed by Karabin and Debevec [1976] and Haas [1981], followed by Serbousek and Signer [1987] and Signer [1990]. Other insights regarding the interaction between roof bolts and the rock mass came from researchers in Australia [Gale 1991; Hurt 1992].

In the early 1990s, the former U.S. Bureau of Mines (USBM) embarked on a major program of roof bolt field studies. One group of studies focused on the behavior of fully grouted, nontensioned resin bolts and is reported by Signer [2000]. The second group of studies, which is described here, had two main goals:

1. To study fundamental aspects of roof bolt performance by comparing different types of bolts in a variety of geologic environments; and
2. To develop an effective instrumentation plan for evaluating roof bolt systems at a particular site.

Ultimately, studies were conducted at 12 sites in 7 mines (mines A through G; see table 1). Most of the studies were conducted under cost-sharing Memorandums of Agreement between the USBM and cooperating coal companies. Unfortunately, the program ended in 1995 when the closing of the USBM resulted in reduced funding for ground control research.

Table 1.—Summary of field studies

Mine	Reference	State	Seam	Depth, m (ft)	Mining method	CMRR	Bolt type	Bolt capacity, kN (tons)	Length, m (ft)	Comments
A	Mucho et al. [1995]	PA	Lower Kittanning	275 (900)	Longwall	48	Fully grouted resin	200 (22.5)	1.8 (6)	Two different hole sizes.
B	Mucho et al. [1995]	PA	Sewickley	180 (600)	Longwall	40	Point-anchor tension	200 (22.5)	2.1 (7)	No instrumented bolts.
C	Mucho et al. [1995]	KY	Kellioka	130 (400)	Room-and-pillar	47	Fully grouted resin	115 (13)	1.8 (6)	No instrumented bolts.
							Fully grouted tension	115 (13)	1.8 (6)	No instrumented bolts.
D	Mucho et al. [1995]; Mark et al. [1998].	PA	Pittsburgh	250 (800)	Longwall	35	Point-anchor tension	135 (15)	2.5 (8)	—
							Point-anchor tension	85 (9.5)	2.5 (8)	Poor anchors reduced capacity.
E	Mark et al. [1998]	PA	Pittsburgh	250 (800)	Longwall	35	Fully grouted resin	115 (13)	1.5 (5)	No instrumented bolts.
							Point-anchor tension	150 (16.5)	2.5 (8)	—
F	Signer et al. [1993].	AL	Blue Creek	670 (2,200)	Longwall	47	Fully grouted resin	100 (11.5)	1.8 (6)	—
							Point-anchor tension	160 (17.5)	1.8 (6)	—
G	Campoli et al. [1996]	PA	Lower Kittanning	100 (300)	Room-and-pillar	50	Point-anchor tension	100 (11.5)	1.5 (5)	Third pattern of mixed bolts.
							Fully grouted resin	75 (8.5)	1.8 (6)	No instrumented bolts.

NOTE.—Point-anchor tension bolts were all resin-assisted mechanical bolts.

STANDARD INSTRUMENTATION

The studies usually began with an assessment of roof geology using the Coal Mine Roof Rating (CMRR). The CMRR rates the structural competence of the roof and allows the roof at different sites to be compared on a single scale [Molinda and Mark 1994]. Instrumentation sites also usually included a 1-in (25-mm) diameter, 16-20 ft (5-6 m) vertical borehole for logging with a stratascope. Stratascope surveys provide a means to assess interfaces, bedding, or other geological features; identify general roof rock lithology; and observe bed separations.

Several means were used to monitor support loads in the studies. For fully grouted resin bolts, strain gauges were mounted in slots machined in the roof bolts [Signer 2000]. For resin-assisted point-anchor systems, loads were monitored using electronic or hydraulic load cells mounted between the plate and the bolt head. Further details may be found in the original reports [Signer et al. 1993; Mucho et al. 1995; Campoli et al. 1996; Mark et al. 1998].

Roof movements were monitored using extensometers, convergence stations, and observation holes. One to three extensometers were included in each instrumented area depending on the detail of roof movement required by the investigation. Most of the studies used the Sonic Probe type of extensometer. The Sonic Probe measures the distance between magnetic anchors placed in a 38-mm (1.5-in) diameter borehole to a claimed accuracy of 0.025 mm (0.001 in). As many as 20 anchors can be placed in a 6-m (20-ft) long vertical borehole.

The data from the extensometer locations can be presented as deformation in each interval or as percent strain. Strain is determined by dividing the movement between the anchor intervals by the original length of the interval. Multiplying this strain by 100 yields percent strain for the interval. A rule of thumb developed abroad is that 1% strain measured above the bolts is an unstable condition [Hurt 1992]. The concept is that once a roof bed experiences 1% strain, it fails and can no longer carry horizontal stress, thus forcing the stresses to move higher into the roof. If a roof bed fails above the bolts, it may indicate a loss of ground control. Large roof strains within the bolted horizon are of much less concern. One goal of the studies was to test whether the concept of roof strain was useful for U.S. conditions.

In the course of the studies, a standard instrumentation plan evolved (figure 1). Unfortunately (as the comments in table 1 indicate), it was not possible to install all of the instrumentation at every site. No instrumented bolts were used at five of the sites, and simple three-point extensometers were used at two others.

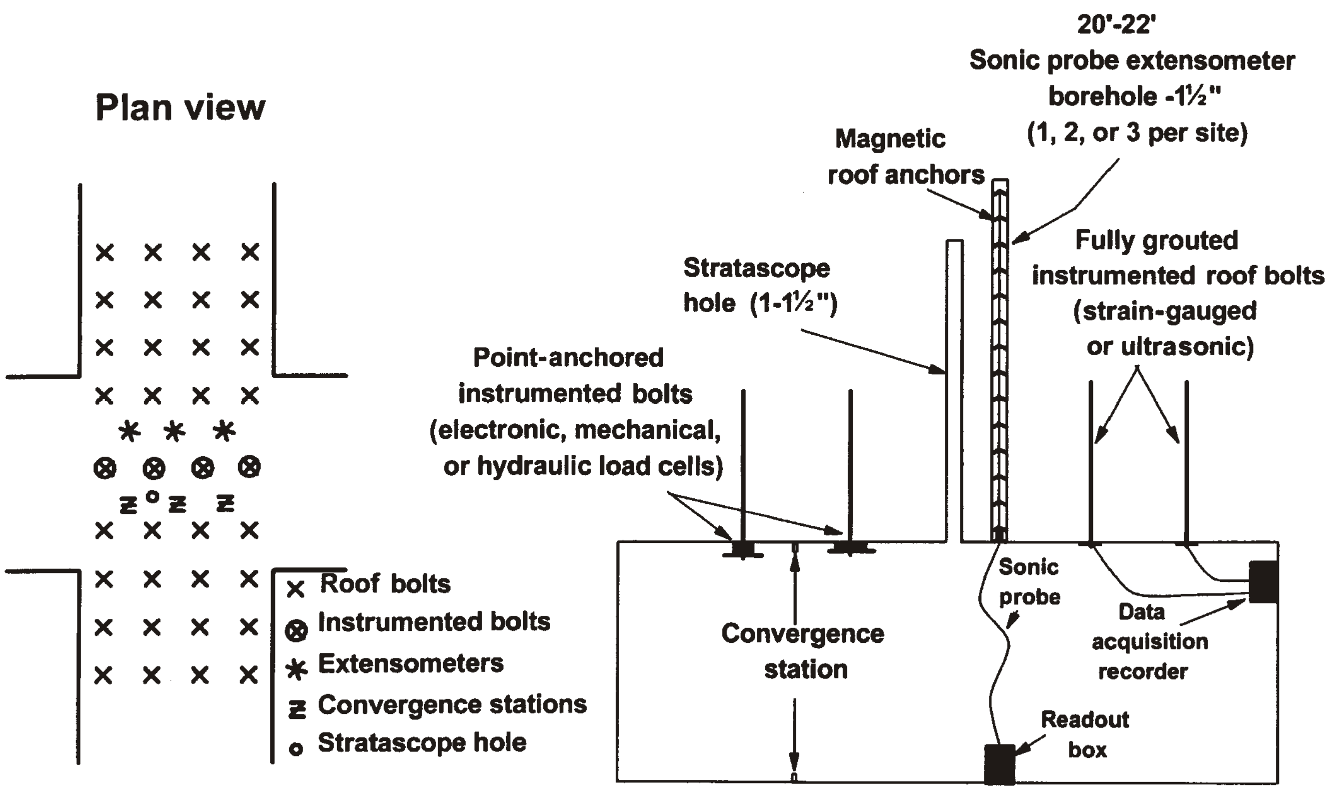


Figure 1.—Standard instrumentation for field evaluation of roof bolt performance.

DISCUSSION

Brief descriptions of the field sites and the individual results are included in the appendix to this paper. Ultimately, the number of sites was too small to provide definitive answers to any of the fundamental questions about bolt mechanics. Moreover, they do not allow a multivariate analysis, in which interactions between the geologic and design factors might be assessed. Nevertheless, the studies provided some valuable insights into the effects of these factors on roof bolt performance, which are discussed below.

GEOLOGY

The tests were performed in a wide variety of geologic environments, with roof rocks that included coal, underclay, shale, "stack rock" sandstone, and siltstone. The CMRR values shown in table 1, however, indicate that the sites can be classified into two groups. The CMRR was 40 or less at five of the sites (mines B, D, and E); the roof there could be described as "weak." At the remaining seven sites, the CMRR was between 47 and 50, which means that the roof strength was "moderate."

Roof strains in excess of 1% were measured above the bolt anchorage at five sites, although no roof falls occurred. Three of these were weak roof sites in the vicinity of longwalls, indicating that weak roof is more likely to experience large roof movements than moderate roof. Large strains were measured within the bolted horizon at many sites in all types of roof, however. No clear differences in bolt loading patterns emerged among the different geologies.

HORIZONTAL STRESS

Horizontal stress was clearly a major factor at many of the sites. At two mines (B and C), extensometers were placed in entries and crosscuts that were oriented in "good" and "bad" directions relative to the horizontal stress. In both cases, the roof strains were at least three times greater in the "bad" direction.

Several studies showed the effect of the "horizontal stress abutment" due to longwall mining that has been described by Mark et al. [1998]. The sites at mines A, D, and F were located in longwall gate entries that were subjected to horizontal stress abutments. Large roof movements were measured at each site, and the majority of the instrumented roof bolts approached or exceeded the yield point. In contrast, the sites at mine E were located in a stress shadow, and no new roof movements occurred as the longwall face approached. In fact, the bolt loads even decreased slightly!

INSTALLED TENSION

One of the most controversial issues in roof bolting is the im-portance of installed tension. Three studies compared tensioned to nontensioned bolts, but the results were ambiguous.

The study at mine C compared tensioned and nontensioned fully grouted bolts. Greater movements were measured at the nontensioned site, primarily within the bolted horizon. The presence of a preexisting cutter in the roof at the nontensioned site, however, may have influenced the results.

At mine G, resin-assisted point-anchor tensioned bolts were compared with nontensioned fully grouted resin bolts. At a third site, the fully grouted bolts were supplemented by some resin-assisted point-anchor bolts. The point-anchor bolts were 1.5 m (5 ft long), while the fully grouted bolts were 1.8 m (6 ft) long. Out of a total of seven instrumented intersections, two experienced strains in excess of 1% above the bolts. One was in the point-anchor site, but the other was one where the fully grouted bolts had been supplemented by point anchors. Overall, the differences among the three bolting systems were probably not statistically significant.

The study at mine F was probably the most informative of the three. Here, the resin-assisted point-anchor bolts clearly provided better roof control than the nontensioned, fully grouted bolts. The point-anchor bolts were also 60% stronger, however, and their greater capacity may have accounted for their better performance. More surprising were the bolt load measurements. These showed that although the fully grouted bolts were installed without tension, within days their loads were equal to or greater than those of the point-anchor bolts. The loadings on the two systems continued to increase at approximately the same rate as mining progressed, up until the point where most of the fully grouted bolts were loaded beyond their yield point.

BOLT CAPACITY

As previously mentioned, the study at mine F found that the higher capacity bolts performed better, although installed tension may have been a contributing factor. However, mine F subsequently switched to higher capacity, nontensioned, fully grouted bolts and used them successfully for many years.

The study at mine D provided a clearer association between greater capacity and better roof control. Here, the two types of resin-assisted point-anchor tension bolts that were compared were nearly identical except for the length of the resin used to assist the mechanical shell. The bolts with the shorter length of resin proved to have 40% less capacity due to inadequate anchorage. The measurements showed that both types of bolts loaded approximately in tandem until the short-anchor bolts slipped. Then the loads on the short-anchor bolts diminished while the roof movements accelerated. Ultimately, a 6% roof strain was measured above the anchorage. In contrast, the loads continued to increase on the long-anchor bolts, and roof control was maintained.

The mine D study also demonstrated the value of instrumentation in evaluating different bolt systems. There was no visible difference between the sites, but the measurements clearly showed that the performance of the short-anchor bolts was inadequate. Also, it was evident that measurements of both roof movement and bolt load were necessary to tell the complete story. Larger roof strains by themselves could have meant that the roof was just more aggressive, while reduced bolt loads alone might have signified more stable roof. But the combination clearly signaled that the bolts had slipped and had lost control of the ground.

BOLT LENGTH

Different lengths of bolts were studied at mines E and G. No meaningful comparison was possible at mine E because all of the sites were stable. At mine G, although the greatest roof strains were measured where the shorter bolts were used, the results were probably not statistically significant.

RESIN ANNULUS

The study at mine A was designed to investigate whether a "reduced annulus" (a smaller difference between the bolt diameter and the diameter of the bolt hole) would improve the performance of the primary bolting system (see Mark [2000] for a discussion of the importance of annulus to fully grouted resin bolt performance). The instrumented bolts were installed with annuluses of 7 mm (0.28 in) and 3 mm (0.12 in), but no significant differences in the bolt-loading histories were observed.

INTERSECTIONS VERSUS ENTRIES

Statistics clearly show that intersections, because of their greater spans, are significantly more prone to roof falls than entries. Measurements from the field studies provide further confirmation. At mine D, roof strains in the intersections were typically twice as great in the intersections, although the bolt loadings were approximately the same in all locations. At mine F, the bolt loads were typically 25% higher in the intersections, while the measured roof sags were similar to those in the entries.

BOLT LOCATION

The field studies found that, in general, the bolts with the highest loads were located in the center of the entry or intersection. When a cutter was present, such as at mines A, F, and G, the bolts nearest the cutter were likely to be the most heavily loaded.

CONCLUSIONS

The field studies were only partially successful in achieving their goals. Of the bolt design parameters that were evaluated, only bolt capacity seemed to clearly affect roof stability. The results concerning installed tension, bolt length, and resin annulus were all ambiguous.

On the other hand, the studies confirmed that greater roof bolt loads and more severe roof movements are likely to occur—

- In intersections;
- Near roof cutters;
- In entries perpendicular to the principal horizontal stress; and
- In areas subjected to horizontal stress concentrations.

It seems that in many cases roof stability could be improved by selectively installing stronger and/or longer bolts in these areas.

Finally, the standard instrumentation plan was shown to be an effective approach to evaluating different roof bolting systems. It provides an unequaled look into the performance of the supports and their interactions with the roof. Hopefully, mines will continue to use the instrumentation to help address their roof support issues and at the same time improve our fundamental knowledge of how roof bolts work.

REFERENCES

Campoli AA, Rebuck D, Mucho TP, Molinda GM [1996]. State-of-the-art room and pillar mining in the Kittanning Coalbed. In: Ozdemir L, Hanna K, Haramy KY, Peng S, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 269-285.

Gale WJ [1991]. Strata control utilizing rock reinforcement techniques and stress control methods in Australian coal mines. The Mining Engineer (U.K.) *Jan*:247-253.

Haas CJ [1981]. Analysis of rockbolting to prevent shear movement in fractured ground. Min Eng *June*:691-704.

Hurt K [1992]. Rockbolting design in UK coal mines. In: Proceedings of Eurock 92. London, U.K.: Thomas Telford, paper No. 71, pp. 414-419.

Karabin GJ, Debevec WL [1976]. Comparative evaluation of conventional and resin bolting systems. U.S. Department of the Interior, Mine Enforcement and Safety Administration, IR 1033.

Mark C [2000]. Design of roof bolt systems. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health.

Mark C, Mucho TP, Dolinar DR [1998]. Horizontal stress and longwall headgate ground control. Min Eng *Jan*:61-68.

Molinda GM, Mark C [1994]. The Coal Mine Roof Rating (CMRR): a practical rock mass classification for coal mines. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9387.

Mucho TP, Mark C, Zelanko JC, Compton CS [1995]. Roof support performance in high stress conditions. In: Peng SS, ed. Proceedings of the 14th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 30-38.

Serbousek MO, Signer SP [1987]. Linear load-transfer mechanics of fully grouted roof bolts. Washington, DC: U.S. Department of the Interior, Bureau of Mines, RI 9135.

Signer SP [1990]. Field verification of load transfer mechanics of fully grouted roof bolts. Washington, DC: U.S. Department of the Interior, Bureau of Mines, RI 9301.

Signer SP [2000]. Load behavior on grouted bolts in sedimentary rock. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health.

Signer SP, Mark C, Franklin G [1993]. Comparison of active versus passive bolts in a bedded mine roof. In: Peng SS, ed. Proceedings of the 12th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 16-23.

Stankus JC [1991]. Tensioned point anchor resin system vs. non-tensioned fully grouted rebar–two case studies. In: Peng SS, ed. Proceedings of the 10th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 65-75.

APPENDIX.—INDIVIDUAL FIELD STUDIES

MINE A

Mine A was located in Cambria County in central Pennsylvania [Mucho et al. 1995]. This multiple-seam longwall mine closed in 1995 largely because of a long history of ground control problems related to horizontal stress.

The goal of the study was to investigate the effect of a reduced annulus on the bolt-loading history. The annulus was reduced from 7.3 mm (0.29 in) to 3 mm (0.11 in) by installing a No. 7 bolt 20.5 mm (0.804 in) in a 26-mm (1-1/32 in) hole rather than the standard 35-mm (1-3/8-in) hole.

The instrumentation consisted of nine instrumented (strain-gauged), fully grouted, grade 75, No. 7 roof bolts installed in the normal bolting pattern (figure A-1). As shown in the figure, the bolts were alternately installed in reduced annulus and normal annulus bolt holes by position in the entry. A centrally located sonic roof extensometer and a stratascope investigation hole were also included at the site.

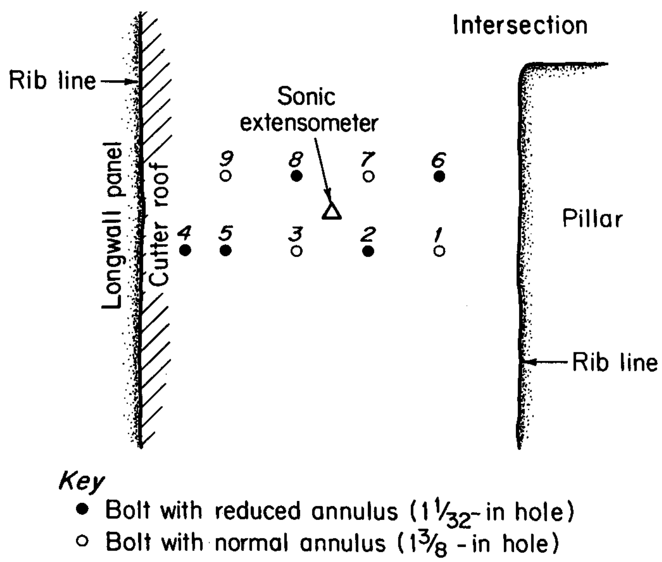


Figure A-1.—Map of study site at mine A.

Roof geology consisted of approximately 1.5 m (5 ft) of thinly laminated shale under a coarse sandstone consisting of 0.15- to 0.6-m (6-in to 2-ft) beds with thin coal streaks separating the beds. The CMRR value for the area was 48. The overburden in the study area was 900 ft (275 m).

As shown in figure A-1, a roof cutter (rock shear failure) formed along the left (panel side) rib coincident with mining and prior to the instrumentation or other supports being installed. Very little roof movement was measured initially, however. Nine days after installation there had only been a few millimeters of movement slightly above the bolts. After several weeks, however, the roof within the development section began to "work" (audible noise, dripping, etc.), and workers had to be called to the mine to set supplemental supports throughout the section, including the instrumented area, to prevent possible roof collapse. The extensometer showed that a movement of slightly over 13 mm (0.5 in) had occurred above the bolts and near the bolt top anchorage zone at the shale/sandstone interface (figure A-2).

The bolt loads during development were a function of the position of the bolt within the entry and relative to the roof cutter failure. Bolts Nos. 3 and 8, in the center of the entry on the same side as the cutter, experienced the highest peak and average loads. Loads on these bolts exceeded the yield strength of the steel (>200 kN (45,000 lb)) within 2 weeks of installation. Next heavily loaded were the side bolts next to the cutter (Nos. 5 and 9). It was assumed they were slightly lower than bolts Nos. 3 and 8 due to being installed in the failing rock of the cutter. The reduced annulus bolts experienced loads only slightly higher than the normal annulus bolts.

As the longwall approached, there was a total of approximately 31 mm (1.25 in) of total deformation. All locations experienced increases in load due to the front abutment of the longwall. Again, annulus appeared to have little effect on either the magnitude, distribution, or timing of bolt loading.

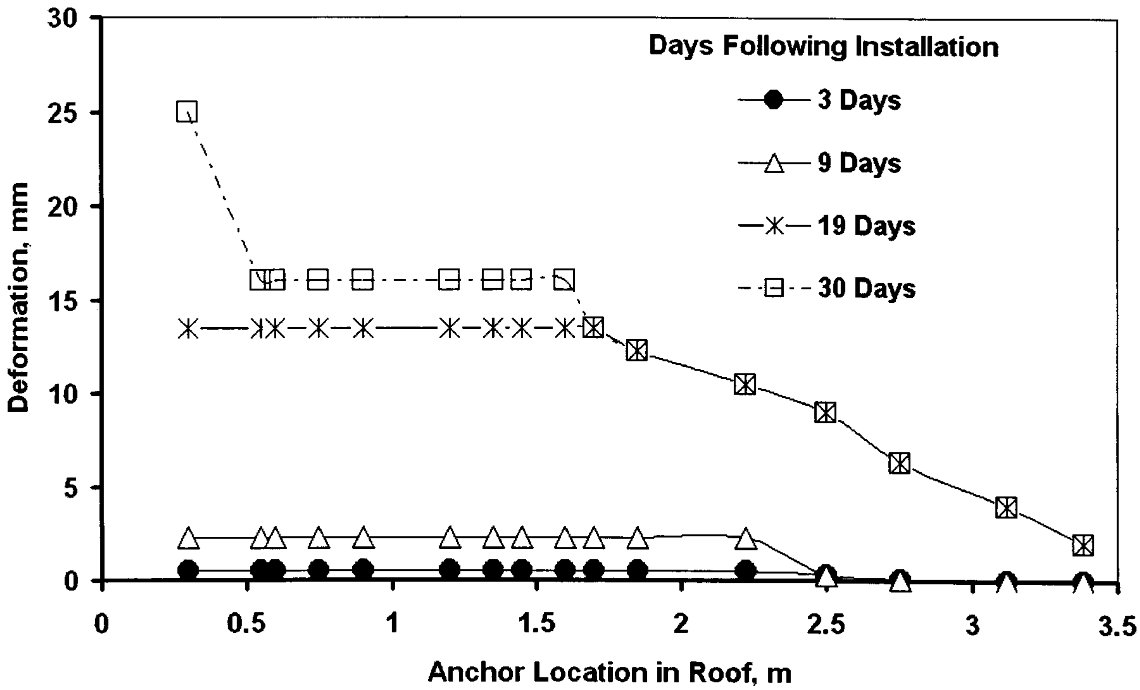


Figure A-2.—Roof deformations measured at mine A.

MINE B

Mine B was located in Greene County in southwestern Pennsylvania [Mucho et al. 1995]. This longwall mine, now also closed, operated in the Sewickley Coalbed. The mine also had a long history of ground control problems associated with horizontal stress.

The immediate roof in the instrumented area was composed of a black shale approximately 0.3 to 0.9 m (1 to 3 ft) thick, overlain by a dark gray shale or layered shale, sometimes with sandy bands or grading into a silty or sandy shale. Pronounced jointing was prevalent in the roof. Overburden in the area was approximately 180 m (600 ft). Stress mapping determined that the maximum horizontal stress was oriented approximately east-west. CMRR values in the study area ranged from the high 30s to low 40s.

To evaluate the effect of mining orientation relative to the horizontal stress field, extensometers were placed in an east-west entry (the "good" direction) and a northwest-angled crosscut (figure A-3). Due to severely broken ground, extensometers could not be installed in the north-south crosscuts (the "bad" direction) as planned.

The roof in the area of the entry extensometer showed immediate roof flaking and appeared as though it was developing a cutter; this area was rebolted in some places along the entry length. However, as can be seen from figure A-4, the strains were less than 1% and confined to the lower portions of the roof (less than the bolted interval of 2.1 m (7 ft)). Total recorded roof deformation was only approximately 6 mm (0.25 in), and the roof evidently was quite stable despite its appearance.

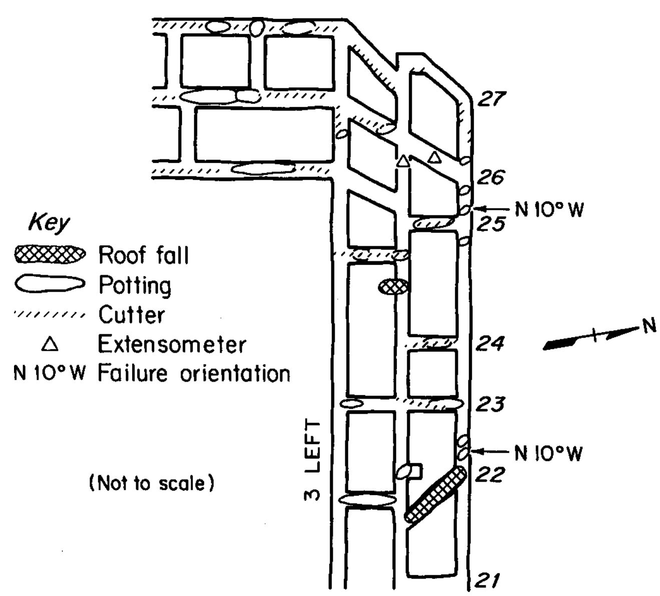


Figure A-3.—Study site at mine B.

The crosscut was far less stable, as evidenced by the strains in the roof within the bolted horizon. Strains were >1% within weeks of development at several locations. By the time the longwall passed, the strains at 2.3 m (7.5 ft), which is above the bolt anchorage zone, had increased to almost 4%. Total roof deformation (sag) at that time was approximately 25 mm (1 in). Despite the large strains, the roof was still standing after the longwall had passed.

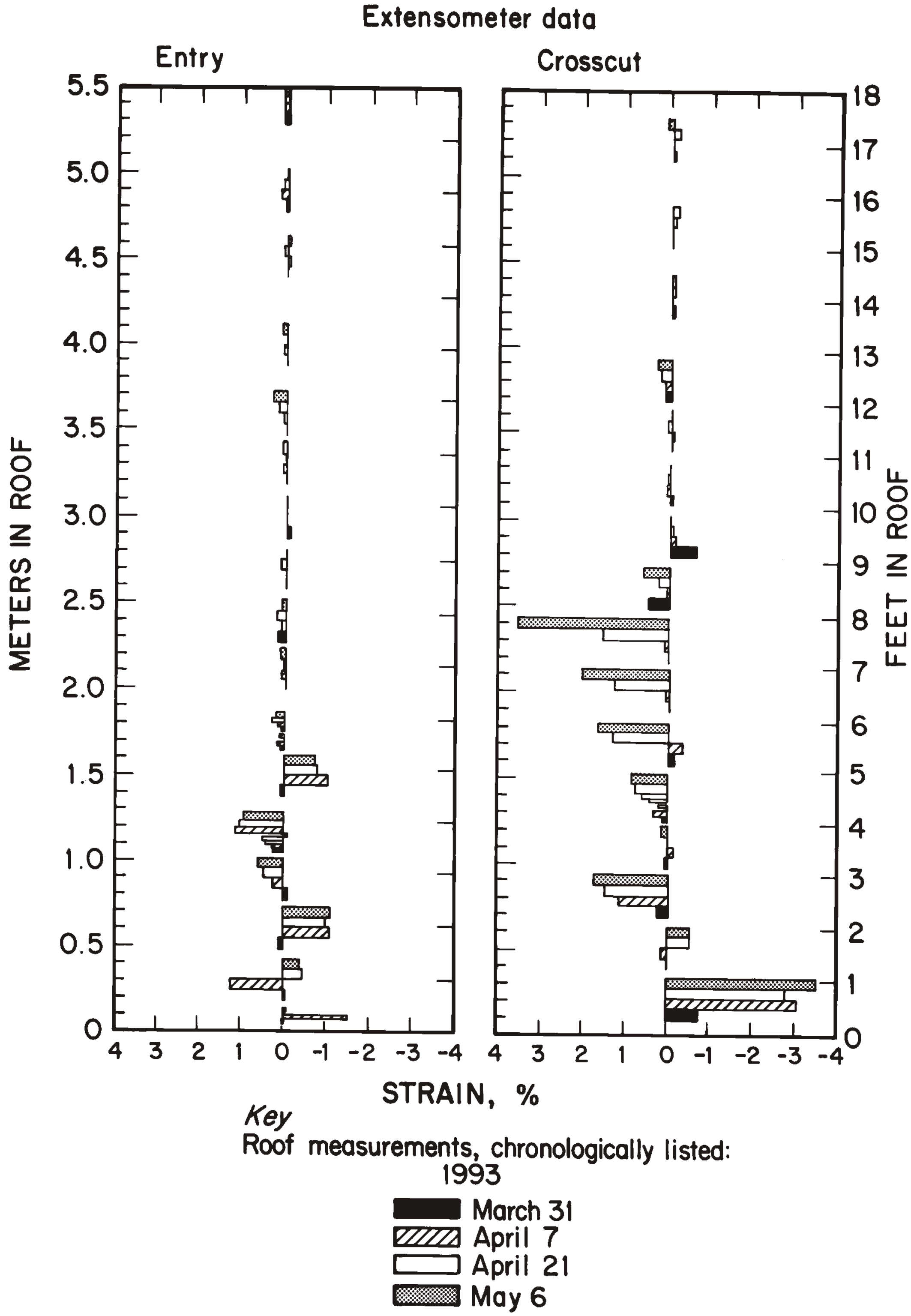


Figure A-4.—Roof deformations at mine B.

MINE C

Mine C was located in Harlan County in eastern Kentucky [Mucho et al. 1995]. This was a drift, room-and-pillar operation mining the Kellioka Coalbed under Black Mountain. Overburden in the study area was 120 m (400 ft). The immediate roof near the study area was a laminated, shaley sandstone, often with coal streaks ("stack rock"). Calculated CMRRs ranged between 54 and 44.

Multipoint roof extensometers were installed in the No. 2 entry, which was supported by 1.8-m (6-ft) tensioned rebar bolts, and in the No. 4 entry, which was supported with 1.8-m (6-ft) fully grouted resin bolts. An adjacent crosscut was also instrumented. The entries were oriented in the "bad" direction approximately perpendicular to the maximum horizontal stress.

A roof cutter developed coincident with mining in the No. 4 entry (the resin bolt area), and a similar cutter developed soon after mining in the No. 2 entry. The extensometers in the resin bolted No. 4 entry detected roof movements within 3 hr of installation. During the next 3 weeks, major roof movements (as much as 7% strain) were recorded within the bolted horizon (figure A-5). Since the cutter developed later in the tensioned rebar area, the roof movements occurred later following mining. The magnitudes of the movements in the tensioned rebar area were also less than the nontensioned fully grouted bolt area, ranging between 1% to 2% strain in the lowest 1.5 m (5 ft) of the roof. Like the fully grouted nontensioned area, no movement was observed above the bolted horizon during the study period.

MINE D

Mine D, located in Greene County in southwestern Pennsylvania, was a longwall mine in the Pittsburgh Coalbed [Mark et al. 1998; Mucho et al. 1995]. Overburden generally ranged from 180 to 300 m (600 to 1,000 ft) at the mine. The immediate roof is typical of Pittsburgh Coalbed geology, alternating relatively weak shales and coals. The CMRR for the study area was 35. The longwall panels at mine D were oriented such that the headgate where the study was conducted was subjected to a horizontal stress abutment.

Bolts from two manufacturers, designated "X" and "Y", were compared in the study. Both were 2.4-m (8-ft) long, 18-mm (0.75-in) diameter, grade 75, two-piece, resin-assisted mechanical-anchor roof bolts. The most obvious difference between the two was that the Y bolts used 0.6 m (2-ft) of resin, while the X version used only 0.3 m (1 ft) of resin with a compression ring. The instrumentation plan used for the two test sections is shown in figure A-6.

All bolts increased load shortly after installation during the development stage; intersection bolts increased the most and center bolts achieved the highest loads (they also were the highest initial set loads).

The maximum load achieved by the short-anchor X bolts averaged 84.5 kN (19,000 lb), but in many cases the load was dropping as the longwall approached. Several Y bolts achieved the yield limit of the steel of 150 kN (33,000 lb), and most continued to increase their load up until the final reading as the longwall face passed (figure A-7). Since the maximum load achieved by the short-anchor bolts was well below the strength of the steel, it appears that the anchors must have been slipping. The most likely explanation is that 0.3 m (1 ft) of resin was insufficient to maintain anchorage in this particular roof rock.

Roof strains measured during the approach of the longwall are shown in figure A-8. At the X bolt stations, roof strains >2% were measured at four locations within the bolted horizon. At one intersection location, a roof strain of 6 was measured *above the bolts*. The X bolts apparently began to lose control of the ground as the horizontal stress concentration developed.

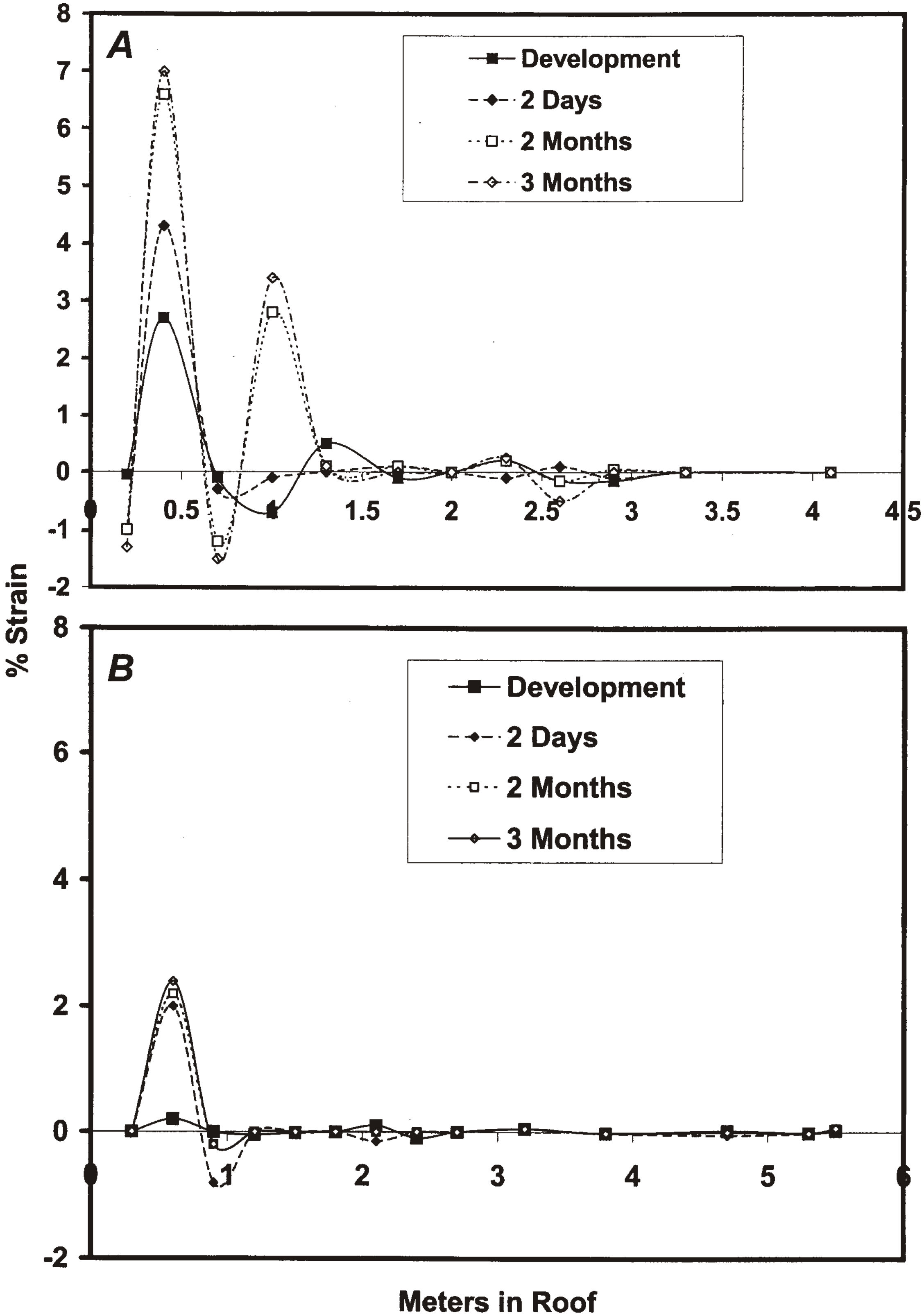


Figure A-5.—Roof strains measured at mine C. *A*, Resin bolt site; *B*, torque-tension bolt site.

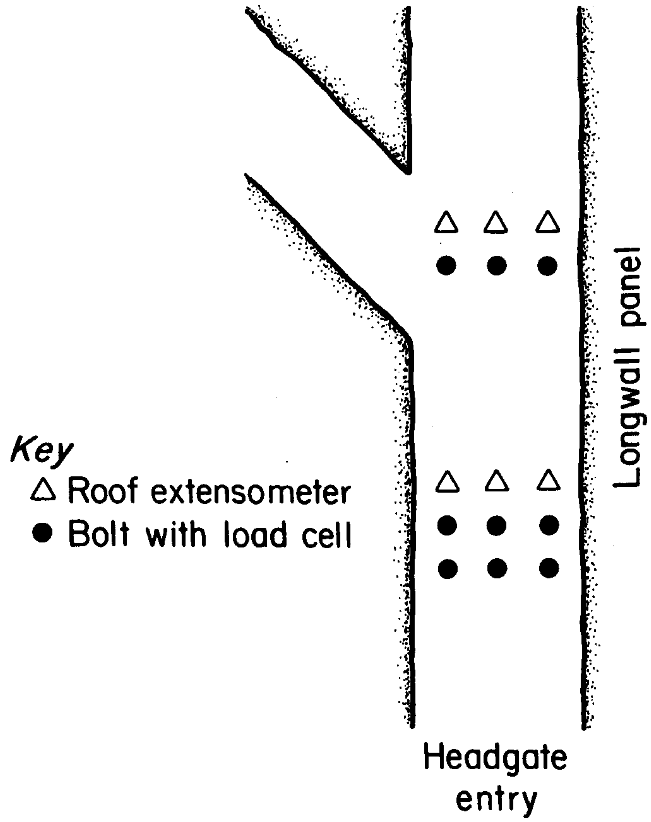


Figure A-6.—Instrumentation plan at mine D.

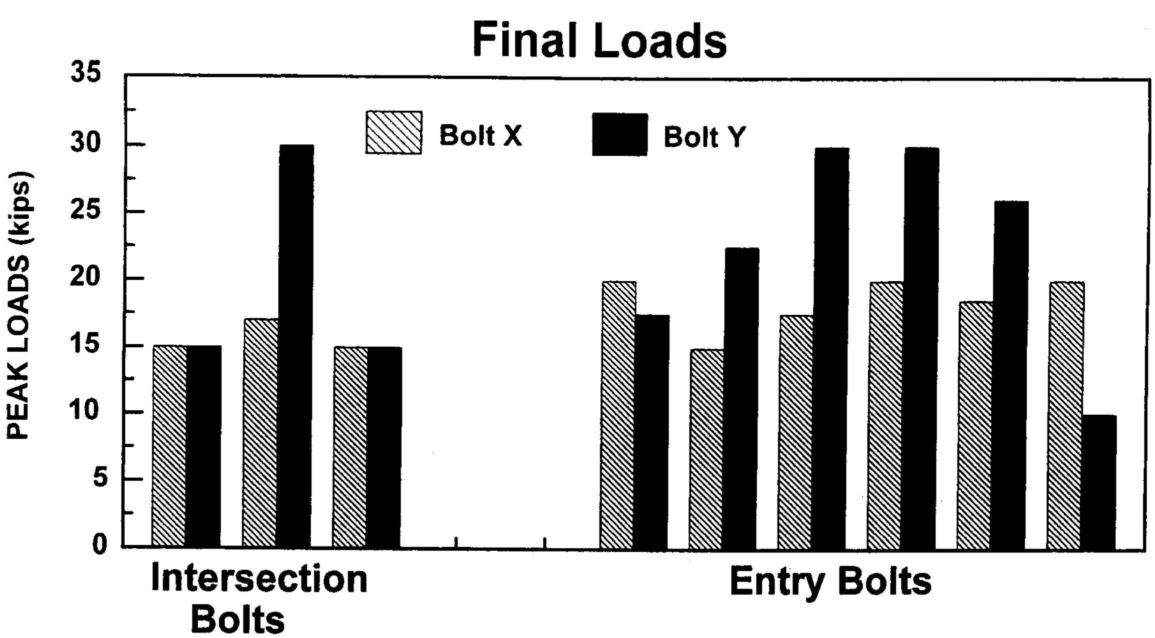


Figure A-7.—Roof bolt loads measured at mine D. Bolts are paired by location (i.e., the first pair shows the intersection right-hand X bolt compared to the intersection right-hand Y bolt.

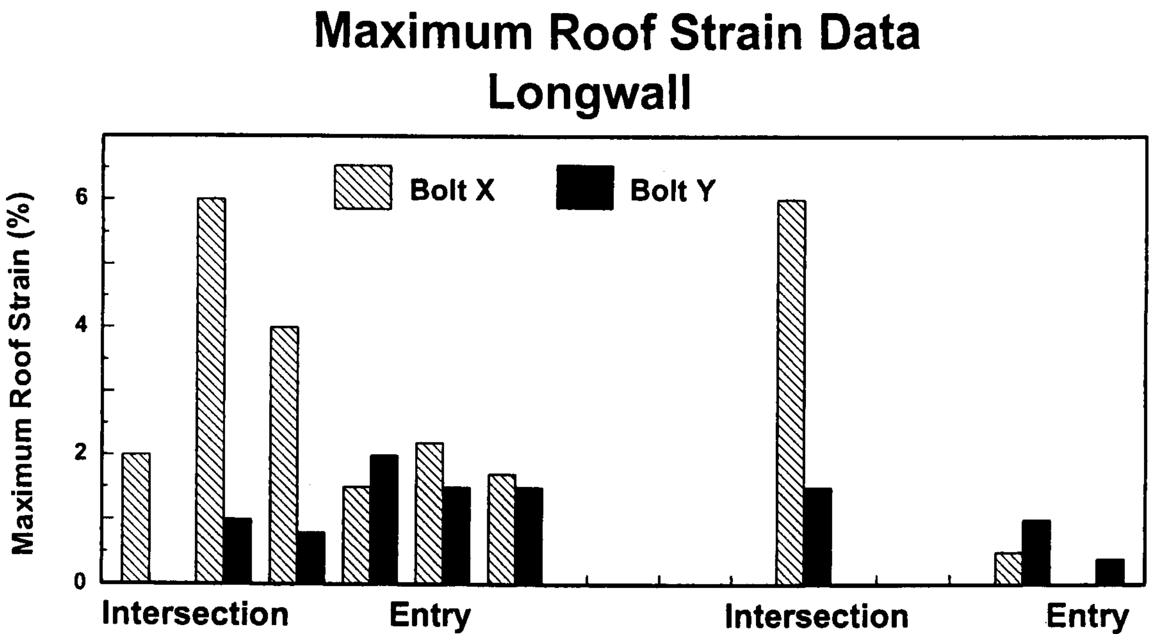


Figure A-8.—Roof strains measured at mine D. The right-hand data show strains measured within the bolted horizon; the left-hand data show strains measured above the tops of the bolts.

MINE E

Mine E was a sister operation to mine D and was similar in most respects [Mark et al. 1998]. One significant difference was that the horizontal stresses are relieved in the headgate by the longwall's stress shadow. Four bolting systems were compared in consecutive intersections at mine E:

- Fully grouted resin bolts, 1.5 m (5 ft) long, 1.4-m (4.5-ft) row spacing;
- Fully grouted resin bolts, 1.5 m (5 ft long), 1-m (3-ft) row spacing;
- Resin-assisted mechanical-anchor bolts, 1.5 m (5 ft) long, 1.4-m (4.5-ft) row spacing; and
- Resin-assisted mechanical-anchor bolts, 2.4 m (8 ft) long, 1.4-m (4.5-ft) row spacing.

The fourth bolting system was essentially identical to that employed at mine E.

Very little change in roof deformation and almost no change in bolt load was observed at any of the four sites as the longwall approached. The maximum increase in roof strain averaged a mere 0.2%, and all of this occurred below the bolt horizon. Final loads on the tensioned bolts ranged between 75 and 135 kN (8 and 15 tons), considerably less than their 150-kN (16.5-ton) yield strength. As the longwall approached, some bolts even decreased load slightly (figure A-9). It appears that relief of the horizontal stress may actually have *enhanced* roof stability!

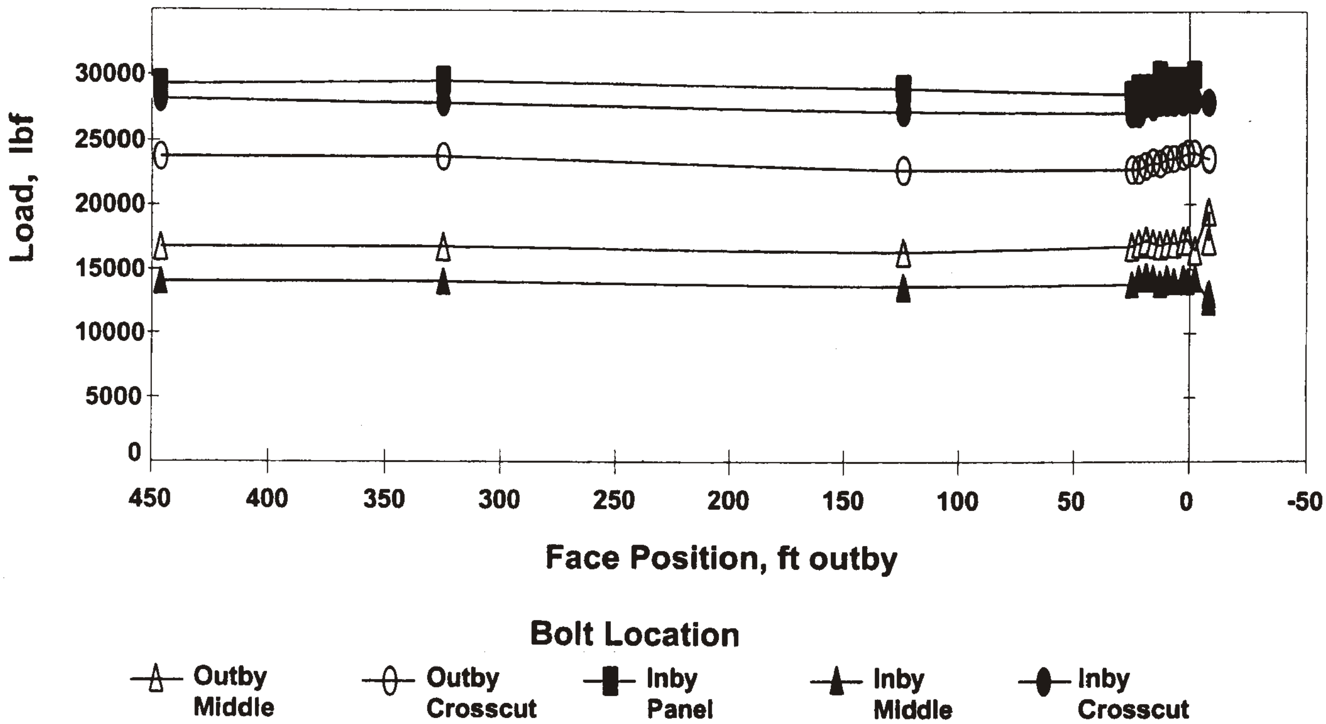


Figure A-9.—Bolt loads at mine E during the approach of the longwall.

MINE F

Mine F is a longwall mine located in Alabama's Black Warrior Basin at a depth of approximately 670 m (2,200 ft) [Signer et al. 1993]. Normally, the immediate roof is the Middleman, a fossiliferous shale that grades into thinly interbedded shale and coals. When the Middleman is the immediate roof, mining is said to be "single-seam." The Middleman is overlain by the 30-cm (1-ft) thick Mary Lee Seam. The main roof above the Mary Lee consists of 30 to 61 cm (1 to 2 ft) of competent siltstone overlain by massive sandstone. Horizontal stress caused roof guttering next to the future longwall panel in most of the test areas. A stress concentration was also carried with the tailgate corner during longwall mining.

The standard primary support in the tailgate was 19-mm (0.75-in) diameter, 1.8-m (6-ft) long, grade 40, nontensioned fully grouted bolts. The study compared these with 22-mm (7/8-in) diameter, 1.8-m (6-ft) long, grade 60, resin-assisted mechanically anchored bolts. Just prior to the headgate pass, an additional row of fully grouted resin bolts was installed between each row of primary supports.

Four study sites were chosen, two in areas supported by fully grouted bolts and two in areas supported by tensioned resin-assisted mechanical-anchor bolts. Two sites were located in intersections and two at midpillar. At each site, four instrumented roof bolts and three 3-point roof extensometers were installed (figure A-10).

The data shortly after development show that there was high localized loading in the fully grouted bolts and that several bolt locations had reached the yield point of the steel. Generally, the maximum load was measured by those gauges near the interface between the Mary Lee Seam and the main roof. At the midpillar site, greater bolt loads tended to develop on the bolts nearest the panel rib where the cutter formed. Bolt loads in the intersections were higher than those at the midpillar. The degree of loading on the tensioned bolts was similar to that on the fully grouted bolts.

Between development and the headgate pass the fully grouted bolts loaded up more rapidly than the point-anchor bolts. Several sections of the resin bolts passed into the strain-hardening phase of the plasticity curve (figure A-11). In the final days before the tailgate passed, loads increased significantly on nearly every bolt. Fifty percent of the strain-gauge stations on the resin bolts showed loads in excess of the yield point of the steel. The load increase on the tensioned bolts during the tailgate pass was very similar to that on the fully grouted bolts. However, the capacity of the point-anchor bolts was higher, and the loads remained below the yield point of the steel.

The roof deformation measurements showed that the greatest movements occurred along the panel rib, where the horizontal stress guttering was present. Greater roof movements also developed at the fully grouted bolt sites during the tailgate pass, and roof conditions were clearly more hazardous in the fully grouted bolt areas.

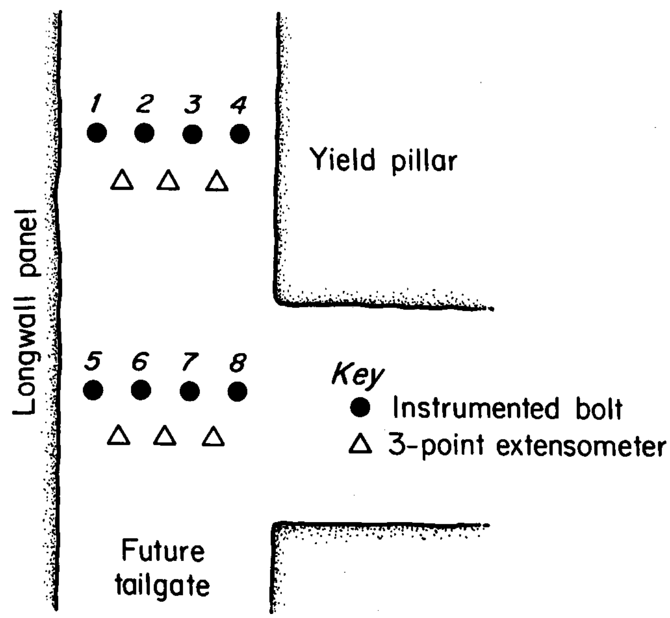


Figure A-10.—Study site at mine F.

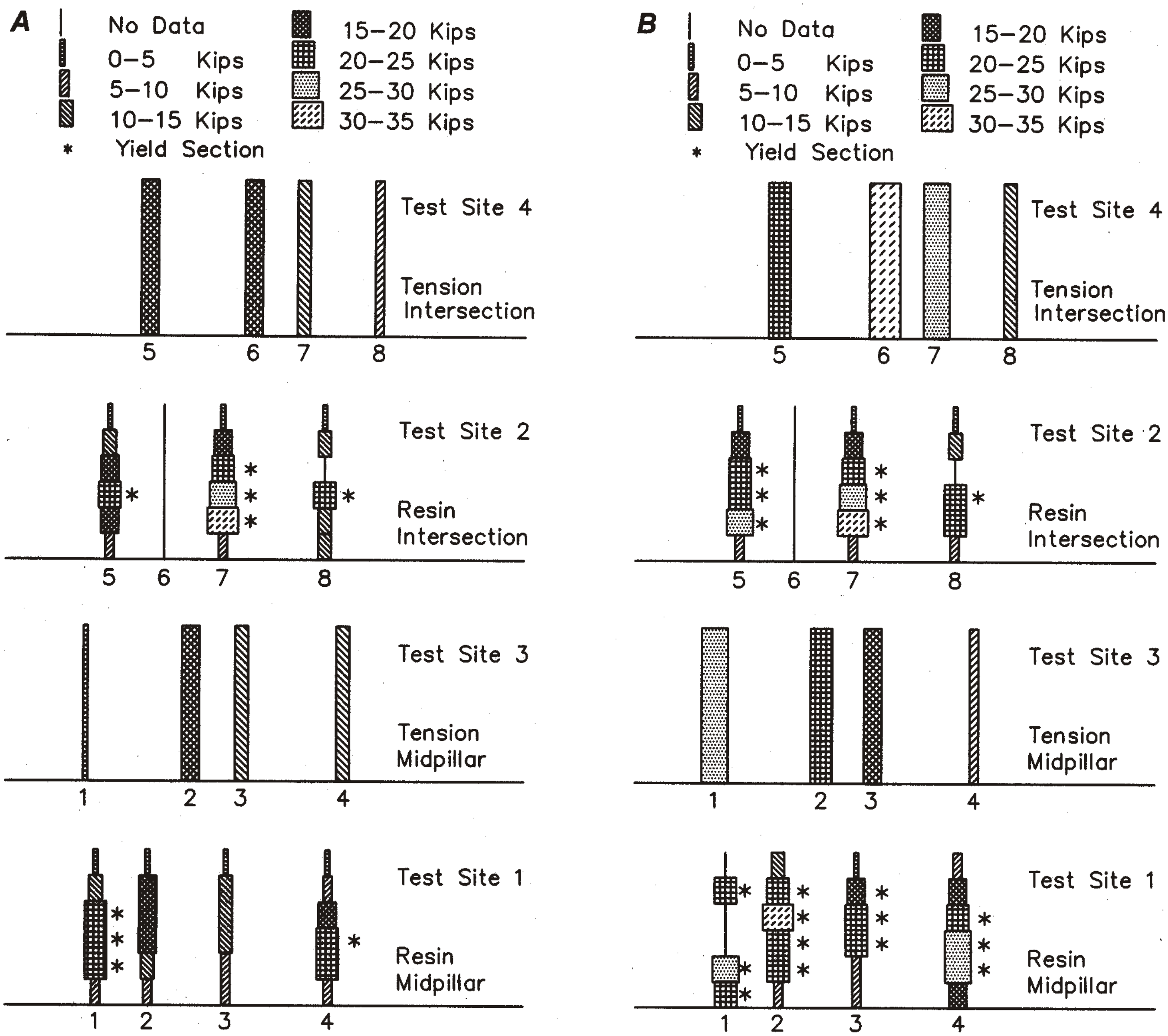


Figure A-11.—Bolt loads measured at mine F. The left-hand bolts are nearest the logwall panel rib. *A*, After the headgate pass; *B*, at the tailgate corner.

MINE G

Mine G employed continuous miner technology, including mobile roof support and continuous haulage, to retreat mine the Lower Kittanning Coalbed in central Pennsylvania [Campoli et al. 1996]. The depth of cover varied from 45 to 120 m (150 to 400 ft). The immediate roof was a sandy shale with a CMRR of about 50. Extensive stress mapping found local damage related to small geologic features (slickensides and fossiliferous bedding planes), but no significant correlation between direction of drivage and roof damage.

Borehole extensometers were installed in the roof of seven intersections, as shown in figure A-12. Two of the intersections (P2 and P4) were supported by 1.52-m (5-ft) resin-assisted point-anchor tension bolts. Three others (R2-A, R2-B, and R4) were supported by 1.83-m (6-ft) fully grouted resin bolts. The final two (R-P2 and R-P4) were supported by fully grouted bolts supplemented by two additional resin-assisted point-anchor bolts between each row. Hydraulic load cells were installed on four of the tensioned bolts at the P2 intersection.

The roof was monitored during both the developmental and retreat mining. Roof strains approaching 1% above the bolts were measured on development in one of the point-anchor sites (P-2 in figure A-13), but R-P4 was not far behind. These two intersections continued to see the greatest deformations as the pillar line approached, probably because of nearby "cutterlike" roof damage. Some appreciable roof deformations in the "skin" near the roof line were observed at a number of sites as the pillaring operations approached.

The bolt loading increased systematically from the No. 1 bolt to the No. 4 bolt. The No. 4 bolt, which was farthest from the coal pillar and near a "cutterlike" feature, saw a load of 100 kN (23,000 lb), which is near its yield point (figure A-14). The bolt loadings did not change significantly over time even when the pillars were recovered.

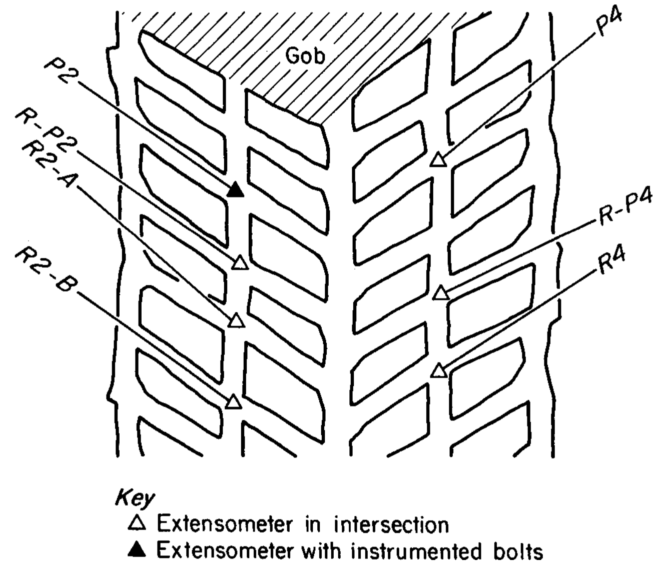


Figure A-12.—Study site at mine G.

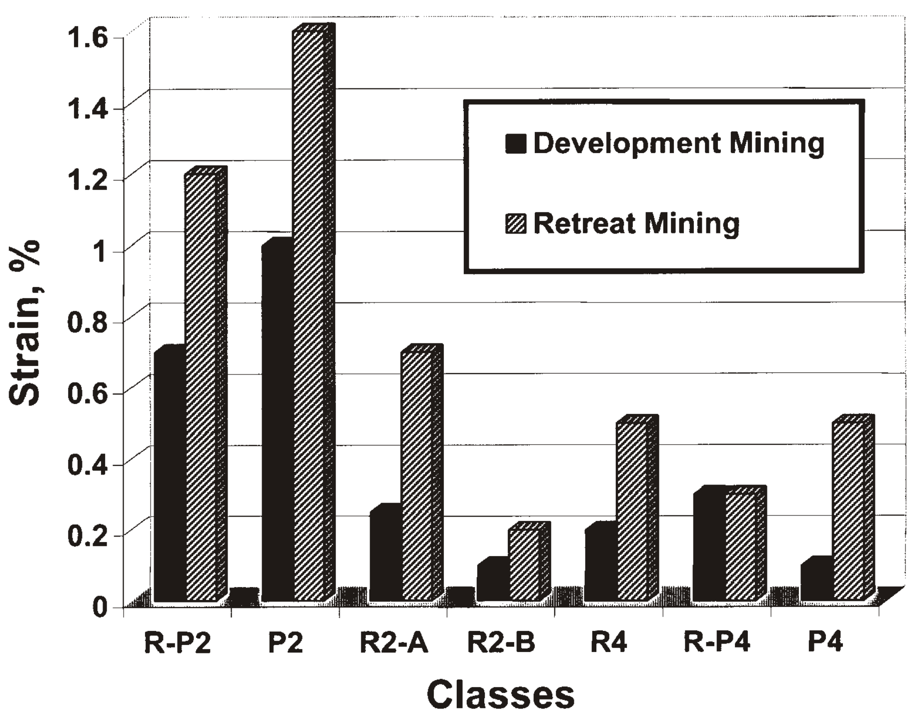


Figure A-13.–Maximum roof strains measured above the bolted horizon at mine G.

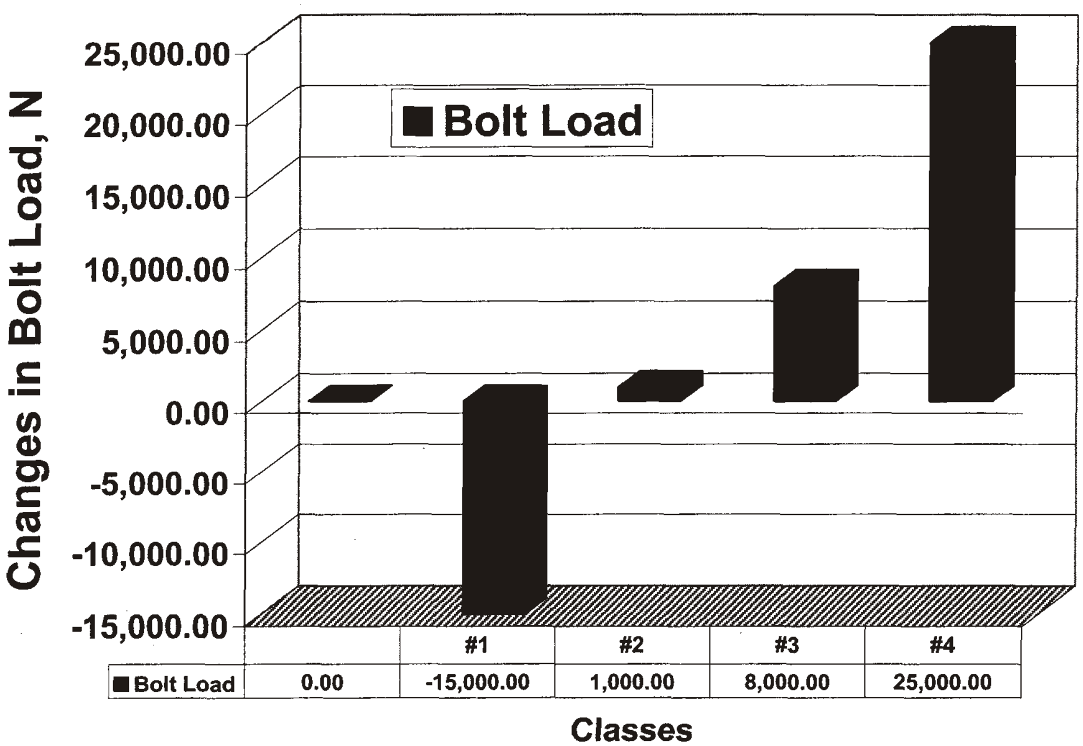


Figure A-14.—Changes in roof bolt loads measured at mine G. Bolt No. 4 is nearest a cutter; bolt No. 1 is nearest a solid rib.

SKIN FAILURE OF ROOF AND RIB AND SUPPORT TECHNIQUES IN UNDERGROUND COAL MINES

By Eric R. Bauer[1] and Dennis R. Dolinar[1]

ABSTRACT

Skin failures of roof and rib in underground coal mines continue to be a significant safety hazard for mine workers. Skin failures do not usually involve failure of the support systems, but result from rock or coal spalling from between the support elements. For instance, in 1997 more than 800 miners were injured by roof and rib falls, of which 98% were the result of skin failures [Bauer et al. 1999]. Also, nearly 80% of the roof and rib failure injuries occurred at or near the working faces in development sections. The face area is a zone where the potential for skin failure accidents and injuries and for roof and rib failures is high because of mining activity, ground readjustment due to changing stress conditions, and the higher exposure of mine workers. In addition, failures occur where the roof and rib are unsupported. This paper reviews the roof and rib accident statistics resulting from skin failure, and highlights the incidences by type, numbers and percentage, in-mine location, supported and unsupported roof, and worker activity at the time of injury. Also discussed are the causes of roof and rib skin failures, current and improved support methods and materials for skin surface control, and machine design modifications for improved roof bolter operator protection. It also reviews the historical literature on skin failures and control methods.

[1]Mining engineer, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

Falls of roof and rib traditionally have been one of the leading causes of mine worker injuries and fatalities in underground coal mines. From 1993 to 1998, nearly 35% of all reported underground incidents resulted from falls of roof and rib. These falls of roof and rib resulted in more than 4,600 injuries, or 12% of the total reported underground injuries. Also, skin failures, which are the failure of small blocks or slabs of roof and rib, have been recognized as a problem in the coal mining industry for many years. Detailed analyses showed that in 1997 alone, approximately 98% of the roof and rib injuries were from skin failures. This suggests that as many as 4,500 injuries may have resulted from skin failures of the roof and rib during this 5-year period.

Reference to skin failures is found in the literature as far back as the late 1920s. Most of the early references discussed the effect of moisture and humidity on roof failures [Paul 1928; Hartman and Greenwald 1941]. Other authors addressed ways to condition mine air, such as water sprays and tempering entries, to prevent roof deterioration [Fletcher and Cassidy 1931; Herbert 1940]. Considerable work was presented on the effectiveness of various sealants to coat mine strata, including coal tar [Brown 1941], Ebonol [Robbins 1937], asphalt-based paints [Shacikaski 1951], sulfur-based coating materials [Dale and Ludwig 1972], cement and cement mixtures [Artler 1974], shotcrete [Cecil 1968], and polymeric sealants [Franklin et al. 1977]. More recently, researchers have investigated the mechanisms of shale roof rock deterioration due to atmospheric moisture, which seems to be a result of stresses from moisture-induced weakening and swelling strain, rather than slaking [Cummings et al. 1983; Pappas and Vallejo 1997]. Finally, although much attention has been given to the effects of moisture and humidity on the mine roof and the resultant roof slaking, moisture-induced skin failure is probably not the most prevalent cause of roof skin injuries. This moisture-induced slaking is primarily a nuisance from the standpoint of cleanup and perception. Skin failure of the roof due to geology and stress, in combination with mining, creates a more substantial hazard to the miners at the face and not the long-term deterioration of the roof due to moisture. Supporting evidence is that nearly 80% of all roof skin injuries occur inby the feeder breaker in development sections. To date, the problem of skin failure at or near the working face has not been adequately addressed. This type of skin failure will have to be addressed by surface control systems other than sealants and by the use of alternative methods, such as removal of a lower roof member during mining.

Although the above literature dealt mostly with roof skin failure, rib skin failure has also received attention by the coal mining industry. The theory and practices regarding rib failure, especially in thick coal seams, were addressed by Smith [1989], who suggested that fracturing begins at a stress level equivalent to one-third to two-thirds of the ultimate strength of the material. Peng [1986], Dolinar and Tadolini [1991], and Dolinar [1993] discussed general coal rib stabilization and the effectiveness of wood dowels, resin bolts, and straps to provide pillar reinforcement. Martin et al. [1988] provided information that demonstrated the superior performance of yieldable rib bolts to stabilize ribs when twin-seam mining at Jim Walters Resources. Wykoff [1950] and Horino et al. [1971] investigated the use of wire rope to wrap pillars. Their research indicated that wire rope can significantly affect the compressive strength and stability of pillars. In addition, many of the references on mine sealants mentioned the use of these for coating and sealing coal ribs.

Many advances have been made in dealing with roof and rib failures. Unfortunately, the problems have not been eliminated. Continued research by government, academia, labor, and the mining industry is needed to address roof and rib skin failures and minimize the associated injuries to underground mine workers. Research at the National Institute for Occupational Safety and Health (NIOSH) is continuing this effort by investigating the causes of skin failure and evaluating control techniques.

DESCRIPTION OF SKIN FAILURE

For the purposes of this paper and analyses, skin failure does not involve the failure of the primary support, but the spalling of rock from between roof bolts and from around the automated temporary roof support (ATRS) system and canopies of roof bolting machines. Rib skin failure includes the spalling of coal from unsupported ribs. Skin failure involves smaller pieces of rock or coal, rather than massive roof failures (above anchorage) or coal pillar failures (bumps and bursts). Skin failures can occur in both supported and unsupported mine strata. Figure 1 shows skin failure of unsupported mine roof; figure 2 is an example of skin failure of supported (bolted) mine roof where the failure occurs between the supports. In general, skin failure of the roof inby permanent support must be controlled by the ATRS or canopies of the roof bolting machine. The skin failure under permanent support can usually be handled by removal or by surface control systems. Rib skin failures are shown in figure 3 (unsupported rib) and figure 4 (supported rib).

The mechanisms responsible for skin failures vary considerably. The most common factors are competence of the

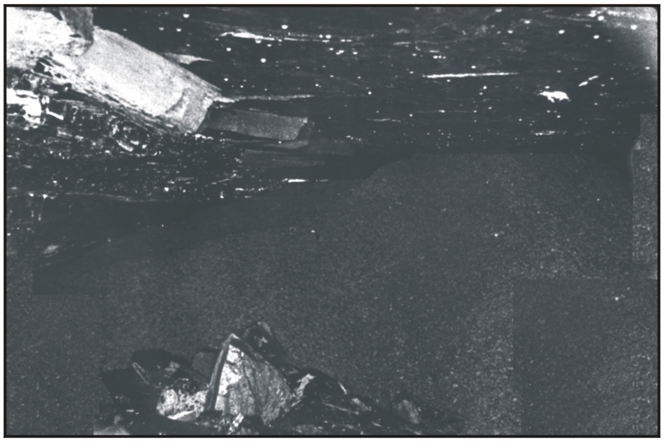


Figure 1.—Skin failure of unsupported roof.

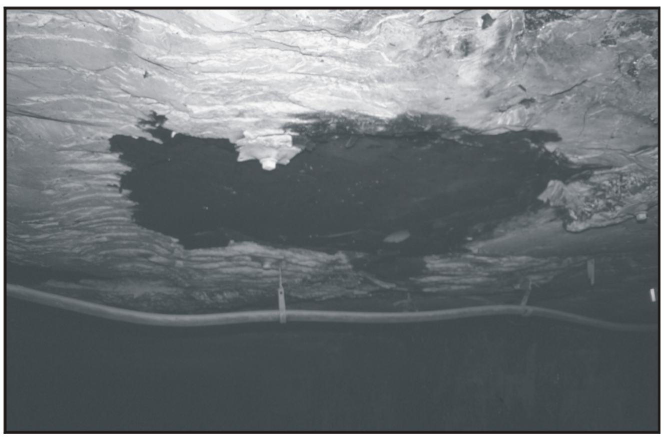


Figure 2.—Skin failure in supported roof.

strata and presence of geologic discontinuities. In many mines, the roof is composed of draw rock (soft slate, shale, or rock), coal, bony material, and other highly stratified, thinly laminated strata. These strata are susceptible to failing in thin layers because of bedding plane weaknesses. Some of the causes of bedding plane failures that result in skin failures include sag of the strata from gravity, overburden pressure as depth increases, horizontal stress, and moisture or temperature sensitivity. Mining-induced stresses and damage are also important in the development of skin failure in both the roof and rib. If geologic discontinuities are present, the likelihood of skin failure increases because the discontinuities weaken and compromise the structural integrity of the rock. It seems logical that the potential for the roof to experience skin failure can be estimated using the Coal Mine Roof Rating (CMRR). The CMRR estimates the structural competence of coal mine roof and considers bedding most important. It includes the factors that

Figure 3.—Skin failure of unsupported rib.

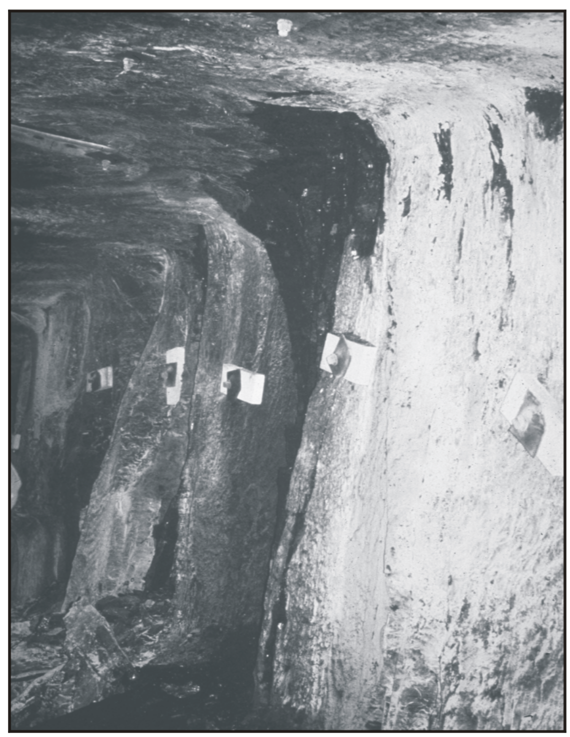


Figure 4.—Skin failure of supported rib.

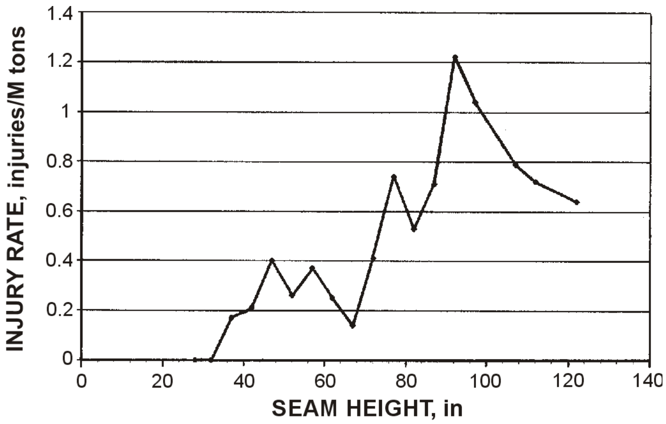


Figure 5.—Seam height versus injury rate for rib skin failures, 1997 (after Bauer et al. [1999]).

weaken bedded coal-measure rocks, such as discontinuities, moisture, and rock strength [Molinda and Mark 1994]. The lower the CMRR, the less competent the roof and the more susceptible it is to skin failure.

Coal ribs can experience skin failure for many of the same reasons. Primarily, rib skin failure is associated with effects of depth or, at times, with the early stages of the failure of insufficiently sized pillars. A general observation about rib skin failures is that rib spalling tends to increase as mining height increases. A plot of rib skin failure injuries and seam height for 1997 indicates that the injury rate increased as the seam height increased, up to 8 ft thick (figure 5). For seam heights >8 ft, a decreasing trend occurs, probably because more rib support is used in the thicker seams. Rib spalling may also increase with depth and is affected by mining-induced stresses. Rib skin failure is also frequently associated with rock partings within the coal pillar or with draw rock located at the roof-rib interface. Rock partings or bands within the pillar create planes of weakness where differential movement (figure 6) and failure can occur, leading to spalling (skin failure). When weak draw rock that is subject to failure during coal extraction is present and is mined with the coal, the draw rock exposed in the coal pillars creates a zone of potential rib skin failure. This inherent weakness makes the draw rock susceptible to spalling from the rib as the coal pillars experience load.

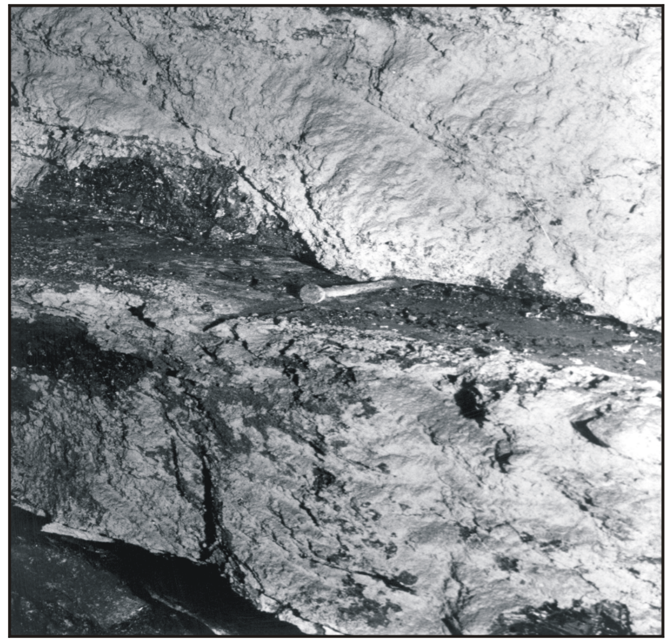


Figure 6.—Example of differential movement along a parting in a coal rib.

SKIN FAILURE INCIDENT ANALYSIS

Two separate incident analyses were conducted. One addressed roof and rib fall fatalities during 1996-98; the other addressed all reported roof and rib fall incidents during 1995-97. These analyses were designed to identify the fatalities and injuries resulting from skin failures, both roof and rib, and massive failures, then draw some statistically based conclusions.

ROOF AND RIB FALL FATALITIES

The underground coal mine fatalities caused by falls of roof and rib for the period 1996-98 were separated by skin and massive failures. Skin failure of the roof and rib has been previously defined. A massive roof fall involves the failure of the primary support system and usually has an areal extent in at least one dimension approaching the width of the opening. For the fatalities, the average thickness of the massive failures was 8.55 ft. For rib failures, nearly all were classified as skin failures, except those listed in Mine Safety and Health Administration (MSHA) fatality reports as an outburst or bump. Table 1 summarizes the classification by failure types. Essentially, 50% of the fatal injuries that occurred under supported roof were caused by skin failure of the roof or rib. Rib failures resulted in over twice as many fatalities as roof skin failures and were caused by the lack of rib support, which allows large slabs to spall from the ribs. Only three fatalities occurred from roof skin failure; however, these occurred under the supposedly safe conditions of supported roof. During this 3-year period, 11 fatalities occurred under unsupported roof. This is a human behavior issue rather than a ground control problem; thus, these fatalities are not included in this analysis.

Table 1.—Roof and rib fatalities by failure type, 1996-98

Year	Rib skin fatalities	Supported roof skin	Massive failures
1996	3	1	1
1997	3	0	6
1998	1	2	3
Total	7	3	10

REPORTED ROOF AND RIB FALL INJURIES

To delineate the extent of worker injuries resulting from skin failures, the MSHA accident database was examined for the period 1995-98. All injuries occurring in underground coal mines that resulted from roof and rib failures were extracted and analyzed. This included degree-of-injury classes from 1 to 6, which were injuries ranging from no lost time or restricted activity to those that resulted in a fatality. They did not include reportable roof falls that occurred when no workers were present. In addition, the accident injury illness types extracted were fall of face, rib (or side), and fall of roof. Some of the roof and rib fall injuries are classified under machinery incidents. These misclassified incidents were sorted out by using the source-of-injury code with a criterion of caving of rock, coal, ore, and waste. Table 2 summarizes the roof and rib injuries for 1995-98. The table reveals that most of the injuries resulted from roof skin failures (82%), followed by rib skin failures (16%), and massive failures (2%).

Table 2.—Number of injuries from roof and rib failures, 1995-98

Failure type	No. of injuries	Percent of injuries
Roof skin	2,716	82
Rib skin	524	16
Massive	58	2
Total	3,298	100

Next, another analysis determined the mining situations in which roof and rib skin injuries occurred (for 1997 data only). Table 3 indicates that 84% of the skin failure injuries occurred during development or retreat mining, with the remaining 16% divided among longwall and other. An attempt was made to determine the location of skin failure injuries with respect to the state of roof support. The best estimate is that 383 of the 669 roof skin injuries (57%) occurred under permanent support. It is possible that many of the roof skin failure injuries occurring where the roof was permanently supported could have been prevented through modified support designs. Another 233 (35%) roof skin injuries occurred under temporarily supported or unsupported roof. Increasing the skin coverage of the ATRS or coverage area of the drill station canopies could help reduce the roof skin failure injuries occurring under temporarily supported roof. For the remaining 53 roof skin injuries, the state of support was uncertain, but was provided by either the ATRS or permanent support. Approximately 85 of the 128 (66%) rib skin injuries occurred where the roof was permanently supported. The rib skin failure injuries occurring under permanently supported roof may be minimized by securing the ribs, if necessary, or through scaling and increased awareness of rib conditions. Another 19 (15%) rib skin injuries occurred under temporarily supported or unsupported roof. The remaining 24 (19%) occurred where the state of the support was unknown.

Table 3.—Roof and rib skin failure injuries classified by mining situation, 1997

Mining situation	Roof skin failures		Rib skin failures	
	Injuries	%	Injuries	%
Development[1]	560	84	108	84
Longwall[2]	38	6	11	9
Other[3]	71	10	9	7
Total	669	100	128	100

[1]Includes advance and retreat mining.
[2]Includes injuries in the headgate and tailgate during panel mining.
[3]Includes injuries outby face and of unknown origin.

Table 4 shows the distribution of skin injuries by location and support type. Temporary support is provided by the ATRS and canopy of the roof bolter, while permanent support is provided by the primary support system. About 78% of all roof

Table 4.—Location of roof and rib skin failures, 1997

Type and location	Injuries	Face[1]	Working section[2]	Face area total[3]	Face area, %	Other/ unknown[4]	Other, %
Roof:							
Permanent support	383	150	111	261	68	122	32
Temporary support	233	215	4	219	94	17	4
Rib:							
Permanent support	85	38	25	63	74	22	26
Temporary support	19	16	1	17	90	2	10

[1]Injuries occurring at the active face or inby the last open crosscut.
[2]All other injuries occurring inby the feeder on working sections.
[3]Total injuries occurring in the face and working section.
[4]Injuries occurring inby the working section, or location unknown.

skin injuries occurred inby the feeder, while 58% of the injuries occurred at the active face. This is a strong indicator that roof slaking due to moisture was not the primary concern in causing these types of injuries. Again, with the coal ribs, nearly 77% of the injuries were inby the feeder.

Finally, the mine worker activities during roof and rib skin injuries were extracted from the MSHA database for 1995-98. The most common activities of workers injured by roof skin failures were drilling or bolting of the roof (39%), operating the continuous mining machine (11%), and general inside labor (9%) (figure 7). These three activities accounted for 59% of the injuries. No other worker activity was involved in more than 7% of the injuries. For injuries resulting from rib skin failures, the most common worker activities were operating the continuous mining machine (18%), drilling or bolting the roof (16%), general inside labor (12%), walking (9%), and maintenance and repair (8%). The total of these accounted for 63% of the injuries (figure 8). All other activities were involved in 5% or less of the rib skin injuries. Surprisingly, scaling of the roof or rib, which deals directly with skin failure and is thought to be a dangerous activity, comprised only 1% of the roof and rib skin failure injuries. This low level of scaling injuries compared with the high number of skin failure injuries may indicate that not enough scaling is done. In addition, cable handling was involved in 3% of the total roof and rib skin failures (figure 9).

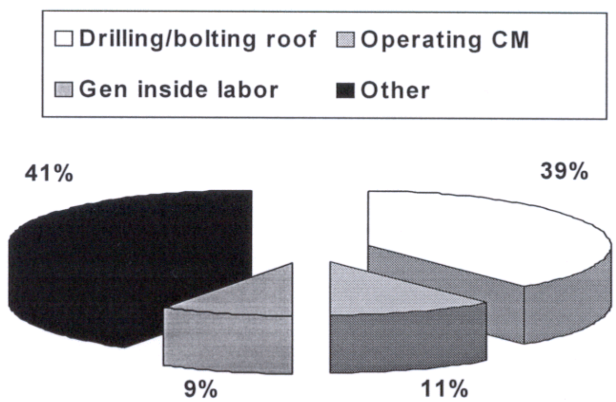


Figure 7.—Mine worker activities during roof skin injuries, 1995-98.

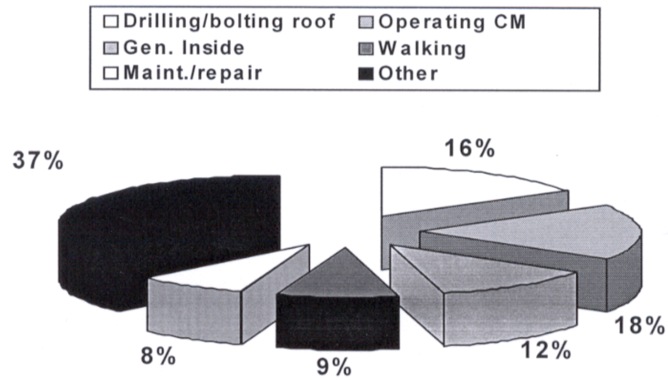


Figure 8.—Mine worker activities during rib skin injuries, 1995-98.

Figure 9.—Mine worker moving mining machine power cables.

SKIN CONTROL METHODS

This review of skin control methods both examines what has been done in the past and describes current control techniques. Our investigation reveals that many of the same methods used in the past are used today.

EARLY SKIN CONTROL EFFORTS

Past control methods were directed primarily at preventing skin failures resulting from changes in temperature and humidity. These included various coating materials designed simply to seal the surface without providing additional strength or reinforcement and attempts to condition the mine air before it was introduced into the mine workings. The air conditioning involved regulating the temperature and humidity to near ambient mine conditions to prevent failures due to expansion and contraction and to prevent moisture variations. In the face area, past attempts at controlling roof and rib skin failure using artificial means reflected the support materials available. Mechanical bolts in combination with wood headers and planks, oversized plates, wire mesh, old hoist rope, wood dowels, and other simple support methods were commonly used.

CURRENT SKIN CONTROL METHODS

Current control methods have built on the successes of past techniques, using the more sophisticated support materials now available. In addition, more thought has been given to matching the type of control to the failure mode. For instance, because the mining industry has an improved understanding of the mechanism of strata failure, cement coatings using steel or glass fibers are available not only to seal the strata, but also to add strength to resist failure. For control of roof skin failure, wood planks, steel straps and channel, and various meshes such as welded wire, chain link, or synthetic grid material are being used.

Rib support methods have changed as well, primarily in the use, type, and location of bolts. The emphasis is to match the deformability of the rib supports to that of the rib. Yieldable bolts, such as those used at Jim Walters Resources [Martin et al. 1988], can stabilize the coal seam and ribs effectively by controlling displacements to reduce stress buildup.

A recent information request from MSHA District 3 revealed the following examples of roof skin control methods: (1) one mine uses screens in one intake, one return, and the track entry, and uses a lot of gunite; (2) another mine uses 8-gauge steel, 5- by 16-ft panels of "welded wire" installed on cycle; (3) one longwall mine is required to use screening or gunite where it has trouble holding up head coal in its gate roads; and (4) one mine that has a history of falls due to deteriorating top has miles of gunited track entry. Information obtained from MSHA District 4 revealed additional skin control methods. These included using oversized bearing plates on pattern bolts, installing 2-ft-long "bacon skins" (straps) with 3-ft-long mechanical anchor bolts in between the pattern bolts or covering the roof with synthetic grid material when roof skin failure is a problem. For rib skin failures at the face, some mines install 4- to 6-ft-long planks with 18- or 36-in bolts. When sporadic rib failures occur outby the face area, mines mainly use timbers set close to the ribs to minimize the dangers to mine workers traveling nearby.

The following is taken from a roof control plan from a mine in Pennsylvania, which describes typical rib skin control methods: "Loose ribs are to be blocked, bolted, or taken down. Steel straps, planks, or header blocks with 4- to 6-ft-long bolts may be used. Bolts are not to exceed 8-ft intervals. In lieu of the above, such ribs may be supported by posts or cribs installed tightly near the rib."

ADVANCES IN SKIN CONTROL

Improved skin control and elimination of skin failure injuries, especially those resulting from roof skin failures, are contingent on providing increased surface control. To this end, safer, faster, and more efficient installation of mesh is the surface control method receiving the most attention. For instance, the walk-through bolter allows sheets of mesh to be installed with minimal worker exposure to the unsupported roof. The mesh can be placed on top of the protective canopy, then slid forward into place without the workers ever leaving the supported roof area. The method of installing synthetic mesh material is also being improved. An automatic grid dispenser has been developed that mounts on the inby side of the ATRS and dispenses the mesh up and over the ATRS (figure 10). As the mesh leaves the dispenser, the folded edges fan out from 9 to 15 ft in width to provide almost complete rib-to-rib coverage. In addition, the mesh has been strengthened to >13,000 psf to provide a material with similar strength and protection as those of conventional mesh materials. Figure 11 shows the use of synthetic mesh to support roof and rib.

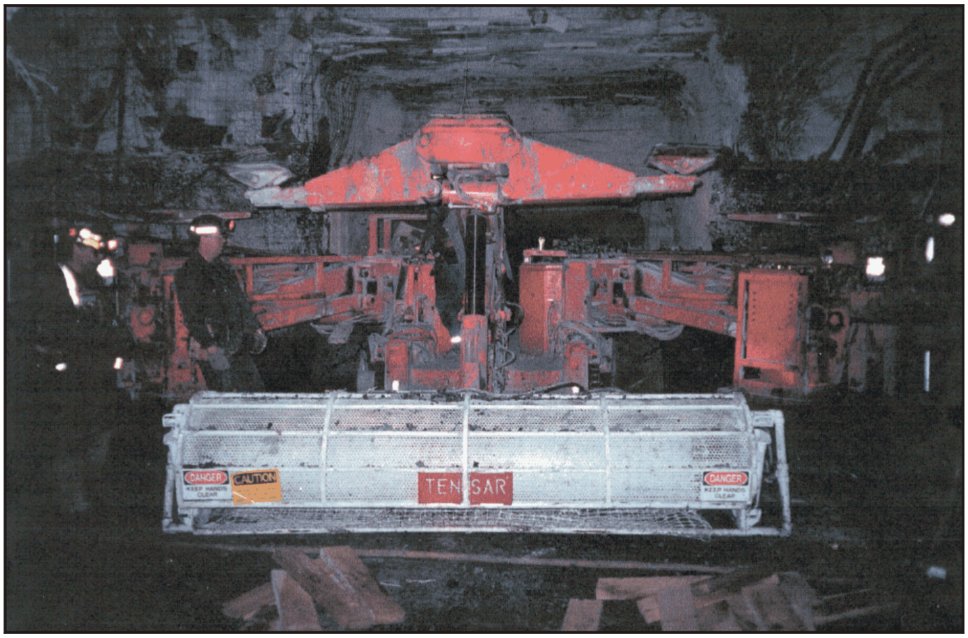


Figure 10.—Automatic grid dispenser. (*Photo courtesy of Tensar Earth Technologies.)*

Figure 11.—Synthetic mesh supporting roof and rib. (*Photo courtesy of Tensar Earth Technologies.*)

EQUIPMENT SAFETY IMPROVEMENTS

Visits to several roof bolting machine (RBM) manufacturers revealed that although they did not use the term "skin failure" to describe these types of roof and rib failures, they are aware of the problem and have been modifying the RBMs accordingly. Most of the safety modifications involved either removing the worker from the hazardous area or increasing the surface coverage of protective canopies, ATRS, etc., to prevent falling roof and rib from striking the RBM operator.

To remove the operator from the hazardous area, roof bolters with walk-through chassis, with or without automated drill functions, have been developed (figure 12). The major advantage of the walk-through chassis is to reduce mine worker exposure to rib hazards. Another manufacturer's RBM, currently available for higher coal seams only, uses four automated booms for drilling the bolt holes. The Multibolter is also a walk-through design (figure 13). It allows the operators to remain under a canopy equipped with side slide extensions that provide substantial work area coverage from roof hazards. This machine also uses side shields or chain curtain on the walkway platform to prevent rib failure injuries.

To provide additional protection to the operator during the bolting process, several machine modifications have been introduced. Many of the ATRS are equipped with hydraulic or manually extendable beams or roof contact pads to provide more coverage between the ATRS and the rib. At least one RBM manufacturer provides rock deflectors, called rocker pads, on the inby side of the ATRS that deflect rocks toward the face rather than allowing them to roll back onto the operator's legs and feet (figure 14). This was developed in response to injuries that occurred from dislodged rocks falling back onto the operator when the ATRS is lowered. The deflector forces the loose rocks to slide toward the face, falling flat against the mine floor, rather than landing on edge and falling over onto the operator's feet and legs. Rock deflector plates are also provided on the ATRS boom that can help deflect falling rocks away from the RBM operators. Another safety improvement is a sliding extension of the drilling canopy to provide additional surface coverage (figure 15).

Because the operator is subject to falling rocks any time that he or she is drilling or inserting bolts, one manufacturer developed a hydraulic resin inserter that keeps the operator from having to reach out from under the drilling canopy. Another improvement is to use reduced thrust, rotation, and feed when starting to drill a bolt hole. Accident statistics have shown that many injuries occur from falling pieces of roof rock when bolt holes are started. Some mines have even adopted the use of metatarsal gloves to protect the hands of RBM operators.

Ultimately, all RBM safety improvements are driven by the desire to provide the safest work environment for the roof bolting machine operators. Unfortunately, acceptance of these design changes can hinge on how they affect the bolting process. Changes that maintain the status quo or reduce bolting cycle times are more readily adopted by the mining industry than those that increase the time to perform any one function in the bolting process. This is because in most room-and-pillar operations the ability to mine the coal has outpaced the ability to support the roof. Thus, the speed and efficiency of the roof bolting operation is the critical production function.

Figure 12.—Walk-through chassis roof bolting machine. (*Photo courtesy of J. H. Fletcher and Co.*)

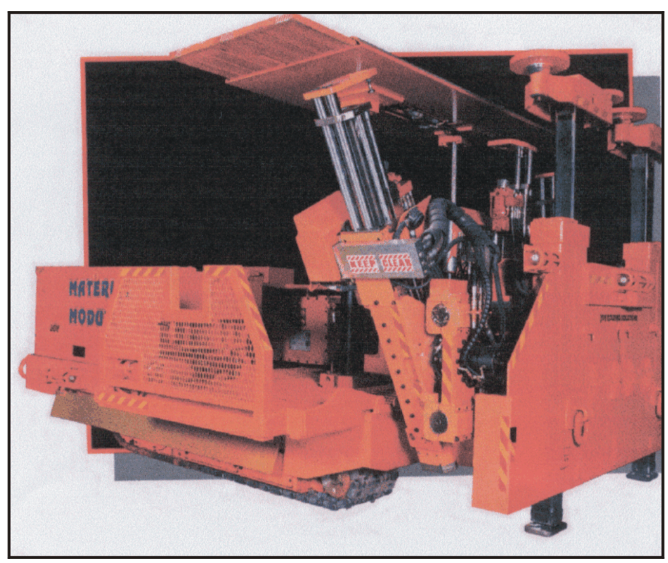


Figure 13.—Joy Multibolter. (*Photo courtesy of Joy Mining Machinery.*)

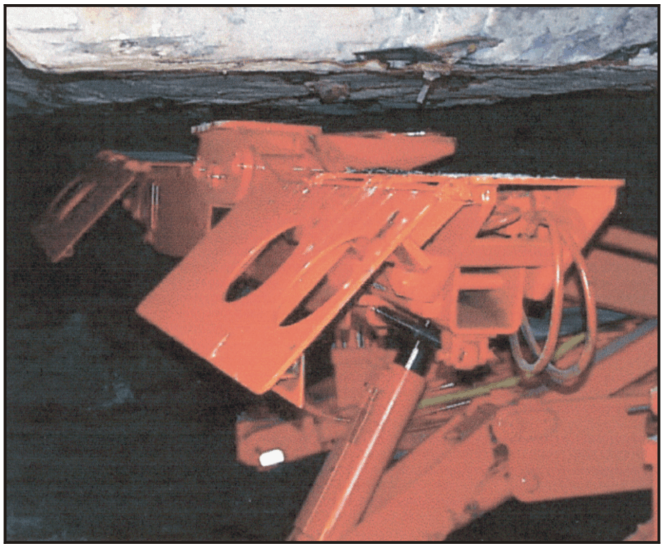


Figure 14.—Inby rocker pads to deflect falling rocks. (*Photo courtesy of J. H. Fletcher and Co.*)

Figure 15.—Pullout canopy extension. (*Photo courtesy of J. H. Fletcher and Co.*)

SUMMARY AND CONCLUSIONS

Skin failure of roof and ribs injures many workers in underground coal mines. Statistics from 1997 indicate that 98% of the injuries from roof and rib falls are due to skin failures, resulting in more than 800 injuries, or approximately 12% of all underground coal mine injuries.

Skin failure is defined as the failure of small pieces of rock and coal from between the primary supports, rather than massive roof falls or coal pillar failures. Coal ribs may not be supported, but when the rib spalls, it is still considered skin failure.

An analysis of roof and rib skin failure injuries revealed that far more injuries resulted from roof skin failures, but that the rib skin failures caused more severe injuries. The analysis also revealed that roof and rib skin failures were three times more likely to occur on the working section than outby in other mine areas because of greater worker activity in the face area and because the face is an active stress zone. From a worker activity standpoint, the roof bolters have by far the most injuries from roof skin failure. By contrast, the risk of injury from coal rib falls seems to be approximately the same for all face workers.

The methods for support of roof and ribs to prevent skin failure are simply extensions of standard roof support methods. As dictated by the extent of skin failure problems, the on-cycle supporting methods are modified to provide additional surface coverage. Common skin control methods include oversized plates, header boards, wood planks, steel straps, mesh, and in rare instances, spray coatings. These control methods can control skin failures. Unfortunately, they are implemented reactively to control problems that are occurring, rather than proactively to prevent future skin failure occurrences. The success of these controls can be enhanced by matching the characteristics of the support to the expected strata reactions to mining and modes of failure. However, the key to preventing injuries will be the amount of surface coverage developed by the surface control systems.

Equipment safety enhancements, especially to the roof bolting machine, have been directed at removing the worker from the dangerous areas and/or increasing the area of protective canopies. The modifications can provide additional measures of safety to the roof bolting machine operators, thereby reducing the potential for injuries from falling roof and rib. Unfortunately, it is difficult to get some of the equipment modifications adopted by the mining industry. Only those changes that either maintain the status quo or that speed up the bolting cycle are readily accepted, whereas other safety modifications are more difficult to implement.

NIOSH research is continuing to address the causes and control of skin failure in underground coal mines. Emphasis will be placed on determining the geologic and stress conditions associated with roof and rib skin failure and the best surface control practices being used by the coal industry to minimize the hazard of skin failure injuries.

REFERENCES

Artler L [1974]. Coal mine sealants. Min Cong J *60*(12): 34-39.

Bauer ER, Pappas DM, Dolinar DR, McCall FE, Babich DR [1999]. Skin failure of roof and rib in underground coal mines. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 108-115.

Brown GM [1941]. Keep damp roof rock from spalling by spraying with coal-tar paint. Coal Age *46*(5):64-65.

Cecil OS III [1968]. Shotcrete for ground support in underground excavations: a state of the art report. Urbana, IL: University of Illinois, Department of Civil Engineering, September.

Cummings RA, Singh MM, Moebs NN [1983]. Effects of atmospheric moisture in the deterioration of roof shales. Min Eng *3*(53):243-245.

Dale JM, Ludwig AG [1972]. Sulfur coatings for mine support. San Antonio, TX: Southwest Research Institute. U.S. Bureau of Mines contract No. H0210062.

Dolinar DR [1993]. Techniques to increase yield pillar residual strength. In: Peng SS, ed. Proceedings of the 12th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 284-291.

Dolinar DR, Tadolini SC [1991]. Entry stabilization utilizing rib bolting procedures. Denver, CO: U.S. Department of the Interior, Bureau of Mines, RI 9366.

Fletcher JH, Cassidy SM [1931]. Air cooling to prevent falls of roof rock. Trans AIME, vol. 94, February, pp. 9-26.

Franklin JC, Weverstad KD, Marquardt RF [1977]. Polymeric sealant used to stop shale degradation in coal mines. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, TPR 103.

Hartman I, Greenwald HP [1941]. Effects of changes in moisture and temperature on mine roof. First report on strata overlying the Pittsburgh Coalbed. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, RI 3588.

Herbert CA [1940]. Cooling mine air during summer months to prevent roof falls. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 7098.

Horino FG, Duvall WI, Brady BT [1971]. The use of rock bolts or wire rope to increase the strength of fractured model pillars. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, RI 7568.

Martin E, Carr F, Hendon G [1988]. Strata control advances at Jim Walter Resources, mining division. In: Peng SS, ed. Proceedings of the 7th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 66-75.

Molinda GM, Mark C [1994]. Coal mine roof rating (CMRR): a practical rock mass classification for coal mines. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9387.

Pappas DM, Vallejo LE [1997]. The settlement and degradation of nondurable shales associated with coal mine waste embankments. In: Hudson JA, ed. Proceedings of the 36th U.S. Rock Mechanics Symposium. New York, NY: Columbia University, paper No. 241.

Paul JW [1928]. Falls of roof in bituminous coal mines (influence of the seasons and rate of production). Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, TP 410.

Peng SS [1986]. Coal mine ground control, second edition. New York, NY: John Wiley & Sons, pp. 417-420.

Robbins VC [1937]. New roof treatment proves out at southwestern mines. Coal Age *42*(5):219.

Shacikaski A [1951]. Coating for roof protection. Coal Age *56*(12):88-90.

Smith WC [1989]. Evaluation of progressive rib failure in thick coal seams. SME preprint 89-174. Littleton, CO: Society for Mining, Metallurgy, and Exploration, Inc.

Wykoff BT [1950]. Wrapping pillars with old hoist rope. Trans AIME *187*(8):898-902. New York, NY: American Institute of Mining, Metallurgical, and Petroleum Engineers.

DESIGN OF ROOF BOLT SYSTEMS

By Christopher Mark, Ph.D.[1]

ABSTRACT

Roof bolt system design means the selection of the type, length, capacity, and pattern of bolts for a particular application. Despite research efforts dating back 50 years, no design methodology has found wide acceptance. This paper begins by identifying four mechanisms that roof bolts use to reinforce the ground. It argues that the reinforcement mechanism is determined by the roof geology and stress level, not by the type of bolt. Next, the attributes of roof bolts are discussed in the light of recent research, including anchorage mechanism, installed tension, length, capacity, timing of installation, and installation quality. Several significant areas of controversy are identified. Design methods from around the world are discussed, including those based on empirical research, numerical modeling, and roof monitoring. Finally, some simple guidelines for preliminary design of roof bolt systems are proposed based on statistical analysis of roof support performance at 37 U.S. mines.

[1]Supervisory physical scientist, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

Roof bolts work with the ground to create a stable rock structure. They are the first line of defense to protect mineworkers from the hazards of ground falls. Because roof bolts use the inherent strength of the rock mass, they have many advantages when compared with earlier standing support systems. Roof bolts were first introduced in the United States shortly after World War II and quickly became the dominant mode of roof support. Resin-grouted systems represented another improvement over mechanical bolts and have been increasingly favored since the 1970s. As other countries have adopted high-production retreat longwall methods, roof bolting has spread internationally. Roof bolts largely supplanted steel sets first in Australia in the 1970s and 1980s and then in the United Kingdom and Canada during the 1990s. Currently, Germany and other European coal-producing countries are adopting them [Martens and Rattmann 1999].

Because of their central importance, roof bolts have received more research attention than any other ground control topic, with the possible exception of coal pillars. Numerous roof bolt design methods have been proposed, but a recent survey paper concluded that none "has gained any acceptance by the coal mining industry" [Fuller 1999]. It seems that the complexities of the bolt-ground interaction continue to defy complete solution.

Nevertheless, some important knowledge can be gleaned from the mass of available literature. This paper presents the state-of-the-art as it applies to reinforcement mechanisms, roof bolt attributes, and design methodologies. Some simple guidelines for roof bolt selection are then proposed.

REINFORCEMENT MECHANISMS OF ROOF BOLTS

The principal objective of roof bolting is to help the rock mass support itself. Some researchers have ascribed different support mechanisms to different types of roof bolts. For example, mechanical bolts were originally thought to work in suspension, whereas resin bolts primarily built beams [Gerdeen et al. 1979]. Others have described the beam-building mechanism of tensioned bolts, and the frictional support of fully grouted bolts [Peng 1998].

It seems, however, that the reinforcement mode is actually dictated to the bolts by the ground, rather than the reverse. The degree of reinforcement required and the principal reinforcement mechanism depends on the geology and the stress regime. Four levels of support, each using a different support mechanism, can be identified:[2]

- *Simple Skin Control:* Strong, massive roof subjected to low stress levels can be essentially "self-supporting," meaning that a major roof collapse is unlikely to occur. However, cracks, joints, crossbeds, or slickensides can create occasional hazardous loose rock at the skin of the opening (figure 1*A*). Pattern bolting is therefore required to prevent local loose rock from falling, but the bolts may be relatively short and light. Skin control is also an important secondary function of roof bolts with the other three support mechanisms.
- *Suspension:* In many mines, a thin layer of weak, immediate roof can be suspended from an overlying thick, strong unit that is largely "self-supporting" (figure 1*B*). Experience has shown that roof bolts are extremely efficient in the suspension mode [Conway 1948; Damberger et al. 1980; Mark et al. 1994b], although suspension becomes more difficult if the weak layer is more than 1 m (3 ft) thick. The Coal Mine Roof Rating (CMRR) somewhat quantifies this effect through the Strong Bed Adjustment [Molinda and Mark 1994].

- *Beam Building:* Where no "self-supporting" bed is within reach, the bolts must tie the roof together to create a "beam" (figure 1*C*). The bolts reinforce the rock by maintaining friction on bedding planes, keying together blocks of fractured rock, and controlling the dilation of failed roof layers [Peng 1998; Gale et al. 1992]. In general, it is much more difficult for roof bolts to build a beam than it is to suspend weak rock from one.
- *Supplemental Support Required:* Where the roof is extremely weak or the stress extremely high, roof bolts alone may not be sufficient to prevent roof failure from progressing beyond a reasonable anchorage horizon (figure 1*D*). In these cases, cable bolts, cable trusses, or standing support may be necessary to carry the dead-weight load of the broken roof, and the roof bolts act primarily to prevent unraveling of the immediate roof [Scott 1992].

In practice, these mechanisms are not always clearly defined. In particular, the transition between suspension and beam building depends heavily on the level of stress. A roof bed that is "self-supporting" when subjected to low stress may require reinforcement when the stresses increase. Wider spans also reduce the self-supporting ability of the roof [Mark and Barczak 2000]. Figure 2 summarizes the concepts presented here.

[2]It is interesting to note that Thomas, in 1954, listed the same first three mechanisms of roof bolt support, although his definitions varied somewhat from the ones given here.

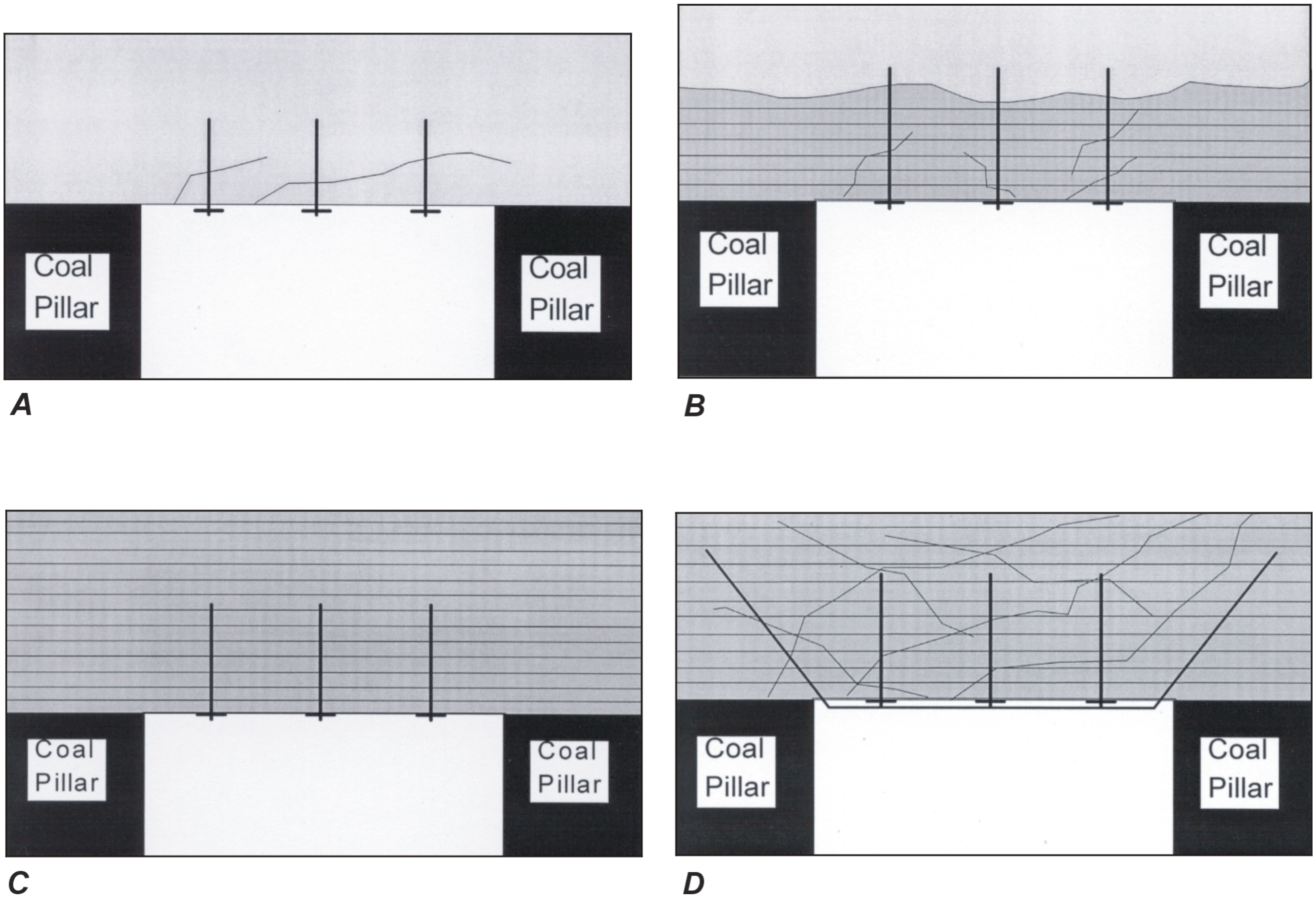


Figure 1.—Roof support mechanisms. *A*, simple skin support; *B,* suspension; *C*, beam building; *D*, supplemental support in failing ground.

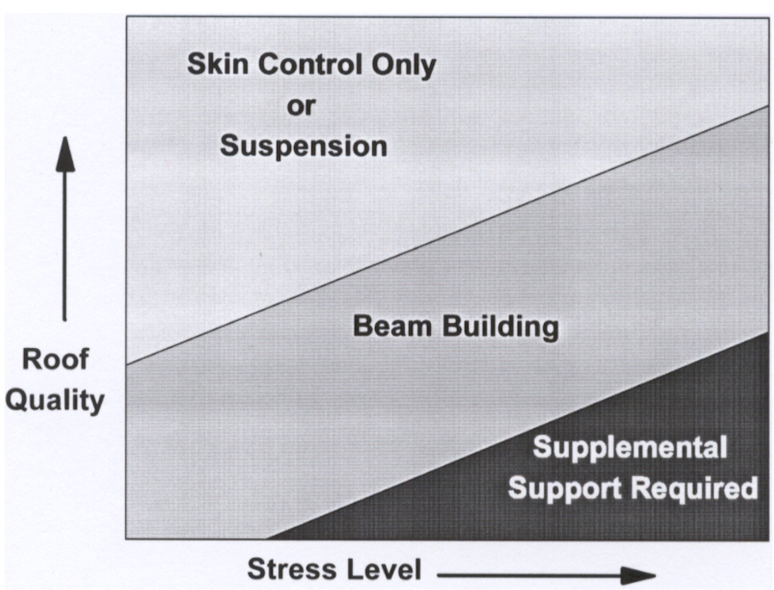


Figure 2.—Roof support mechanisms determined by stress level and roof quality.

CHARACTERISTICS OF ROOF BOLTS

Roof bolts are defined by a number of characteristics, including anchorage mechanism, installed tension, length, etc. The relative importance of these individual attributes have sometimes been the subject of much controversy.

Anchorage Mechanism–Point-Anchor Bolts: Two basic types of anchorage are available: *point-anchor* and *fully grouted*. Mechanical shells are the older type of point anchors, but these have now largely disappeared from U.S. mines [Dolinar and Bhatt 2000]. Today, resin-assisted mechanical anchor bolts are often used to support difficult conditions.

Point-anchor bolts carry high loads at the anchor and at the collar, but do not contact the rock over most of their length. Since they must be installed with tension, their initial stiffness is "infinite" until the rock load exceeds the initial tension. However, because their further response to any rock movement is distributed along their entire length, the stiffness of point-anchor bolts is lower than that of fully grouted bolts [Karabin and Hoch 1980] (figure 3).

Pullout tests are the standard technique for determining the anchorage capacity of point-anchor bolts. The anchorage is considered adequate if it exceeds the breaking strength of the bolt. If the anchorage is found to be inadequate, it may be improved in a number ways [Mazzoni et al. 1996]. Because point-anchor bolts that lose their installed tension are almost entirely ineffective, Federal regulations at 30 CFR 75.204 require that they be tested. Anchor creep was the biggest problem with mechanical bolts, but this is seldom a problem with resin-assisted point-anchor bolts. Roof deterioration at the plate is another concern, and wooden headers should be avoided because they can creep under load and shrink as they dry.

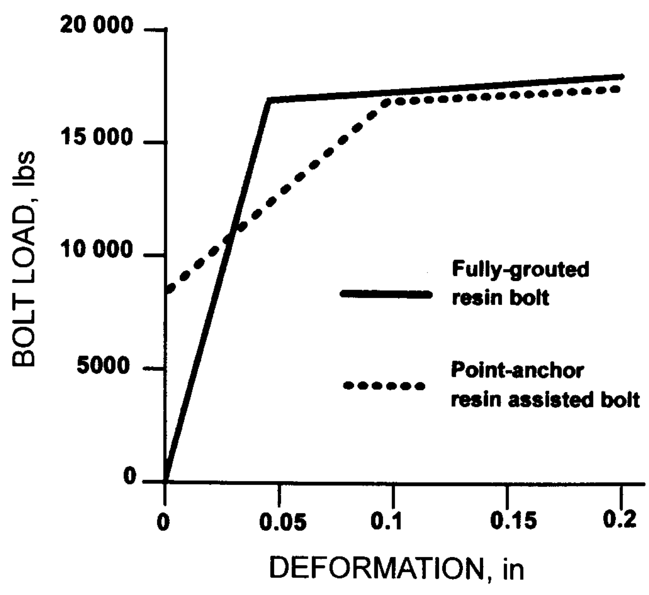


Figure 3.—Stiffness of fully grouted and resin-assisted point-anchor bolts compared (using data from Karabin and Hoch [1980]).

Anchorage Mechanism–Fully Grouted Bolts: Fully grouted bolts are loaded by movement of the rock. The movement may be vertical sag, shear along a bedding plane, or dilation of a roof layer buckled by horizontal stress (figures 4-5). The move-ments cause tensile forces in the bolt, often combined with bending stresses [Signer 2000; Fabjanczyk and Tarrant 1992]. Figure 6 shows typical load distributions in a fully grouted bolt.

The stiffness of a fully grouted bolt is determined by the load-transfer mechanisms between the rock, the grout, and the

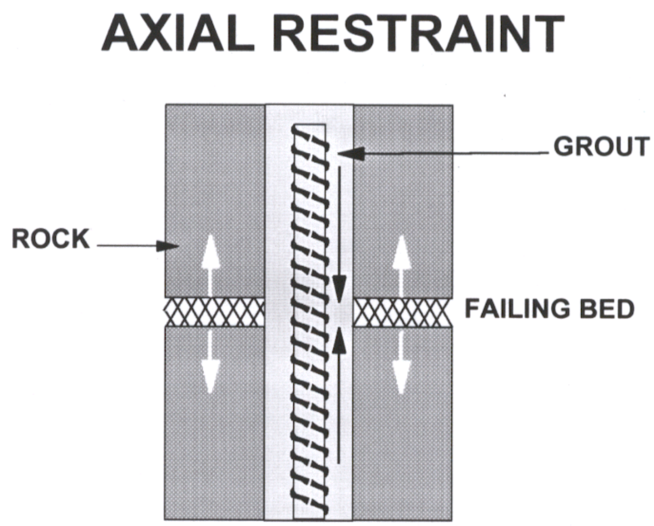


Figure 4.—Tension in a fully grouted bolt caused by dilation of a failed roof bed.

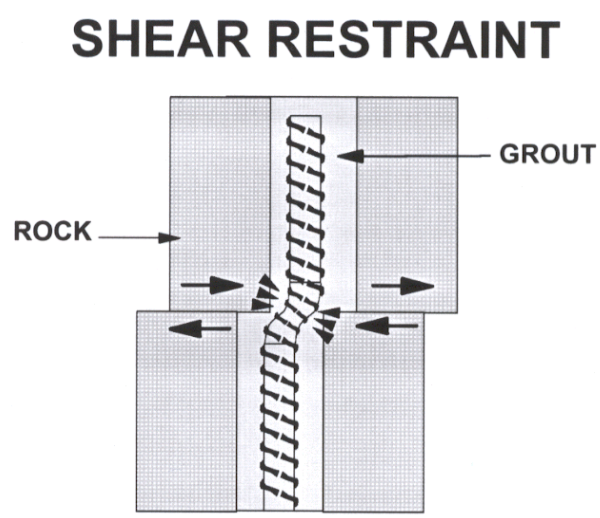


Figure 5.—Tension and bending in a fully grouted bolt caused by slip on a bedding plane.

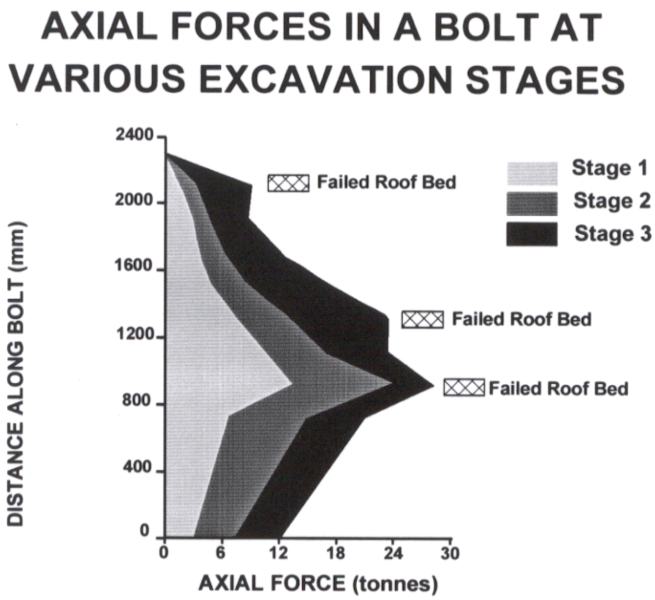


Figure 6.—Typical load distributions measured in a fully grouted bolt at three time during its service life (after Gale [1991]).

bolt. Signer [1990] provides an excellent discussion of load transfer mechanisms. Good load transfer exists when very high loads develop in the bolt in response to small ground movements, and these loads are rapidly dissipated away from the zone of roof movement. Poor load transfer can result in [Fabjanczyk and Tarrant 1992]:

- Large plate loads;
- Larger roof movements before maximum bolt response; and
- Lower ultimate bolt capacity, particularly if roof movements occur near the top of the bolt.

One way of expressing the effectiveness of load transfer is the "bond strength." Bond strength is actually a misnomer because there is no adhesion between the resin and the rock, just mechanical interlock [Karabin and Debevic 1976]. In this paper, the term "anchorage factor" will be substituted for "bond strength." The anchorage factor is obtained from short encapsulation pull tests (figure 7), in which the grouted length is short enough that the anchorage fails before the bolt yields [Karabin and Debevic 1976; Health and Safety Executive 1996]. The anchorage factor, in kilonewtons per millimeter or tons per inch, is determined by dividing the applied pulling load by the anchorage length. Typically, no more than 300 mm (12 in) of the bolt is grouted in a short encapsulation test, and tests may be conducted at a variety of depths to evaluate the load transfer characteristics in different roof beds. Standard pullout tests should not be employed with full-length resin bolts because the pulling forces seldom extend more than 450 mm (18 in) up the resin column [Serbousek and Signer 1987].

Table 1 gives typical anchorage factors and anchorage obtained from the literature. Short encapsulation tests are apparently rather rare in the United States; the only available published data were obtained from Peng [1998]. Although the

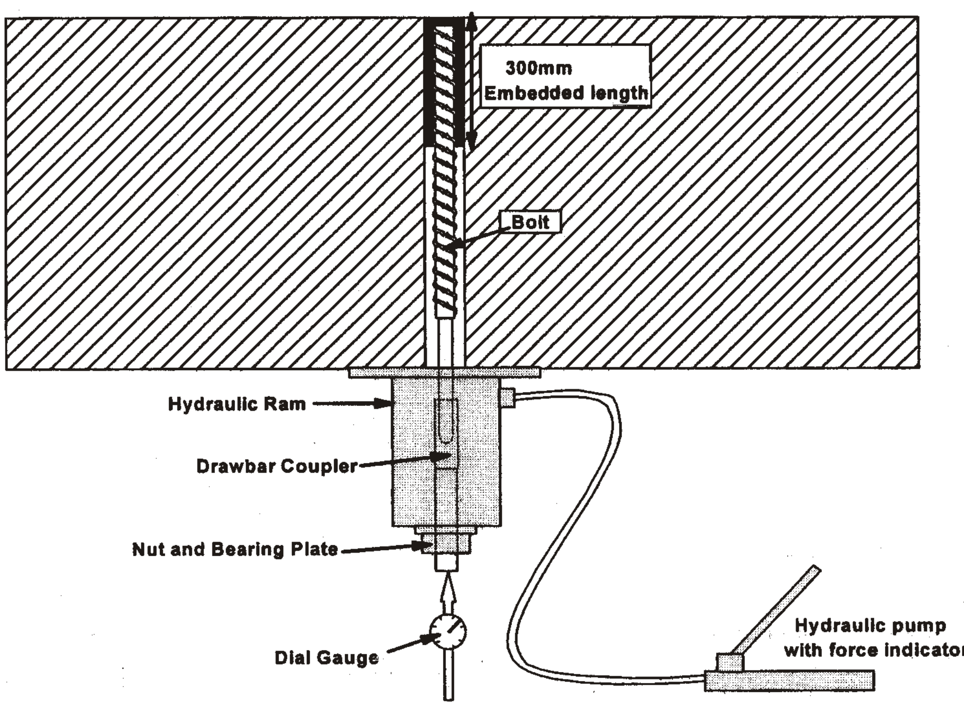


Figure 7.—A short encapsulation pull test.

Table 1.—Anchorage factors for fully grouted resin bolts

Rock type	Country	Anchorage factor, N/mm	(tons/in)	Length for 90 kN (10 tons) of anchorage, mm	(in)
Coal, shale	Australia	300- 900	(0.7-2.1)	100-300	(4-12)
Hard sandstone, limestone	Australia	1,000-2,500	(2.3-5.8)	35- 90	(1.4-3.6)
Minimum allowable	U.K.	400	(1.1)	225	(8.9)
Soft rock	U.S.A.	180	(0.5)	510	(20)
Strong rock	U.S.A.	720	(2)	125	(5)

Australian data [Yearby 1991] and the U.K. data [Bigby 1997] probably apply to slightly larger bolts, there seems to be a clear difference. The implication is that in weak rock in the United States, the top 500 mm (20 in) or more of a fully grouted bolt may required to develop an anchorage force equal to the breaking strength of the rod. In such conditions, the "effective capacity" of the upper portion of the bolt may be considerably less than its nominal capacity.

A number of factors can affect the load transfer characteristics and anchorage factor, including—

Rock Strength: Weaker rock requires a longer grouted length to achieve the same anchorage capacity as strong rock [Franklin and Woodfield 1971; Karabin and Debevic 1976]. One study of the former U.S. Bureau of Mines [Cincilla 1986] found that coal and shale roofs required an average of 800 mm (31 in) of grouted length to achieve full anchorage, while sandstone required 460 mm (18 in) and limestone needed just 300 mm (12 in). In very weak rock, anchorage factors can be so low that 1.6-m (6-ft) bolts have been pulled from the rock at 14 tons even though they were fully grouted for their entire length [Rico et al. 1997].

Hole annulus: Numerous tests over the years have found that optimum difference between the diameter of the bolt and the diameter of the hole is no greater than 6 mm (0.25 in), giving an annulus of about 3 mm (0.125 in) [Fairhurst and Singh 1974; Karabin and Debevic 1976; Ulrich et al. 1989]. For example, a 3-mm (0.125-in) annulus is obtained by a 19-mm (0.75-in) bolt in a 25-mm (1-in) hole. Results from short encapsulation pull tests on 19-mm (0.75-in) bolts are shown in figure 8.

Larger holes can result in poor resin mixing, a greater likelihood of "finger-gloving," and reduced load transfer capability. One Australian study found that the load transfer improved more than 50% when the annulus was reduced from 4.5 to 2.5 mm (0.35 to 0.1 in) [Fabjanczyk and Tarrant 1992]. Smaller holes, on the other hand, can cause insertion problems and magnify the effects of resin losses to roof cracks or to overdrilled holes [Campoli et al. 1999]. However, one recent U.S. study found that annuli ranging from 2.5-6.5 mm (0.1-0.25 in) all provided acceptable results in strong rock [Tadolini 1998]. Also, if failure is occurring at the resin-rock interface in very weak rock, increasing the hole diameter is one way to decrease the shear stress on the interface [Rico et al. 1997].

Hole and bolt profile: Because resin grout acts to transfer load by mechanical interlock, not by adhesion, rifled holes and rougher bolt profiles result in better load transfer [Karabin and Debevic 1976; Haas 1981; Aziz et al. 1999]. Reportedly, wet drilled or water-flushed holes can also improve load transfer [Siddall and Gale 1992]. One study found that the pullout load of standard rebar was seven times that of a smooth rod [Fabjanczyk and Tarrant 1992].

Resin characteristics: Tests in the United Kingdom in the late 1980s demonstrated that the compressive strength of resin was important to the performance of grouted roof bolts [British Coal Technical Department 1992], and current U.K. regulations require resin strength to exceed 80 MPa (11,000 psi). A strength test was recently added to the American Society for Testing and Materials (ASTM) standards for resin. However, an extensive series of laboratory "push tests" found little correlation between shear stress and resin strengths in the 20-60 MPa (3,000-6,000 psi) range [Fabjanczyk and Tarrant 1992].

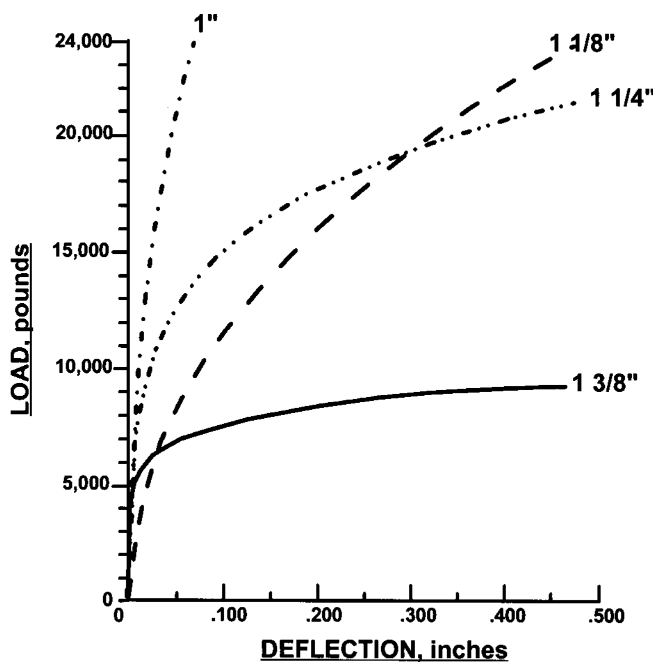


Figure 8.—Effect of hole annulus on grouted bolt performance. Results were obtained from short encapsulation pull tests on 19-mm (0.75-in) diameter rods (after Karabin and Debevic [1976]).

In summary, good load transfer is essential for optimizing the performance of resin bolts, particularly in weak rocks. U.S. mines have been criticized for using "vacuum drilling, large diameter holes, and low strength resin" [Hurt 1992]. Although field measurements indicate that U.S. resin bolts usually respond quickly to roof movements, which indicates good load transfer properties [Signer and Jones 1990; Signer et al. 1993; Maleki et al. 1994; Signer and Lewis 1998], low anchorage factors may reduce the effective capacity of the upper portion of bolts installed in some weak rock conditions. It may be possible to improve bolt performance by adjusting load transfer properties such as hole size or rifling. More widespread use of short encapsulation pull tests (figure 8) could be very helpful in identifying when and where low anchorage factors may be a problem.

INSTALLED TENSION

One of the most controversial topics in roof bolting is the importance of installed tension. Numerous papers have been written pro and con in Australia and the United States. The issue can be further confused because there are actually three possible systems: fully grouted nontensioned, fully grouted tensioned, and point-anchor tensioned.

In the United States, Peng [1998] argues that resin-assisted point-anchor tensioned bolts can be used to clamp thinly laminated roof beds into a thick beam that is more resistant to bending. Stankus and Peng [1996] add that by "increasing frictional resistance along bedding planes, roof sag and deflection is minimized, and lateral movement due to horizontal stress is unlikely to occur." Tensioned bolts are also said to be more efficient, because "a stronger beam can be built with the same bolt by utilizing a larger installed load."

Frith and Thomas [1998] advocate pretensioning fully grouted bolts using two-stage resins and special hardware. They argue that active preloads modify roof behavior by dramatically reducing bed separation and delaminations in the immediate 0.5-0.8 m (2-3 ft) of roof. A key reason that tension works, they say, can be understood if the roof is seen as an Euler buckling beam. Small vertically applied loads therefore have a mechanical advantage that allows them to resist high horizontal forces (figure 9). Fuller [1999] concludes that "the generally positive results of field trials indicates that pre-tensioning when combined with full bonding of bolts provides the maximum strata reinforcement."

Gray and Finlow-Bates [1998] put the case that non-tensioned, fully grouted bolts with good load transfer characteristics may be just as effective. They argue that a preload of 100 kN (12 tons) results in a confining stress of only 70 kPa (10 psi) on the roof, which is minimal compared with in situ horizontal stresses which are at least 100 times greater. Also, the loads dissipate rapidly into the rock. Others have observed that in field measurements, resin bolts have quickly achieved loads that are even greater than those on nearby point-anchor bolts [Mark et al. 2000]. McHugh and Signer [1999] found that in laboratory tests, the confining loads applied by pretensioned, fully grouted bolts did little to strengthen rock joints.

Unfortunately, direct comparisons of the three systems are relatively rare. Anecdotal evidence is often cited, sometimes from situations where bolt length and capacity were changed as well as tension [Stankus 1991]. There is general consensus that large preloads are not necessary for resin bolts to function effectively in the suspension mode [Peng 1998; Frith and Thomas 1998; Maleki 1992], but broader conclusions apparently must wait for more research.

It should be pointed out that fully grouted bolts are not entirely tension-free. In the United States, there is typically about 11 kN (1 ton) of plate load when the bolts are installed [Signer 1990]. Plate loads can increase by a factor of 10 or more in highly deforming ground [Tadolini and Ulrich 1986]. The thrust bolting technique can apply upwards of 44 kN (4 tons) of initial plate load [Tadolini and Dolinar 1991], which is similar to what is measured on the typical Australian "nontensioned" roof bolt [Frith and Thomas 1998].

BOLT CAPACITY

The yield capacity (C) of a roof bolt is normally determined by the bolt diameter (D) and the grade of the steel (G):

$$C = \left(\frac{\pi}{4}\right) G D^2 \tag{1}$$

For rebar, the diameter is usually given as a number, where #5 rebar is 5/8 in (16 mm) in diameter, #6 is 0.75 in (21 mm), and so on. The grade of the steel is normally given in thousands of psi, where a grade 40 steel is 40,000 psi (280 MPa), etc. The grade and the diameter, and some other information including the bolt length, are stamped on the head of the bolt, using the symbols shown in table 2.

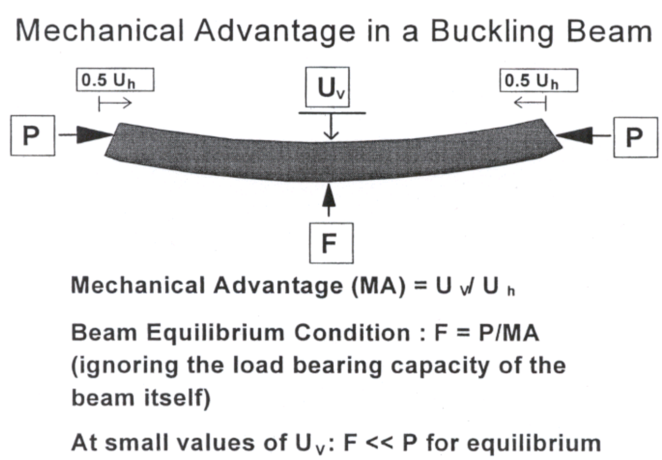


Figure 9.—The Euler buckling beam concept (after Frith [1988]).

Table 2.—Markings on the heads of roof bolts

(ASTM F432-95, "Standard Specification for Roof and Rock Bolts and Accessories")

Headed bolts	Nominal product size, in	Mfg. symbol[1]	Diameter[2]	Grade[3]	Length, in
GR 40	3/4 and over	Yes	Yes	None	Yes
GR 55	5/8 and over	Yes	Yes	✳	Yes
GR 60	5/8 and over	Yes	Yes	▵	Yes
GR 75	5/8 and over	Yes	Yes	X	Yes
GR 100	5/8 and over	Yes	Yes	□	Yes

[1]Enter alpha-numeric symbol.
[2]Enter numerical value of bolt diameter measured in eighths of an inch; numerical value of deformed bars placed in circle.
[3]Grades above 100 are produced in 20-ksi increments; they are marked 2 for 120 ksi, etc.

The ultimate capacity of a bolt is often considerably greater than the yield. Table 3 shows yield and ultimate capacities for several common bolts. In general, lower grade steels are more ductile than high-strength steels, meaning that there is a relatively greater difference between the yield and the ultimate strength. Signer [1990] points out that while a typical rebar will yield after 0.8 mm (0.030) in of deformation, an additional 50 mm (2 in) is required to break it.

Table 3.—Load-carrying capacities of mine roof bolts

Roof bolt material	Minimum yield, MPa (psi)	Minimum ultimate tensile, MPa (psi)
5/8 Grade 55	86 (12,400)	132 (19,200)
5/8 Grade 75	117 (17,000)	156 (22,600)
3/4 Grade 75	173 (25,100)	230 (33,400)
#6 Rebar Grade 40 ...	121 (17,600)	212 (30,800)
#6 Rebar Grade 60 ...	182 (26,400)	273 (39,600)
#7 Rebar Grade 40 ...	166 (24,000)	290 (42,000)
#7 Rebar Grade 60 ...	248 (36,000)	372 (54,000)
#5 Rebar Grade 60 ...	127 (18,600)	190 (27,900)

Several factors may cause the actual bolt capacity to be somewhat less than the capacity of the rod. The most obvious is if the anchorage is inadequate. Although all bolts must be tested to ensure that they meet ASTM specifications, coupled bolts are sometimes prone to fail at the coupler. Poor installation can also cause a stress concentration at the bolt head. In thin seam mines, bolts are sometimes notched so that they can be bent more easily. The cross-section area of the steel left in the notch then determines the bolt capacity. In general, notches rolled into the bar reduce strength less than machined notches.

Many authors argue in favor of greater capacity to improve the effectiveness of roof bolts [Gale 1991; Stankus and Peng 1996]. One obvious advantage is that stronger bolts can carry more broken rock. Higher capacity bolts are also capable of producing more confinement and shear strength in the rock, and they may be pretensioned to higher levels. Larger diameter bolts are also stiffer.

The increased capacity may not be utilized in all circumstances, however. Field studies show that bolts are not loaded equally, and the roof may fail on one side of the entry before the bolts on the other see significant loads (figure 10). More importantly, if the roof is failing above the bolts, it may fall without ever loading them. On the other hand, if broken bolts are observed in roof falls, increased bolt capacity is clearly indicated.

BOLT LENGTH

The optimal roof bolt length depends on the support mechanism. Where bolts are merely acting as skin control, they may be as short as 750 mm (30 in). In the suspension mode, bolts should obtain at least 300 mm (1 ft) of anchorage in the solid strata. Federal regulations at 30 CFR 75.204 require that when point-anchor bolts are used, test holes must be drilled at least another 300 mm (1 ft) above the normal anchorage.

In some mines, the thickness of the weak, immediate roof layer can vary by as much as 1 m (3 ft) over very short distances. In these mines, roof bolt crews select the proper length bolt based on their observations while drilling. They sense where they contact the strong bed from the sound and penetration rate of the drill. Computerized feedback control technologies are now being developed which may aid drill operators in identifying strong anchorage horizons [Thomas and Wilson 1999].

The proper bolt length is more difficult to determine in the beam-building mode. Some empirical rules of thumb that have been suggested include:

$$B_L = S^{2/3} \quad \text{[Lang and Bischoff 1982]} \tag{2}$$

$$B_L = \frac{S}{3} \quad \text{[Bieniawski 1987]} \tag{3}$$

$$B_L = \left(\frac{S}{2}\right)\left(100 - \frac{RMR}{100}\right) \quad \text{[Unal 1984]} \tag{4}$$

where B_L = bolt length;

S = span; and

RMR = rock mass rating [Bieniawski 1987].

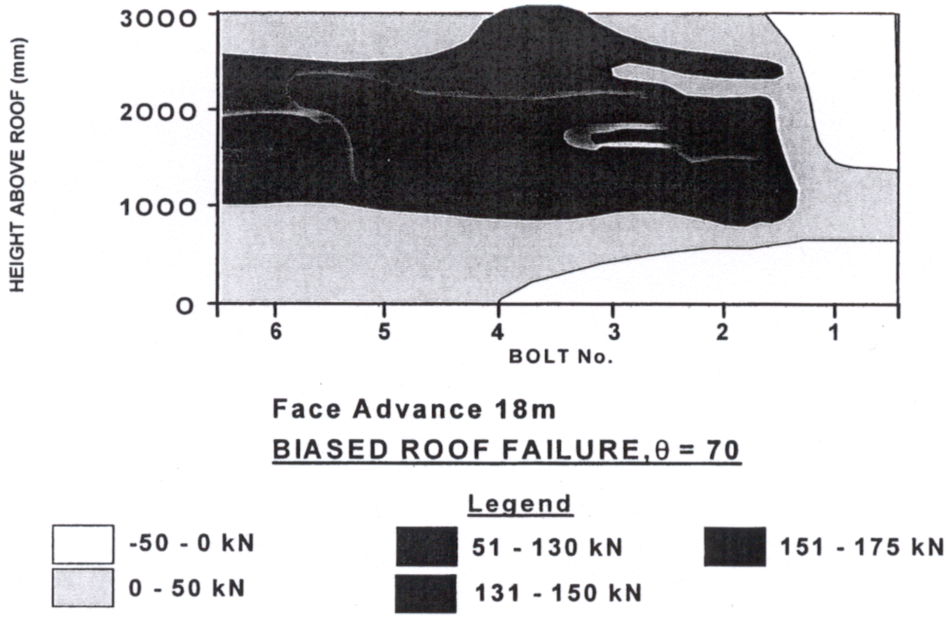


Figure 10.—Nonuniform bolt loading measured in an entry developed at an angle to the maximum horizontal stress [Gale 1991].

The Unal equation is the most appealing of the three because it considers the rock quality in addition to the span (note that the CMRR may be substituted for the RMR in equation 4). The Unal equation was not intended for intersection spans, however, nor does it consider stress level. None of the three equations have been validated for use in coal mines.

It seems that increasing bolt length can be a very effective measure for reducing roof falls. The study reported by Molinda et al. [2000] found that out of 13 mines where 2 different lengths of bolts were used in similar roof conditions, the fall rate was lower for the longer bolts 84% of the time. The same study found little support for the theory that shorter bolts installed at higher than normal tensions can reduce roof fall rates [Stankus and Peng 1996]. It should be noted, however, that the effective capacity of the upper portion of a fully grouted bolt can be significantly reduced if the load transfer is poor, whereas a resin-assisted point-anchor bolt should function along its entire length (as long as the length of the resin column is adequate).

As equations 2 through 4 suggest, wider spans require longer bolts for beam building. In coal mines, the widest spans are generally found in intersections. However, most mines use the same length bolt both in intersections and entries. This may help explain why intersections are as much as 10 times more likely to collapse (on a foot-per-foot basis) than entries [Molinda et al. 1998]. Many mines that are experiencing high rates of roof falls might be able to improve conditions by using longer bolts just in intersections.

ROOF BOLT PATTERN

The density of roof bolt support varies little in the United States. With the advent of dual-head roof bolting machines, four bolts per row has become the near-universal standard. Bolt spacing is limited by law to a maximum of 1.5 m (5 ft), but is seldom <1.2 m (4 ft). With entries varying in width from about 4.5-6 m (15-20 ft), bolt densities range from approximately one bolt per 2.4 m^2 (25 ft^2) to one bolt per 1.4 m^2 (15 ft^2).

Such patterns are appropriate for the vast majority of U.S. applications, which are for simple skin control, suspension, and beam building at relatively low stress. By international standards, however, they are quite light for beam building in high-stress conditions. In the United Kingdom, the minimum bolt density allowed by statute is one bolt/m^2 (11 ft^2), and many Australian mines use similar bolt densities. In these countries, higher bolt densities are considered necessary to maximize the strength of failed rock around the roadways [Gale et al. 1992]. The lighter patterns used in the United States may help explain why some mines have such difficulty controlling the weakest roof in highly stressed ground. Unfortunately, higher bolt densities are probably not economically feasible in the United States, primarily because of their impact on drivage rates.

One partial alternative that might be helpful in some cases is to put extra bolts in where the bolts are most heavily loaded. The field study reported by Maleki et al. [1994] found that increasing the bolt density reduced the average bolt load, while the total load remained approximately the same. Other researchers have found that when one side of the entry suffers greater stress damage, bolts on that side receive significantly more load [Mark and Barczak 2000; Siddall and Gale 1992]. Additional bolts on the stress-damage side can help maintain overall stability.

TIMING OF BOLT INSTALLATION

As soon as a cut is mined, the roof begins to move. Some relaxation is necessary to relieve the in situ stress, but excessive movement can reduce the strength of the rock mass by reducing the confinement on bedding planes and other discontinuities. The longer a roof remains unbolted, the more likely that some damage will occur.

The degree of potential damage depends on the stress level, the span, and the roof quality. Whereas strong roof may not suffer at all, weak roof under high stress may collapse before the miner completes the cut. The study of extended cuts reported by Mark [1999] found that when the CMRR exceeded 55, extended cuts were nearly always stable. In these conditions, very little damage apparently occurs before the bolts are installed. When the CMRR was between 55 and 40, most mines had mixed experiences with extended cuts, indicating that the roof tends to degrade with time and should be bolted soon after mining. Deeper mines also had more trouble than shallower ones, indicating that elevated stresses also require quick support (figure 11).

The study also found that mines with a CMRR < 38 could rarely employ extended cuts. Place-change mining, which requires that the roof stand unsupported until the bolting machine arrives, may not be economic under such conditions. The difficulties in place-change mining highly stressed, weak roof explains the prevalence of miner-bolters in the Pittsburgh Seam. Miner-bolters are single-pass machines that mine a narrower entry and allow the roof to be bolted minutes after it is exposed. Pittsburgh Seam mines have found that roof fall rates are reduced substantially with this mining method. Most mines in Australia and the United Kingdom use similar systems.

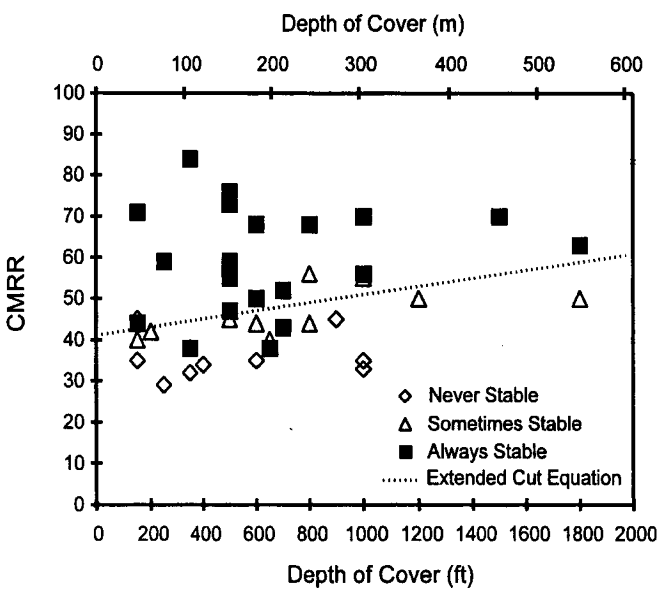


Figure 11.—Effect of the depth of cover on the stability of unsupported roof.

SKIN SUPPORT

Skin support is an essential function of roof bolt systems, serving the dual purposes of—

- Protecting miners from small rocks that could fall between the bolts; and
- Preventing the roof from unraveling and ultimately negating the purpose of the bolts.

Skin support is achieved through a combination of plates, headers, mats, straps, mesh and sealants. Skin support is the subject of a current research study under the National Institute for Occupational Safety and Health (NIOSH). Some preliminary results are reported by Bauer and Dolinar [2000].

INSTALLATION QUALITY

Poorly installed support is, at best, ineffective and, at worst, provides a false sense of security. Unfortunately, it is difficult to check the installation of most modern roof supports. Whereas timber supports can be checked visually and mechanical bolts can be checked with a torque test, resin anchors have thus far defied attempts to develop an effective testing technique.

The troubleshooting guide prepared by Mazzoni et al. [1996] provides the most complete information available on roof support quality. The guide attributes problems with roof bolts to three main sources:

- Geology;
- Poor installation quality; and
- Defective support hardware.

With fully grouted bolts, potential installation problems include—

- *Defective grout* due to improper storage, improper temperature at the time of installation, or manufacturing error;
- *Defective hole* due to crookedness, cracks, improper length, or improper diameter;
- *Poorly mixed grout* due to improper insertion, rotation, thrust, torque, spin time, or hold time; and
- *Defective bolt.* Tensioned grouted systems can suffer from all of the problems listed above, as well as defective couplers, shear mechanisms, threads, washers, and anchors.

The miners who operate roof bolt machines are the key to maintaining high-quality support installations. Certainly, there is no substitute for job training and experience. In addition, knowledge about strata reinforcement principles can be very effective in motivating roof bolt crews to ensure quality support throughout the mine [Fuller 1999].

ROOF BOLT FAILURE MECHANICS

Roof bolts can fail in one of several ways:

- The head or the plate can fail;
- The rod may break, either in tension, or a combination of tension and bending; or
- The anchorage may fail.

In addition, roof bolts may be intact, but the support system can fail if—

- The bolts are too short, allowing the roof to fail above them; or
- The bolts fail to provide adequate skin control, allowing loose rock to create a hazard or letting the roof unravel over time.

Point-anchor bolts normally fail by anchor slip or by exceeding the capacity of the steel. A sudden break can cause the freed bottom end to be released at high speed [Peng 1998]. This hazard is known as the "shotgun effect."

Studies have shown that a very high percentage of resin bolts are loaded to their yield point, sometimes very early in their service lives [Signer 2000]. Data presented by Signer [1990] seem to indicate that once the steel yields, it pulls away from the grout, greatly reducing the load transfer that takes place along that portion of the bolt. If the lower portion of the bolt yields, it can be manifested as increased plate loads (figure 12*A*). Loading in the central portion may ultimately break the rod (figure 12*B*). However, anchorage failure may occur if there is poor load transfer near the top of the bolt, whether caused by bolt yielding or not (figure 12*C*). Considering the anchorage factor data presented in table 1, if a typical U.S. roof bolt installed in weak rock was loaded in its upper 500 mm (20 in), it could be pulled out of the hole before the rod yielded.

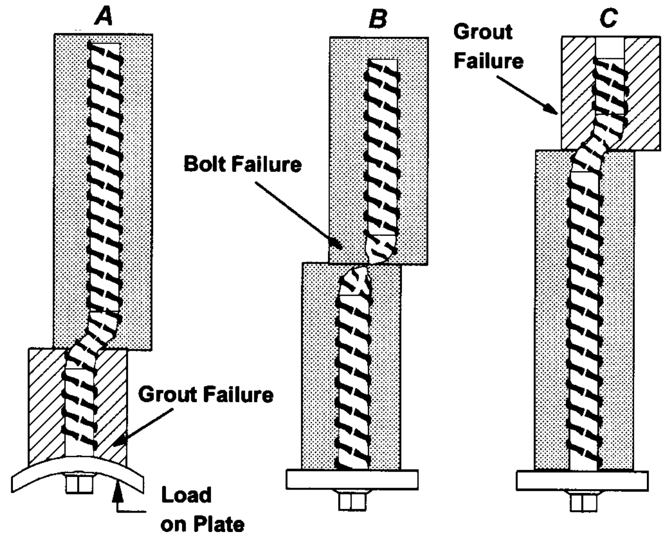


Figure 12.—Failure mechanisms of a fully grouted bolt [after Serbousek and Signer 1987]. ***A*, roof movement near head; *B*, roof movement in central portion; *C*, roof movement in anchorage zone.**

Once a standard roof bolt is loaded to its ultimate capacity, it usually has very little residual strength. Compared with many supplemental supports (e.g., wood cribs and cable trusses), roof bolts are normally effective over a relatively small range of deformation. However, there is a class of yielding roof bolts that are designed to maintain high loads through deformation ranges of 300 mm (12 in) or more. Yielding bolts normally employ a slip-nut at the bolt head. They are designed for very high deformation environments, such as long-term applications in creeping salt, or pillarless longwall extraction under extremely deep cover [Terrill and Francke 1995; VandeKraats et al. 1996; 1998; Martens and Rattmann 1998].

APPROACHES TO THE DESIGN OF ROOF BOLT SYSTEMS

Various methods for the design of roof bolts have been proposed through the years. None has achieved wide success. Today, most roof bolts are still selected using a combination of past experience, trial and error, and regulatory requirements. Much can still be learned from a review of the different concepts. The survey below briefly describes a number of theories, an approximately chronological order. The bolt design attributes that they address are also identified.

Dead-weight design (capacity/pattern): The oldest, simplest, and probably still most widely used equation for bolt design is dead-weight suspension [Obert and Duvall 1967]:

$$P = \left[\frac{U * t * W_e * R}{n + 1}\right] SF, \qquad (5)$$

where P = required bolt capacity;

U = unit weight of the rock;

t = thickness of suspended rock;

n = number of bolts per row;

W_e = entry width;

R = row spacing; and

SF = safety factor.

Figure 13 gives dead-weight loads calculated for various bolt spacings. This method is probably suitable for suspension bolting in low-stress environments. However, horizontal forces can greatly increase the loads applied to roof bolts [Wright 1973; Fairhurst and Singh 1974]. Signer et al. [1993] found that measured loads on roof bolts are often twice what would be predicted by dead-weight design.

Rock Load Height (capacity/pattern): The rock load height concept is a slightly more sophisticated version of the deadweight theory. Originally proposed by Terzaghi [1946], the theory predicts the load on the supports based on the rock quality and the span. Unal [1984] defined the rock load height for coal mining:

$$h_t = B\left[\frac{100 - RMR}{100}\right] \tag{6}$$

The rock load height is illustrated in figure 14. Again, the CMRR may be substituted for the RMR in equation 6.

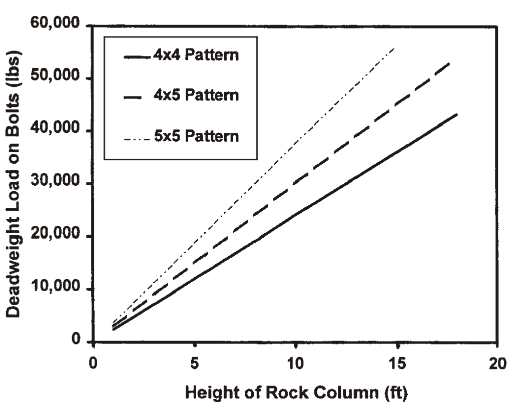


Figure 13.—Dead-weight loads on roof bolts.

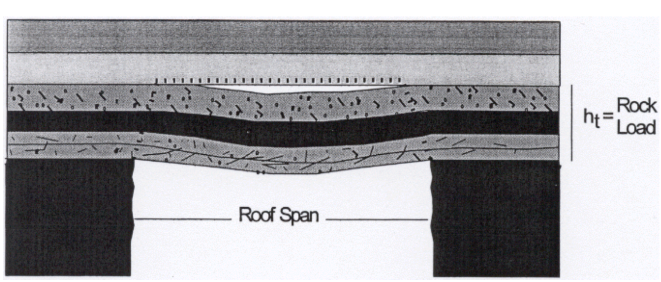


Figure 14.—The rock load height concept (after Unal [1984]).

Panek's Chart (length/tension/pattern): An early attempt at a comprehensive design procedure was presented by Panek [1964]. He conducted a series of scale model tests using limestone slabs to represent roof beds. His results were presented in the form of a nomogram that related bed thickness and roof span to the required bolt length, tension, and pattern. Remarkably, Panek's nomogram continues to be republished, although it is very doubtful that it has been used for practical design in decades [Fuller 1999].

Other Physical Models (location): In the prenumerical modeling era, several researchers used physical models to explore roof bolting performance [Fairhurst and Singh 1974; Dunham 1976; Gerdeen et al. 1979]. All of these studies assumed that the roof was perfectly bedded, and they consistently found that bolts located in the center of the entry added little to roof stability. In contrast, one model study of roof containing low-angle shears as well as bedding found that an evenly spaced pattern performs best [Mark 1982].

Peng and Guo (pattern): Peng and Guo [1989] used a hybrid boundary-element/finite-element model to design the spacing for fully grouted bolts. The models incorporated weak bedding planes, and parametric analyses were performed in which roof stiffness, layer thickness, and horizontal stress were varied. By applying dimensional analysis, they derived a series of equations that give the number of bolts required to prevent bed separation, tensile fracture, shear fracture at midspan, and shear fracture at the entry corners. Some simple guidelines for bolt length were also presented.

Two-Phase Ground Support (support type/timing): Scott [1992] proposed that when longwall entries that are expected to undergo large deformations, a two-phase ground support system might make sense. The first phase would consist of short, closely spaced rock anchors that would slip at their load-carrying capacity, but continue to prevent the immediate roof from unraveling as it deformed. The second phase would consist of long cable anchors or standing supports capable of carrying the weight of the fractured ground while accepting its dilation. Scott cited the gabion analogy in support of his theory. Scott's approach could result in a more efficient design than one that tried to prevent all deformation, and it can be argued that many U.S. longwalls that install heavy standing support in the tailgate already use a version of it.

Maleki (bolt type): Maleki [1992] proposed a preliminary criterion for bolt selection based on his analysis of 20 case histories. The factors determining the type of bolt required are the stress level and the rock mass strength. The laboratory rock strength is downgraded to give the rock mass strength as follows:

$$\text{Rock mass strength} = \frac{\text{Uniaxial compressive strength}}{K}, \tag{7}$$

where K = 1 for massive strata; K = 2 for cohesive, medium bedded strata; and K = 3 for finely laminated, noncohesive strata (figure 15).

In Maleki's approach, tensioned, fully grouted bolts are recommended for the most difficult conditions.

Design by Measurement (pattern/length): This design approach was developed in Australia [Gale 1991; Gale and Fabjanczyk 1993] and was largely adopted by the U.K. Code of Practice [Bigby 1997]. The basic concept is that as individual roof beds become overstressed and fail, they force stresses higher into the roof, which can in turn fail more beds (figure 16). Reinforcement aims to mobilize the frictional strength of failed roof beds in order to restrict the height and severity of failure in the roof. It involves measuring the loads developed in roof bolts during mining, together with a definition of the height and severity of roof deformation obtained from multipoint extensometers. Based on the measurements, optimization of the bolting design might include—

• *Adjusting the bolt length* so that adequate anchorage is achieved above the highest level in the roof where failure is occurring;

• *Adjusting the bolt density and placement* to maximize reinforcement where the roof needs it most;

• *Improving load transfer* by reducing hole size, optimizing bit type, or flushing the hole.

The results are considered valid for environments that are similar to the one studied. Significant changes in the geology or stress field requires additional monitoring.

Optimum Beaming Effect (tension/length): Stankus and Peng [1996] proposed the Optimum Beaming Effect, which is defined as the roof beam that has no separation within or above the bolted range and uses the shortest bolt possible. Its basic tenet is that high installed tensions can be substituted for bolt length. They also argue that longer bolts elongate more in response to load, therefore allowing more roof deformation. The method has been implemented in a finite-element model (see section on "Numerical Modeling" below). Unfortunately, there does not seem to be sufficient justification for this theory. Molinda et al. [2000] found that shorter, tensioned bolts had higher roof fall rates than longer, nontensioned ones in three of four cases where both bolts were used in the same mine.

Structural Engineering Model (tension): In Australia, Frith [1998] proposed a model that divides mine roof into two classes:

• *Static roof* that is essentially self-supporting and requires minimum reinforcement; and

• *Buckling roof* that is thinly bedded and tends to fail layer-by-layer due to horizontal stress.

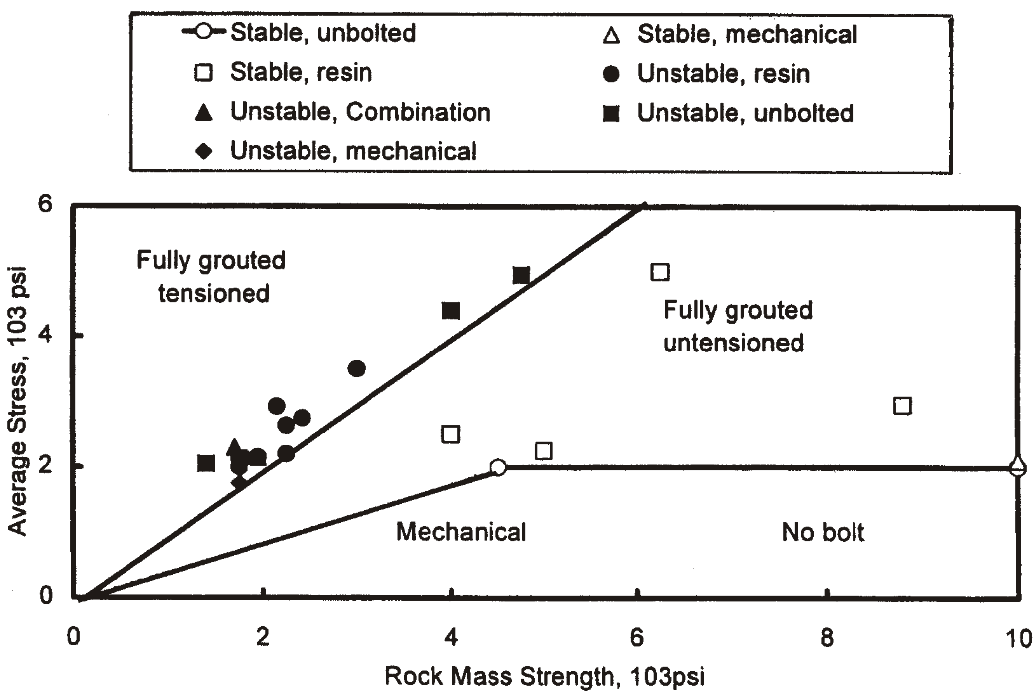


Figure 15.—Maleki's [1992] roof bolt selection chart.

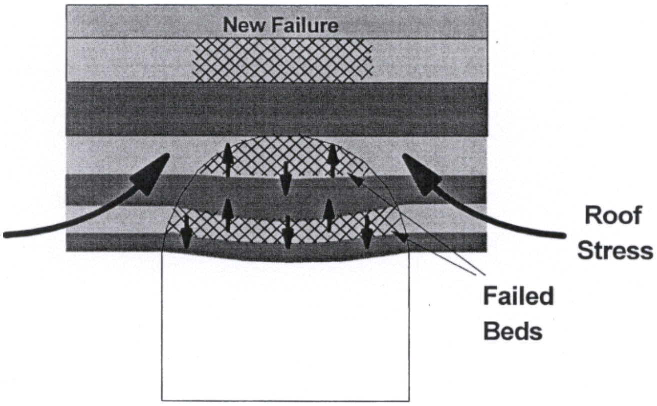


Figure 16.—Failure sequence in highly stressed roof (after Gale [1991]).

Frith proposes that the behavior of the second type of roof can be explained by the basic structural engineering concept of the Euler buckling beam (see previous section on "Installed Tension"). There have been a number of trials of high-tension fully grouted bolts in Australia, and the results are reported to be positive. Unfortunately, the field evidence that has been presented to date has been largely anecdotal (see, for example, Rataj et al. [1997]).

Numerical Modeling: As computers and software have grown more powerful, numerical modeling has become the standard design tool in many branches of engineering. Rock mechanics, however, has lagged behind. The reason is that rock engineers cannot specify the properties of the materials that they use, nor can they usually define the their loading conditions adequately.

For effective, quantitative design using numerical models, three basic prerequisites must be met [Hayes and Altounyan 1995; Gale and Fabjanczyk 1993]:

- *Model:* The model must be capable of replicating the behavior of coal measure rock, which means it must be able to simulate the various failure modes and large deformations which typically occur.
- *Material Properties and Stress:* Input rock mass properties must reflect both pre- and postfailure mechanics of the different roof layers encountered, and in situ stress levels must be measured in the field.
- *Validation:* To ensure that the model and the ground are behaving the same way, stresses and displacements must be measured. Important parameters include the magnitude and location of deformations, the distribution of bolt loads, and the behavior of interfaces at the top of the pillar and within the roof.

Numerical models used in the United States seldom meet any of these requirements. Stankus and Guo [1997] and Guo and Stankus [1997] describe a finite-element model that uses gap elements every 300 mm (1 ft), but otherwise assumes the rock to be homogeneous, elastic, and isotropic. The model looks for zero separation within or above the bolted range, which the study's authors cite as a weakness because bedding separations are commonly observed underground even where the roof is adequately supported [Stankus and Guo 1997]. The model's results are also extremely sensitive to the frictional strength coefficient [Guo and Stankus 1997]. The movements predicted by the model also seem quite small. In one instance cited by Stankus and Peng [1996], the total modeled roof deflection was <1 mm (0.032 in), and the longest bolt resulted in just 6% more deformation than the shortest.

Rigorous models that seem to meet all of the necessary requirements for quantitative design have been described overseas [Bigby 1997; Gale and Tarrant 1997]. Such models implement as many as seven rock failure modes, including bedding slip, shear failure of intact rock, tensile failure, and buckling. However, the expenses associated with such elaborate models, including the associated rock testing, stress measurement, and monitoring, are probably beyond customary U.S. practice. Moreover, the rapid changes in geology that often occur underground raise the question of the number of models and verification sites that might be needed.

Fortunately, numerical models can be very valuable tools even if there is not enough information to use them for quantitative design. As Starfield and Cundall [1988] pointed out, models can be used as controlled experiments to investigate the qualitative effects of different parameters. Well-designed model studies could be very helpful in moving the science of roof bolting forward.

ROOF MONITORING

Regardless of roof bolt design, failures are always possible. Often, an unstable area can be controlled with secondary support if the problem is detected in time. In the United States, instability is usually detected from visible and audible signals that become apparent shortly before collapse. Instruments are far more sensitive and can detect ground movements much earlier.

Routine monitoring of roof movements is much more common abroad. In the United Kingdom and Canada, two-point extensometers (often known as "telltales") are required every 20 m (65 ft) in bolted roadways and in all intersections (figure 17). The telltales have two movement indicators, one that shows displacement within the bolted height, and the other that shows movement above the bolts. Telltales are visible to everyone using the roadway, and their information can be recorded for later analysis [Altounyan et al. 1997].

The key to the effective use of monitoring is the determination of appropriate "action levels." For example, in gate roads at the Phalen Mine in Nova Scotia, Canada [McDonald and McPherson 1994]:

• *Spot bolting* when 25 mm (1 in) of movement is recorded either within or above the bolts.
• *Additional bolting and center props* when 50 mm (2 in) of movement is recorded.
• *Cable bolts* when 75 mm (3 in) of displacement is observed.

In the United Kingdom, typical action levels are 25 mm (1 in) within the bolted horizon and 10-25 mm (0.4-1 in) above [Kent et al. 1999a]. A survey of action levels in Australian mines, however, found no such uniformity. Some mines used total movement criteria; others used rates of movement ranging from 1 to 10 mm (0.04 to 0.4 in) per week [Mark 1998]. In the United States, the data are scarce, but action levels or "critical sag rates" have usually been about 5 mm (0.2 in) per week [Mark et al. 1994c].

In the United States, the lack of available personnel to install, read, and interpret roof monitors has always hindered their widespread use. However, preventing even a single roof fall in a critical belt or travel entry could justify the expense of a fairly extensive monitoring program. Hopefully, the time is not far away when computerized systems will help mines to make better use of roof monitors.

Often, roof monitoring can uncover a hidden geologic factor that can then be used directly in design. For example, a back analysis of monitoring data from the Selby coalfields in the United Kingdom found that excessive roof movements occurred where entries were unfavorably oriented relative to the horizontal stress or where the mudstone thickness exceeded 2.5 m (8 ft) [Kent et al. 1999b]. At the Plateau Mine in Utah, Maleki et al. [1987] found that excessive sag rates correlated with the presence of a channel sandstone within 1.5-2.2 m (5-7 ft) of the coal. A program of test holes helped locate the sandstone and reduced the number of sagmeters needed.

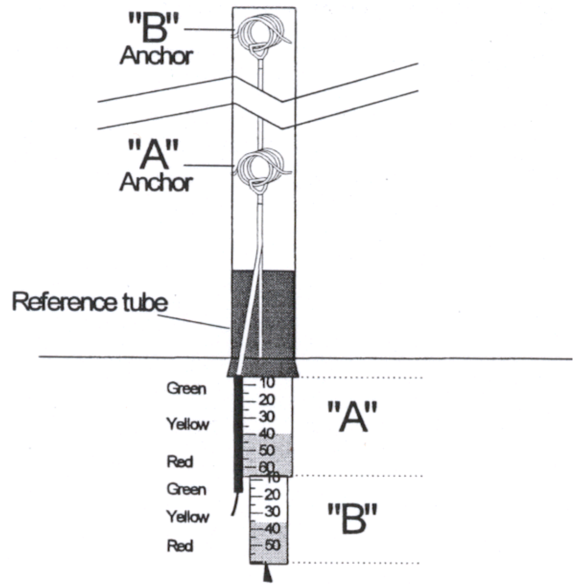


Figure 17.—A telltale (after Altounyan et al. [1997]).

GUIDELINES FOR ROOF BOLT DESIGN

Currently, there are no reliable methods for designing roof bolt systems. To begin to fill the mining community's need for better guidelines, NIOSH conducted a study of roof fall frequencies at 37 coal mines. The study's methodology, data collection procedures, and statistical analyses are reported by Molinda et al. [2000].

The study found that there was considerable scatter in the results, so that it was not possible to develop a universal design equation. In particular, it was not possible to determine the relative importance of individual rock bolt parameters including tension, length, capacity, and pattern.

However, some valuable relationships were found. It was not surprising that the geology, represented by the CMRR, was the most important variable. However, the next most important parameter was the depth of cover. With all else equal, deeper mines were more likely to have high roof fall rates. Horizontal stress could not be measured directly, but since it is known that the intensity of horizontal stress tends to increase with depth, the inference is that the depth of cover is a surrogate for the stress level. When the data were separated into a shallow cover group (<125 m (400 ft) and a deeper cover group (>125 m (400 ft), bolt design equations were determined for each.

Following are step-by-step guidelines:

1. *Evaluate the geology.* The CMRR should be determined either through underground observation or from exploratory drill core. Zones of markedly different CMRR should be delineated. If the thickness of individual beds varies within the bolted horizon, this effect should be noted. Special features, such as faults or major geologic transition zones, should be treated separately.

2. *Evaluate the stress level.* It is unusual for stress measurements to be available, so the design procedures use the depth of cover as a rough estimator. However, horizontal stress can sometimes be intensified by stream valleys or by driving in an unfavorable orientation. Roof support may need to be increased in these areas.

3. *Evaluate mining-induced stress.* Vertical, and sometimes horizontal, stresses may also be intensified by retreat mining or multiple seam interactions. These areas are likely to require supplemental support.

4. *Determine the intersection span.* An equation was derived from the data which suggests that the appropriate diagonal intersection span (I_s) is approximately:

$$(I_s) = 9.5 + (0.2 * \text{CMRR}) \ \text{(meters)} \tag{8a}$$

$$(I_s) = 31 + (0.66 * \text{CMRR}) \ \text{(feet)} \tag{8b}$$

If the CMRR > 65, it should be set equal to 65 in equation 8.

The intersection span can also be estimated from the entry width using table 4 where the typical spans are based on the field data:

Table 4.—Diagonal intersection spans (I_s)

Entry width, m (ft)	Ideal span, m (ft)	Typical diagonal intersection spans	
		Shallow cover, m (ft)	Deep cover, m (ft)
4.9 (16)	7.0 (23)	8.9 (29)	9.5 (31)
5.5 (18)	7.8 (25)	9.5 (31)	10.1 (33)
6.2 (20)	8.7 (28)	9.8 (32)	10.5 (34)

NOTE: The "ideal span" is determined by applying the Pythagorean theorem ($a^2 + b^2 = c^2$). "Typical" spans are based on actual measurements [Molinda et al. 2000].

As table 4 shows, the field data indicated that for the same entry width, spans at deep cover (depth > 130 m (400 ft)) exceeded the shallow cover spans by an average of 0.6 m (2 ft) due to pillar sloughing.

5. *Determine the bolt length.* Where the roof geology is such that the suspension mode is appropriate, the bolt length should be selected to give adequate anchorage in the strong rock. For the beam building mode, a bolt length formula was derived by modifying the Unal [1984] rock load height equation. The intersection span was substituted for the entry width, a depth factor was added, and then the constant was adjusted to fit the data:

$$L_B = 0.12\left(I_s\right)\log_{10}(3.25\,H)\left[\frac{100 - \text{CMRR}}{100}\right] \ \text{(meters)} \tag{9a}$$

$$L_B = 0.12\left(I_s\right)\log_{10}(H)\left[\frac{100 - \text{CMRR}}{100}\right] \ \text{(feet)} \tag{9b}$$

where (I_s) = diagonal intersection span (meters in equation 9a; feet in equation 9b); and

H = depth of cover (meters in equation 9a; feet in equation 9b).

These equations are illustrated in figure 18.

6. *Determine bolt pattern and capacity:* As has already been stated, the data could not determine which bolt parameter was most important. Therefore, the design variable is PRSUP, which includes both, plus the bolt length:

$$\text{PRSUP} = 29\,\frac{Lb * Nb * C}{Sb * We} \tag{10a}$$

$$\text{PRSUP} = \frac{Lb * Nb * C}{Sb * We} \tag{10b}$$

where Lb = length of the bolt (meters in equation 10a; feet in equation 10b);

Nb = number of bolts per row;

C = capacity (kilonewtons in equation 10a; kips in equation 10b);

Sb = spacing between rows of bolts (meters in equation 10a; feet in equation 10b); and

We = entry width (meters in equation 10a; feet in equation 10b).

Note that PRSUP differs from the PSUP used in past studies [Mark et al. 1994a] in that the bolt capacity has been substituted for the bolt diameter.

The suggested value of PRSUP for shallow cover is determined as:

$$\text{PRSUP} = 15.5 - 0.23\ \text{CMRR} \tag{11a}$$

and for deeper cover:

$$\text{PRSUP} = 17.8 - 0.23\ \text{CMRR} \tag{11b}$$

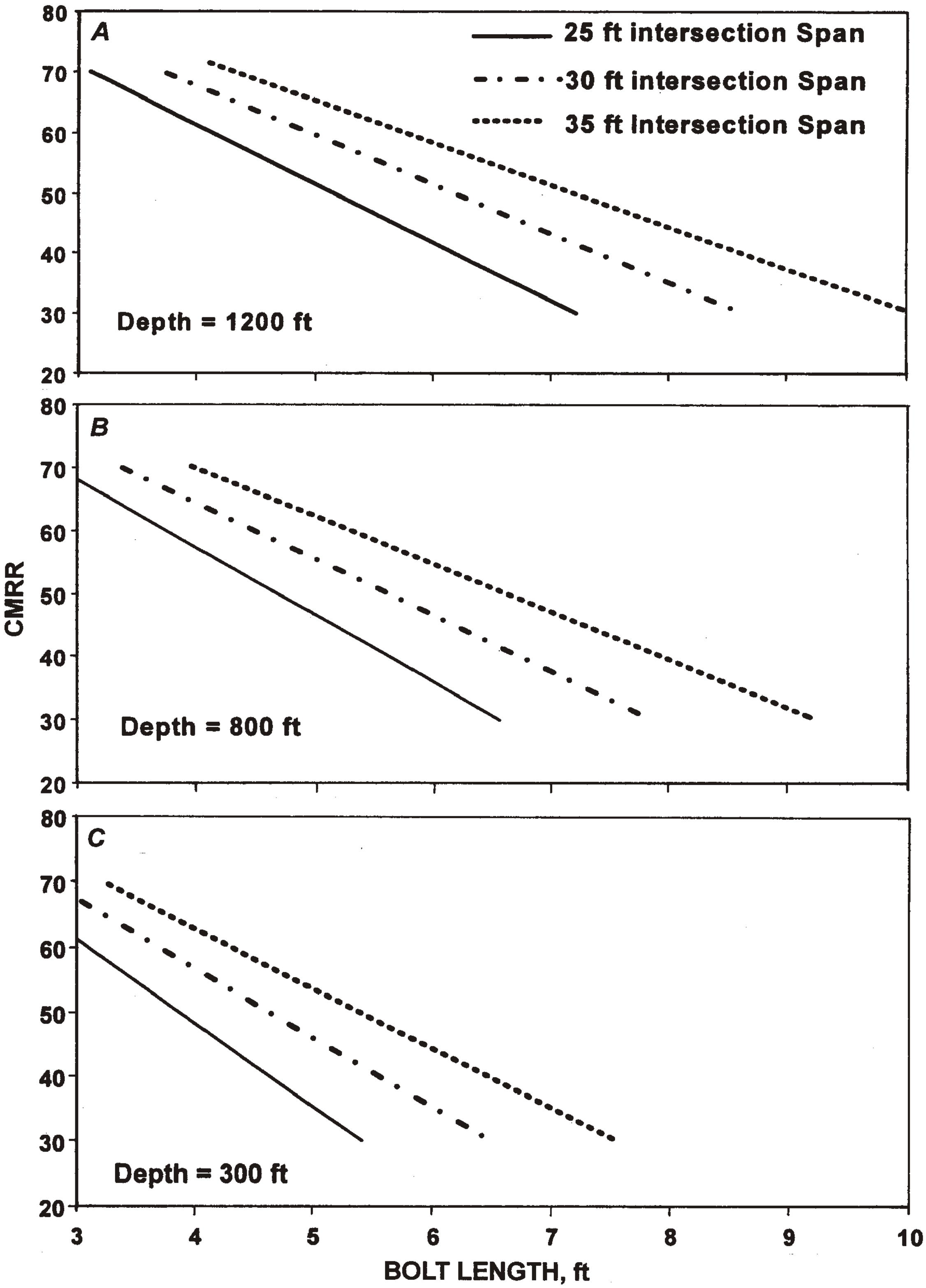


Figure 18.—Formula for selecting the bolt length. ***A*****, depth = 1,200 ft;** ***B*****, depth = 800 ft;** ***C*****, depth = 300 ft.**

Figure 19 shows these equations, together with the field data from which they were derived. The design equations are slightly more conservative than the discriminant equations on which they are based.

The field data also indicated that in very weak roof, it may be difficult to eliminate roof falls using typical U.S. roof bolt patterns. When the CMRR was <40 at shallow cover and <45-50 at deeper cover, high roof fall rates could be encountered, even with high roof bolt densities. Faced with these conditions, special mining plans, such as advance-and-relieve mining (Chase et al. [1999]), might be considered.

It should also be noted that these equations have been derived to reduce the risk of roof falls in intersections. In some circumstances, it may be possible to reduce the level of support between intersections.

Finally, the minimum recommended PRSUP is approximately 3.0.

7. *Select skin support:* Plates, header, mats, or mesh should be specified to ensure that loose rock between the bolts does not pose a hazard.

8. *Monitoring:* The installation of telltales or other simple extensometers should be considered for critical intersections so that, if it becomes necessary, supplemental support can be installed in a timely fashion.

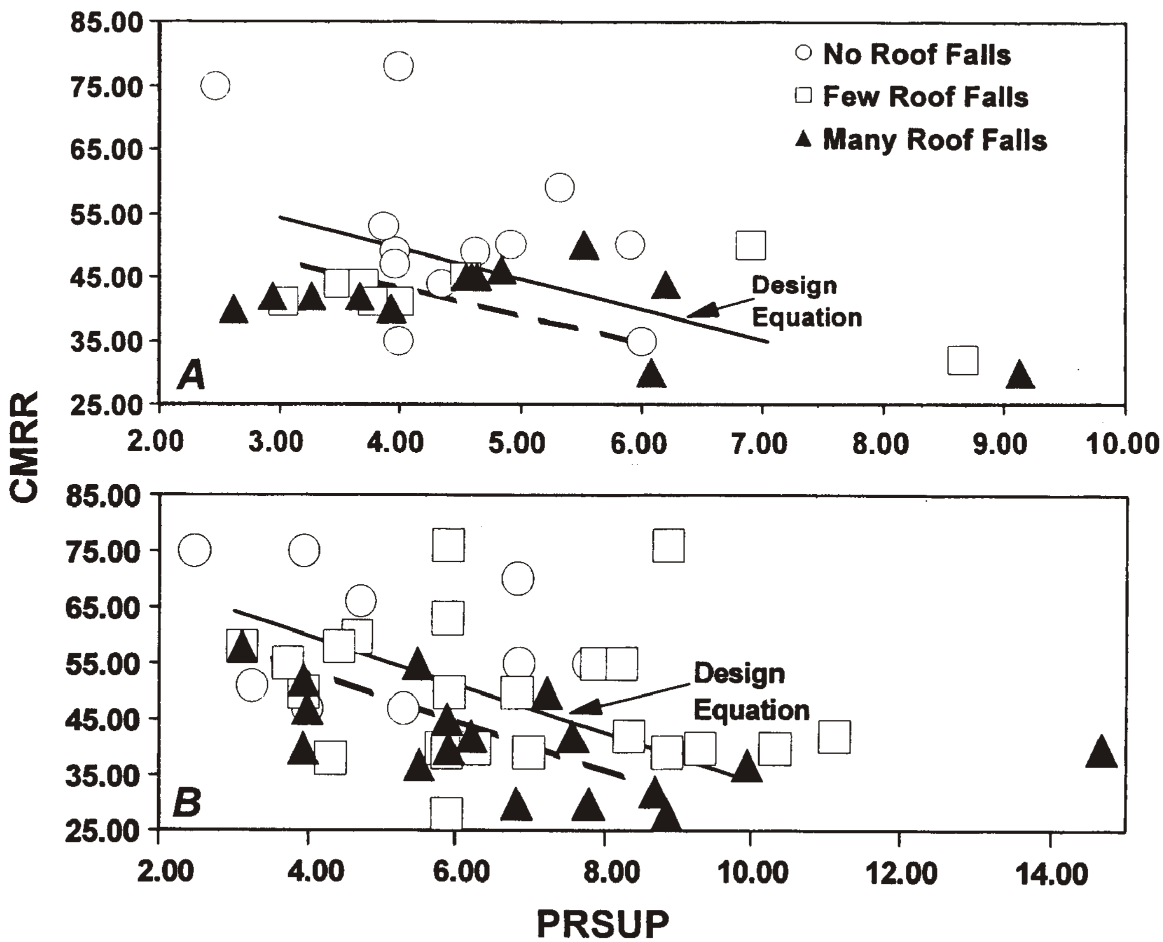


Figure 19.—Design equations for selecting bolt pattern and capacity. The field data used in the derivation of the formulas are shown, along with the original "discriminate equations" (dotted line). ***A*****, shallow cover (depth < 120 m (400 ft);** ***B*****, deep cover (depth > 120 m (400 ft)).**

CONCLUSIONS

1. Four support levels and reinforcement mechanisms are identified for roof bolts: *simple skin control, suspension, beam building*, and *supplemental support required*. The mechanism required for a particular application depends on the geology and the stress level.

2. The performance of fully grouted roof bolts can be determined by the load transfer effectiveness, which is indicated by the anchorage factor. Poor load transfer can reduce the effective capacity of the upper 300-600 mm (1-2 ft) of the bolt. Installations in weak rock are most at risk. Short encapsulation tests can be used to determine if the Anchorage Factor is adequate. Load transfer can be improved by optimizing the hole annulus, rifling or cleaning the hole, or roughening the bolt profile.

3. The importance of installed tension remains a subject of controversy. High tension is probably not necessary for simple skin control or suspension applications, but it may be helpful for beam building.

4. Increasing the bolt length can be effective in reducing the number of roof falls.

5. In weak roof, it is important that roof bolts be installed as soon as possible after the roof is exposed.

6. Effective skin control is an essential function of all roof support systems.

7. Proper installation is critical to the performance of roof bolting systems. Unfortunately, it is difficult to check the installation of fully grouted systems. Training and retraining of roof bolt crews is therefore essential.

8. Roof bolts may fail at the head, in the rod, or at the anchor. In addition, the system may fail if the rock breaks above it or if the support does not provide effective skin control.

9. Field measurements have shown that the loads on roof bolts commonly exceed the dead-weight loads by factors of two or more. Unfortunately, most of the other available empirical design approaches are qualitative at best.

10. Before numerical models can be used for design, they must—

- Be sophisticated enough to replicate complex rock mass behavior;
- Incorporate detailed rock property and in situ stress data; and
- Be validated by extensive field measurements.

Models used in the United States rarely meet these criteria.

11. Roof monitoring, particularly with two-point extensometers, could greatly improve our capacity to optimize the performance of roof bolt systems in the United States. However, such instruments will have to be computerized before they are widely accepted by the mines. A better understanding of the appropriate "action levels" for U.S. conditions will also be needed.

12. Guidelines are suggested for the preliminary design of roof bolt systems, based on analysis of field data collected from 37 U.S. coal mines. Formulas are provided that may be used to select appropriate intersection spans, bolt lengths, and bolt capacity/patterns. The formulas require a determination of the roof quality (using the CMRR) and the stress level (using the depth of cover). The equations should be used with caution, however, because the data used in their derivation were highly scattered.

13. The data also suggest that typical U.S. bolting systems may not always be capable of controlling roof falls in weak rock subjected to high stress.

14. Much more progress is needed before roof bolt design can truly be said to have advanced from an "art" to a "science."

REFERENCES

Altounyan PRF, Bigby DN, Hurt KG, Peake HV [1997]. Instrumentation and procedures for routine monitoring of reinforced mine roadways to prevent falls of ground. In: Dhar BB, Bhowmick BC, eds. Proceedings of the 27th International Conference of Safety in Mines Research Institutes. Vol. 2. New Delhi, India: Oxford & IBH Publishing Co. Pvt. Ltd., pp. 759-766.

Aziz N, Indraratna B, Dey A, Wang Y [1999]. Laboratory study of shear loading and bolt load transfer mechanisms under constant normal stiffness conditions. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 239-247.

Bauer ER, Dolinar DR [2000]. Skin failure of roof and rib and support technology in underground coal mines. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Bieniawski ZT [1987]. Strata control in mineral engineering. New York, NY: John Wiley & Sons, Inc.

Bigby DN [1997]. Developments in British rock bolting technology. Coal International, Rockbolting Technology, May, pp. 111-114.

British Coal Technical Department [1992]. Roofbolting. Min Technol *Nov/Dec*:296-300.

Campoli AA, Mills PA, Todd P, Dever K [1999]. Resin annulus size effects on rebar bolt pull strength and resin loss to fractured rock. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 222-231.

Chase FE, Mark C, Mucho TP [1999]. The advance and relieve mining method: a horizontal stress control technique. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 300-308.

Cincilla WA [1986]. Determination of effective column lengths for resin-grouted roof bolts. In: Peng SS, eds. Proceedings of the Fifth International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 6-14.

Conway CC [1948]. Roof support with suspension rods. Min Cong J *June*: 32-37.

Damberger HH, Nelson WJ, Krausse HF [1980]. Effect of geology on roof stability in room-and-pillar mines in the Herrin (No. 6) coal of Illinois. In: Proceedings of the First Conference on Ground Control Problems in the Illinois Coal Basin. Carbondale, IL: Southern Illinois University, pp. 14-32.

Dolinar DR, Bhatt SK [2000]. Trends in roof bolt application. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Dunham RK [1976]. Anchorage tests on strain-gauged resin bonded bolts. Tunnels and Tunnelling *8*(6):73-76.

Fabjanczyk MW, Tarrant GC [1992]. Load transfer mechanisms in reinforcing tendons. In: Aziz NI, Peng SS, eds. Proceedings of the 11th International Conference on Ground Control in Mining. Wollongong, New South Wales, Australia: University of Wollongong, pp. 212-219.

Fairhurst C, Singh B [1974]. Roof bolting in horizontally laminated rock. Eng Min J *Feb*:80-90.

Franklin JA, Woodfield PR [1971]. Comparison of a polyester resin and a mechanical rockbolt anchor. Trans Inst Min Metall (section A: Mining Industry), pp. A91-100.

Frith R [1998]. Strata engineering summary. Australian Coal Association Research Program (ACARP) Technical Seminar Course Notes.

Frith R, Thomas R [1998]. The pretensioning pedigree. Australia's Mining Monthly Supplement-Underground Equipment and Technology *June*:68-72.

Fuller PG [1999]. Keynote lecture: roof strata reinforcement–achievements and challenges. In: Proceedings of the Rock Support and Reinforcement Practice in Mining. Balkema, pp. 405-415.

Gale WJ [1991]. Strata control utilizing rock reinforcement techniques and stress control methods, in Australian coal mines. The Mining Engineer *Jan*:247-253

Gale W, Fabjanczyk MW [1993]. Design approach to assess coal mine roadway stability and support requirements. In: Proceedings of the Eighth Australian Tunnelling Conference (Sydney, Australia).

Gale WJ, Tarrant GC [1997]. Let the rocks tell us. Proceedings of the Symposium on Safety in Mines, The Role of Geology 24-25 *Nov*:153-160.

Gale WJ, Fabjanczyk MW, Guy RJ [1992]. Optimization of reinforcement design of coal mine roadways. In: Aziz NI, Peng SS, eds. Proceedings of the 11th International Conference on Ground Control in Mining. Wollongong, New South Wales, Australia: University of Wollongong, pp. 272-279.

Gerdeen JC, Snyder VW, Viegelahn GL, Parker J [1979]. Design criteria for roof bolting plans using fully resin-grouted nontensioned bolts to reinforce bedded mine roof. Volume III: Experimental model studies; Volume IV: Theoretical analysis. Houghton, MI: Michigan Technological University. U.S. Bureau of Mines contract No. J0366004.

Gray P, Finlow-Bates P [1998]. The pre-tensioned placebo. Australia's Longwalls Magazine *Mar*:78-81.

Guo S, Stankus JC [1997]. Control mechanism of a tensioned bolt system in the laminated roof with a large horizontal stress. In: Peng SS, ed. Proceedings of the 16th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 167-172.

Haas CJ [1981]. Analysis of rockbolting to prevent shear movement in fractured ground. Min Eng *Jun*:691-704.

Hayes AW, Altounyan PFR [1995]. Strata control the state-of-the-art. Mining Technology, Institute of Mining Engineers *77*(892):354-358.

Health and Safety Executive [1996]. Guidance on the use of rockbolts to support roadways in coal mines. U.K.: Health and Safety Executive, Deep Mines Coal Industry Advisory Committee.

Hurt K [1992]. Rockbolting design in UK coal mines. In: Proceedings of Eurock 92, paper No. 71. London, U.K.: Thomas Telford, pp. 414-419.

Karabin GJ, Debevec WL [1976]. Comparative evaluation of conventional and resin bolting systems. U.S. Department of the Interior, Mine Enforcement and Safety Administration, IR 1033.

Karabin GJ, Hoch MT [1980]. Contemporary roof support systems provide a diversified approach to modern ground control. VPI Annual Institute on Coal Mine Safety and Health. Blacksburg, VA: Virginia Polytechnic Institute and State University, pp. 249-267.

Kent L, Cartwright PB, Bigby DN [1999a]. Tunnel deformation monitoring "action levels" in coal mines for support/reinforcement design. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 98-107.

Kent FL, Coggan JS, Altounyan PFR [1999b] Investigation into factors affecting roadway deformation in the Selby coalfield. In: Proceedings of the Geotechnical and Geological Engineering, pp. 273-289.

Lang TA, Bischoff JA [1982]. Stabilization of rock excavations using rock reinforcement. In: Proceedings of the 23th U.S. Rock Mechanics Symposium, pp. 935-944.

MacDonald B, MacPherson K Sr. [1994]. Roofbolting in the Cape Breton Development Corporation's Phalen Colliery. In: Peng SS, ed. Proceedings of the 13th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 32-41.

Maleki H [1992]. Significance of bolt tension in ground control. Proceedings of the Rock Support in Mining and Underground Construction. Balkema, pp. 439-449.

Maleki H, Carlisle T, Hunt G, Agapito JFT [1987]. A novel ground control program at Plateau Mining Company. In: Peng SS, ed. Proceedings of the Sixth International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 118-125.

Maleki H, Signer SP, King M, Edminster P [1994]. Evaluation of support performance in a highly stressed mine. In: Peng SS, ed. Proceedings of the 13th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 9-17.

Mark C [1982]. A physical model study of coal mine roof using the base friction technique [Thesis]. University Park, PA: The Pennsylvania State University.

Mark C [1998]. Comparison of ground conditions and ground control practices in the United States and Australia. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 63-71.

Mark C [1999]. Application of coal mine roof rating (CMRR) to extended cuts. Min Eng *51*(4):52-56.

Mark C, Barczak TM [2000]. Fundamentals of coal mine roof support. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Mark C, Chase FE, Molinda GM [1994a]. Design of longwall gate entry systems using roof classification. In: Proceedings—New Technology for Longwall Ground Control Technology Transfer. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, SP 94-01, pp. 5-18.

Mark C, Molinda GM, Schissler AP, Wuest WJ [1994b]. Evaluating roof control in underground coal mines with the coal mine roof rating. In: Peng SS, ed. Proceedings of the 13th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 252-260.

Mark C, Watts R, Marshall TE, Serbousek MO, Carr F [1994c]. Using computerized mine-wide monitoring for ground control. SME preprint 94-168. Littleton, CO: Society for Mining, Metallurgy, and Exploration, Inc.

Mark C, Dolinar DR, Mucho TP [2000]. Summary of field measurements of roof bolt performance. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Martens PN, Rattmann L [1998]. Developments in rockbolting. J Mines Metals Fuels, Min Ind Ann Rev, pp. 405-411.

Mazzoni, RA, Karabin GJ, Cybulski JA [1996] A trouble-shooting guide for roof support systems. Pittsburgh, PA: U.S. Department of Labor, Mine Safety and Health Administration, IR 1237.

McHugh E, Signer SP [1999]. Roof bolt response to shear stress: laboratory analysis. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 232-238.

Molinda GM, Mark C [1994]. Coal Mine Roof Rating (CMRR): A Practical rock mass rating for coal mines. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9387.

Molinda GM, Mark C, Bauer ER, Babich DR, Pappas DM [1998]. Factors influencing intersection stability in U.S. coal mines. In: Peng SS, ed.

Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 267-275.

Molinda GM, Mark C, Dolinar DR [2000]. Assessing coal mine roof stability through roof fall analysis. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Obert L, Duvall WI [1967]. Rock mechanics and the design of structures in rock. New York, NY: John Wiley & Sons, Inc.

Panek LA [1964]. Design for bolting stratified roof. Trans SME, 229. Littleton, CO: Society for Mining, Metallurgy, and Exploration, Inc., pp. 113-119.

Peng SS [1998]. Roof bolting adds stability to weak strata. Coal Age Magazine *Dec*:32-38.

Peng SS, Guo LB [1989]. Design criteria for resin bolting. SME preprint 89-166. Littleton, CO: Society for Mining, Metallurgy, and Exploration, Inc.

Rataj M, Arnall A, Thomas R [1997]. New methods and technologies of roofbolting in Australia coal mines. In: Peng SS, ed. Proceedings of the 16th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 149-157.

Rico GH, Orea RR, Mendoza RL, Tadolini SC [1997]. Implementation and evaluation of roof bolting in MICARE Mine II. In: Peng SS, ed. Proceedings of the 16th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 139-148.

Scott JJ [1992]. Roof bolting in mining–USA. In: Proceedings of the Norsk Jord-og Fjellteknisk Forbund. Oslo, Norway, November 27.

Serbousek MO, Signer SP [1987]. Linear load-transfer mechanics of fully grouted roof bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9135.

Siddall RG, Gale WJ [1992]. Strata control–a new science for an old problem. The Mining Engineer *151*(369):342-355.

Signer SP [1990]. Field verification of load transfer mechanics of fully grouted roof bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9301.

Signer SP [2000]. Load behavior of grouted bolts in sedimentary rock. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Signer SP, Jones SD [1990]. A case study of grouted roof bolt loading in a two-entry gateroad. In: Peng SS, ed. Proceedings of the Ninth International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 35-41.

Signer SP, Lewis JL [1998]. A case study of bolt performance in a two-entry gateroad. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 249-256.

Signer SP, Mark C, Franklin G, Hendon G [1993]. Comparisons of active versus passive bolts in a bedded mine roof. In: Peng SS, ed. Proceedings of the 12th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 16-23.

Stankus JC [1991]. Tensioned point anchor resin system versus nontensioned fully grouted rebar–two case studies. In: Peng SS, ed. Proceedings of the 10th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 65-76.

Stankus JC, Guo S [1997]. New design criteria for roof bolt systems. In: Peng SS, ed. Proceedings of the 16th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 158-166.

Stankus JC, Peng SS [1996]. A new concept for roof support. Reprint from Coal Age Magazine, September.

Starfield AM, Cundall PA [1988]. Towards a methodology for rock mechanics modelling. Int J Rock Mech Min Sci *25*(3):99-106.

Tadolini SC [1998]. The effects of reduced annulus in roof bolting performance. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 230-236.

Tadolini SC, Dolinar DR [1991]. Thrust bolting: a new innovation in coal mine roof support. In: Peng SS, ed. Proceedings of the 10th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 76-84.

Tadolini SC, Ulrich BF [1986]. Evaluation of bearing plates installed on full-column resin-grouted bolts. U.S. Department of the Interior, Bureau of Mines, RI 9044.

Terrill L, Francke C [1995]. Analysis of rockbolt performance at the Waste Isolation Pilot Plant. In: Peng SS, ed. Proceedings of the 14th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 182-188.

Terzaghi K [1946]. Rock tunnelling with steel support. Procter RV, White T, eds. Youngstown, OH: Commercial Shearing Co.

Thomas E [1954]. Rock bolting finds wide application. Min Eng *Nov.*:1080-1085.

Thomas B, Wilson G [1999]. Control technology for roof drill operators. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 216-221.

Ulrich BF, Wuest J, Stateham RM [1989]. Relationships between annulus thickness and the integrity of resin grouted bolts. Denver, CO: U.S. Department of the Interior, Bureau of Mines, RI 9253.

Unal E [1984]. Empirical approach to calculate rock loads in coal mine roadways. In: Peng SS, ed. Proceedings of the Fifth International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 232-246.

VandeKraats JD, Watson SO [1996]. Direct laboratory tensile testing of select yielding rock bolt systems. In: Ozdemir L, Hanna K, Haramy KY, Peng SS, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 321-332.

VandeKraats JD, Watson SO [1998]. Laboratory and in situ results of a slip nut yielding rock bolt system. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 237-242.

Wright FD [1973]. Roof control through beam action and arching. In: Given IA, ed. SME Mining Engineering Handbook *1*(13). Littleton, CO: Society for Mining, Metallurgy, and Exploration, Inc., p. 80.

Yearby M [1991]. Practical guide to rock bolting, ANI Arnall. Newcastle, New South Wales, Australia.

DESIGN METHODOLOGY FOR STANDING SECONDARY ROOF SUPPORT SYSTEMS

By Thomas M. Barczak,[1] Thomas P. Mucho,[2] and Dennis R. Dolinar[3]

ABSTRACT

Maintaining ground stability in the gate roads, particularly the tailgate, has always been critical to the success of longwall mining, both in terms of safety and productivity. Several new support technologies have been developed in recent years to replace conventional wood and concrete cribbing for secondary roof support. Since their performance characteristics are unique, the best practices that have been developed with conventional wood cribbing may not be applicable for these alternative support technologies. Therefore, with so many options to consider and the importance of achieving adequate ground control at minimal cost, the trial-and-error approach to longwall gate road support is no longer prudent. This paper discusses a design methodology for standing secondary tailgate supports. This design technique requires in-mine measurements of tailgate support loading and convergence to establish a tailgate ground reaction behavior based on support and strata interaction. The methodology uses the performance characteristics generated in the National Institute of Occupational Safety and Health's (NIOSH) Mine Roof Simulator (MRS) to match the stiffness and load characteristics of various supports to the measured ground reaction behavior. It can be used to determine the appropriate application of alternative roof support systems or to design in-mine trials so that a fair and equitable comparison of different support systems can be made. A case study of the methodology at a western Pennsylvania mine site is presented in the paper, including a comparison of four alternative support technologies to the conventional wood and concrete cribbing historically used at this particular mine.

[1]Research physicist.
[2]Branch chief, Disaster Prevention and Response.
[3]Mining engineer.
Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

The longwall tailgate support system must flawlessly control the tailgate ground conditions. Safety considerations, especially the limited escape routes from a longwall face, demand that the tailgate entry be a negotiable travelway. The location of the face electrical systems, support equipment, and belt line in the head gate entries dictate that the tailgate may be the only option for mine workers to escape from the face in the event of an emergency. A recent example is the longwall gob fire that occurred at a mine in Utah in 1998 in which several miners evacuated through the tailgate entry to safety. In addition to the emergency travelway requirements, inadequate tailgate support that results in poor ground control and blocked tailgates due to roof falls can severely retard or halt production. The heavy reliance by mines on the longwall production for survival dictates that loss of production for protracted time periods cannot be tolerated. Ventilation is another issue that depends on proper tailgate support. As the panel lengths continue to increase, excessive closure or restriction of the tailgate entry by deformation and/or density of the standing support can be problematic and potentially unacceptable. In gassy mines, it also may be required that the tailgate be kept open inby the longwall face in order to establish effective bleeder ventilation of the tailgate area. Another important issue to consider is the material handling aspects of tailgate supports. Therefore, the onus is on mine engineers to design a support system that maintains adequate control of the tailgate ground conditions at all times and with minimal ventilation resistance and material handling considerations.

Historically, the importance of ground control has led to very conservative applications of tailgate support. Most mines use conventional wood crib structures. When properly designed, conventional wood cribs provide effective ground control in most longwall tailgate entries and in the past have been cheap enough that mines could afford to use a high density of cribs at relatively little cost. However, increasing timber costs, inconsistent timber quality that has led to poor crib performance, and inadequate supplies of timber for Western mines have reduced the advantages of conventional wood cribbing and have encouraged many mines to consider other options for tailgate support. All of these factors have prompted support manufacturers to develop innovative support technologies as alternatives to conventional wood cribbing.

Today, there are several alternative support technologies that have been developed by various support manufacturers and tested for safety and performance characteristics at the National Institute for Occupational Safety and Health's (NIOSH) Safety Structures Testing Laboratory. With increasing pressures to reduce support costs while ensuring the safety of the mine workers, mine operators more than ever before are looking for ways to optimize longwall tailgate support. The performance characteristics of these supports are unique, so the best practices that have been developed for conventional wood cribbing largely through trial and error may not be applicable for these alternative support technologies. In addition, these new supports have limitations, which, if not properly recognized, can lead to poor installation and inadequate tailgate ground control.

Therefore, with so many options to consider and the importance of achieving proper ground control at minimal cost, the trial- and- error approach to longwall tailgate support can be costly and indecisive. This paper proposes a design philosophy for longwall tailgate secondary support whereby the ground reaction behavior, as a function of support load density and stiffness of the support system, is determined by measurements of underground support loading and roof-to-floor convergence. The goal of support design is then to optimize the use of the support by designing to the ground reaction curve and controlling convergence to acceptable limits that will ensure stability of the mine roof. This approach will allow mine operators to maximize the use of alternative support systems while ensuring the safety of mine workers by avoiding risky and time-consuming trial-and-error assessments of support technologies. In addition, it will provide Mine Safety and Health Administration (MSHA) with a means to assess various support systems on an equivalent basis when approving roof control plans. A case study is included in the paper that relates the use of the proposed design methodology to a trial of alternative support technologies at a western Pennsylvania longwall mine operating in the Pittsburgh coal seam.

TERMINOLOGY

Secondary Support - Secondary support is support that is intentionally added to assist the primary support (roof bolts) in controlling the mine roof when it is known that additional roof loading will occur. In longwall mining, secondary support is installed in advance of abutment loading. Secondary support is not to be confused with supplemental support, which is support installed in addition to primary and secondary support either for insurance purposes or in response to unanticipated poor ground conditions.

Ground Reaction Curve - A concept of how the ground reacts to the presence of a newly created opening. Specifically, as the ground deforms and sheds load to other structures, there will be a proportional decrease in roof loading and required support capacity to maintain equilibrium of the mine roof and floor.

Critical Convergence - In relation to the ground reaction curve, critical convergence is the point where failure of the ground is inevitable, and the full weight of the failed rock mass above the mine entry must be supported by the secondary roof system to prevent a roof fall. The goal of secondary support design is to prevent this convergence.

Support Load Density - The load-carrying capacity of an installed support system per unit area of exposed ground ($tons/ft^2$) at a particular amount of ground deformation.

Minimal Acceptable Support Load Density - Lowest load density of support that should be provided. A lower support load density would allow convergence greater than critical convergence and thereby allow failure of the roof rock that may lead to a roof fall.

Support Density - Term typically used to refer to the number of supports per unit area. Support density should not be confused with support load density, which is the capacity of the support system per unit area as a function of convergence.

Support Stiffness - A measure of how quickly a support develops its load capacity in relation to convergence. For an individual support, stiffness can be determined from the slope of the load-deformation performance curve. "Softer" supports have a flatter slope than "stiffer" supports when plotted to the same load-displacement scale. Softer supports require more convergence to develop an equivalent load-carrying capacity than stiffer supports.

Support System Stiffness - The resistance to load of a group of supports. System stiffness is the sum of the stiffnesses of individual supports. Hence, a double row of supports would have twice the system stiffness of a single row of the same type of supports.

KEY FACTORS TO CONSIDER IN GATE ROAD SUPPORT

A design philosophy for standing supports must be based on the interaction of the support with the surrounding rock mass. The question that needs to be addressed to formulate a design methodology is to determine to what extent the support system is controlling the ground. To do this, it is necessary to understand both ground behavior and the characteristics of both the individual support and the support system.

UNDERSTANDING THE FUNCTION OF THE SUPPORT SYSTEM

Obviously, the support system is employed to prevent roof falls. How this is accomplished is the important issue. While secondary supports provide the last means of support in the event there is roof failure above the bolted horizon, the primary function of the secondary support system is to assist the primary support system in maintaining the integrity of the immediate roof. As the ground deforms by the creation of an opening during mining, it gradually sheds load to the surrounding mine structures, which, in the case of longwall mining, are the gate road pillars and the longwall panel. Secondary support must be placed in sufficient time and develop sufficient capacity to bring deformation of the ground into equilibrium before a critical deformation is reached, at which point failure of the ground is inevitable. Otherwise, the secondary support will be required to carry the entire dead weight of the detached rock mass to prevent a roof fall. This embodies a fundamental concept in rock mechanics known as the "ground reaction curve"[Deere et al. 1970].

In longwall mining, the tailgate entry is subjected to three phases of loading and equilibrium. Each will have a distinct ground reaction curve. The first phase occurs on development where the mine opening is created and the primary support (roof bolts) is installed. Relatively little ground movement takes place during this phase since the development loads are small and the primary support is sufficient to provide equilibrium. The next phase is adjacent panel mining. The future tailgate is subjected to side abutment loading, and while secondary support is typically installed to ensure that equilibrium of the rock mass is obtained, the convergence is typically minimized by the load density of the support.

The final phase of tailgate behavior is where the active tailgate is subjected to front abutment loading from panel extraction. It is this phase where the secondary supports play their most important role in preserving the stability of an entry. A hypothetical tailgate ground reaction curve is shown in figure 1. It should be noted that the ground reaction curve will be a function of several factors in addition to the load density of the support system. These include geology, roof spans, vertical and horizontal stress around the opening, and some time-dependent factors such as creep. Hence, the ground reaction curve is generally unique to a specific mine and can change within the mine as these factors change. From the perspective of secondary support design, it is important that the ground reaction curve be examined under worst-case load conditions where ground control is required. Since ground reaction is dependent upon roof span, a different ground reaction behavior will typically be observed in intersections as compared to the nonintersection areas of the entry. Hence, the support design must be altered for the intersections to accommodate this difference in ground reaction.

It is seen from the hypothetical tailgate ground reaction curve (figure 1) that if the goal is to prevent convergence completely, then the full abutment load must be resisted by the

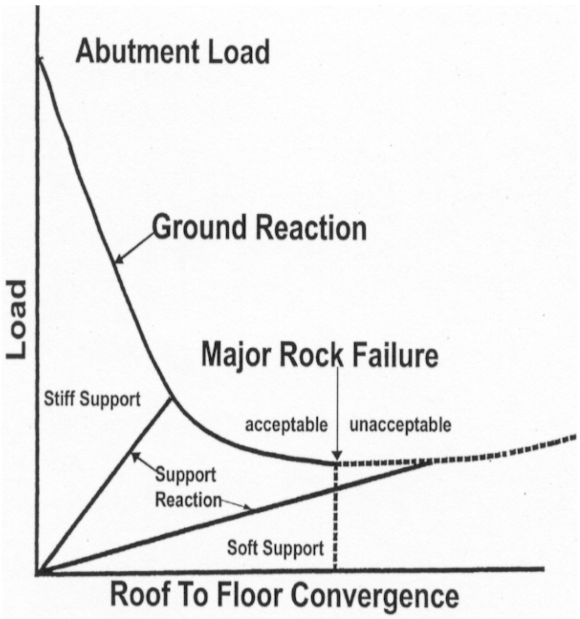


Figure1.–Ground reaction curve concept.

support system. For all practical purposes, this resistance cannot be obtained by installation of secondary support since the required capacity would need to be equivalent to that of all the coal removed by the mining process. The support capacity required to achieve equilibrium is reduced as deformation increases, since the roof is shedding load to other mine structures as it deforms. In other words, by allowing the roof to deform and shed some load to the coal pillars and longwall panel, less support capacity is required because the roof load is decreased. Hence, the lowest required support capacity would be one that is developed just before critical roof deformation reaches the point where failure of the immediate roof is fast approaching. However, designing to this lower limit of support capacity leaves no margin of error in the event that load conditions worsen. Also, it can be seen that if a support system is too soft (develops load-carrying capacity too slowly), equilibrium of the mine roof will never be achieved and failure of the roof will be inevitable.

In summary, since most standing supports are passive supports, convergence must take place before sufficient support loads are developed to provide equilibrium. While some supports have active loading capability, the magnitude of active loading is not sufficient to achieve equilibrium or the active loading cannot be maintained indefinitely. The supports must be loaded in compression to provide the required load-resisting forces to achieve equilibrium of the rock mass. The amount of convergence required to produce equilibrium is then a function of the stiffness of the support system. Equilibrium will be achieved at less displacement for a stiffer support system because in a stiff support system, resistance to roof loading will develop quicker (at less displacement) than in a softer support system. Hence, as the support load resistance (load density) of a support system increases, the convergence at which equilibrium is attained will decrease. If too much convergence is permitted through use of too soft a support system, failure of the rock mass (mine roof) will be inevitable. Hence, the goal of support design is to provide sufficient support stiffness to ensure that the required support capacity to achieve equilibrium of the rock mass occurs before the rock mass deforms to the point of failure. However, a prudent mine engineer would ensure that sufficient support capacity is developed long before critical convergence is reached. Since minimizing convergence is achieved by increasing the support capacity (load density) (and generally the cost of support), the goal of optimizing support selection is not to install more support than is necessary to provide a reasonable margin of safety to prevent roof failure.

UNDERSTANDING THE PERFORMANCE CHARACTERISTICS OF THE SUPPORT

The load-displacement characteristics of numerous roof support technologies have been determined at the Safety Structures Testing Laboratory through full-scale tests in the unique Mine Roof Simulator (MRS) load frame [Barczak 1994]. The load-displacement response of these various support systems are documented in figures 2 through 7, grouped by the following description of the support type: (1) conventional wood cribbing (figure 2), (2) engineered wood crib supports (figure 3), (3) conventional and engineered timber post supports (figure 4), (4) nonyielding concrete supports (figure 5), (5) deformable concrete supports (figure 6), and (6) yielding steel supports (figure 7).

As previously indicated, all secondary supports must be loaded in compression to produce the required capacity to achieve equilibrium of the mine roof. In other words, the roof has to move down before the standing support develops sufficient load-carrying capacity to achieve equilibrium of the mine roof and floor. Since it is this very downward movement of the roof that we are trying to control, the most important design parameter for standing supports is the stiffness of the support system. Stiffness is simply a measure of how quickly a support develops its load-carrying capability in response to convergence of the mine roof and floor. Stiffer supports develop equivalent load carrying capacity with less displacement than softer support systems.

While the stiffness of the support is the primary design parameter, it is not the only parameter to consider in support application. Another important design parameter is the load-carrying stability of the support. More specifically, it is important to know how well the support can sustain its load-carrying capability as a function of convergence. Stiff supports, such as the nonyielding concrete support (figure 5), which develop load-carrying capacity quickly, but fail at little convergence, are not practical in many longwall tailgate applications. To keep such supports from failing prematurely, a large number of supports must be installed so that roof loading is sufficiently shared among several supports while achieving

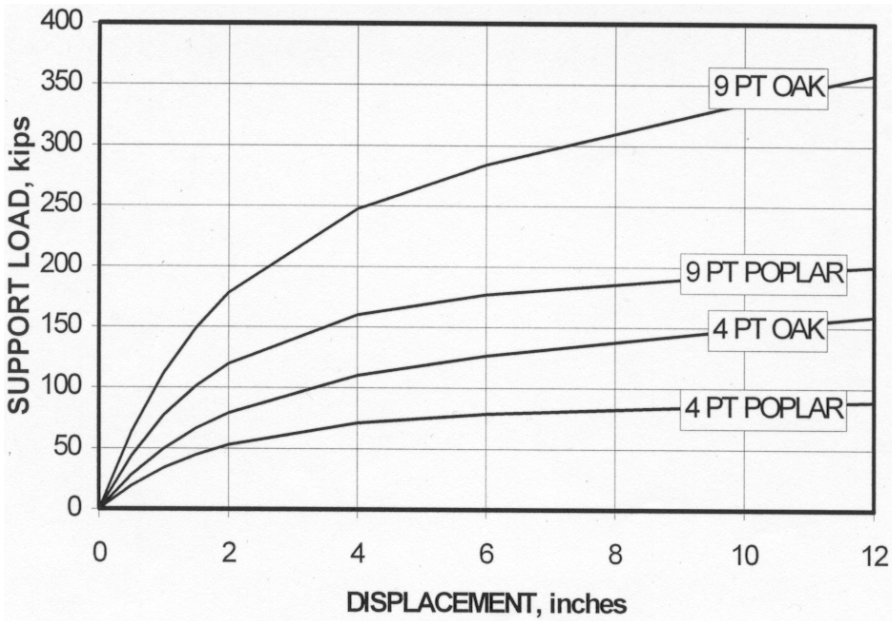


Figure 2.–Load-displacement performance data for conventional wood.

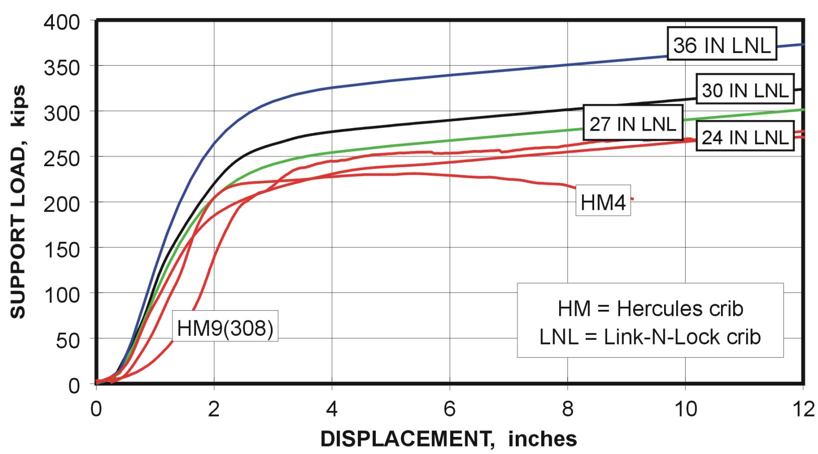


Figure 3.–Load displacement performance data for engineered wood crib supports.

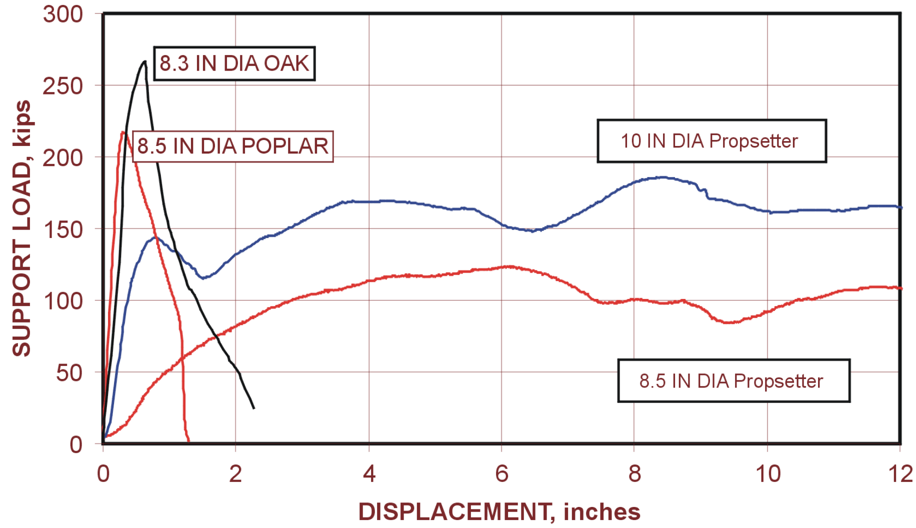


Figure 4.–Load-displacement performance data for conventional and engineered timber props.

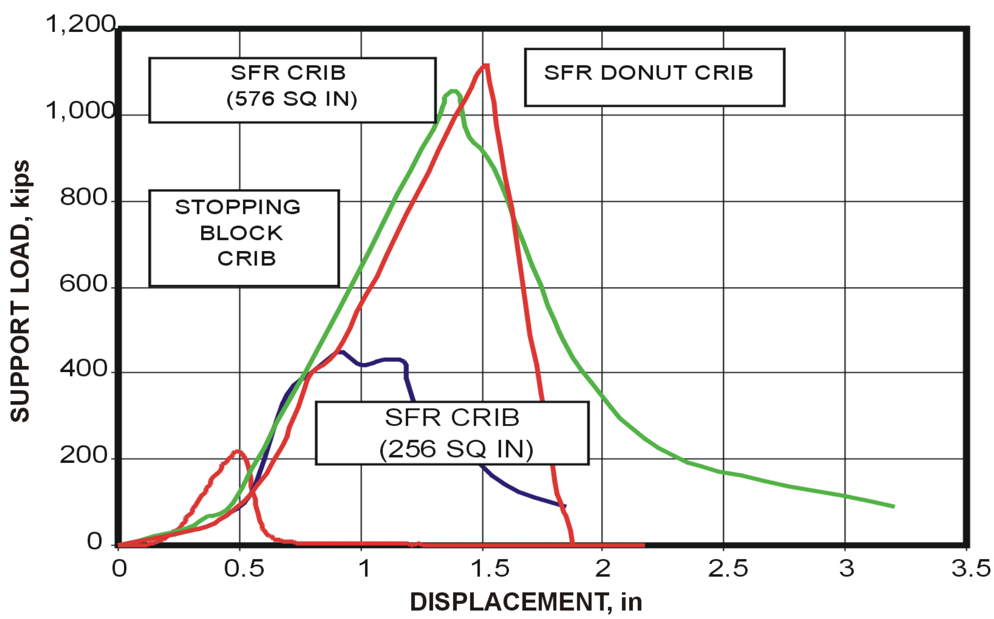


Figure 5.–Load-displacement performance data for nonyielding concrete support.

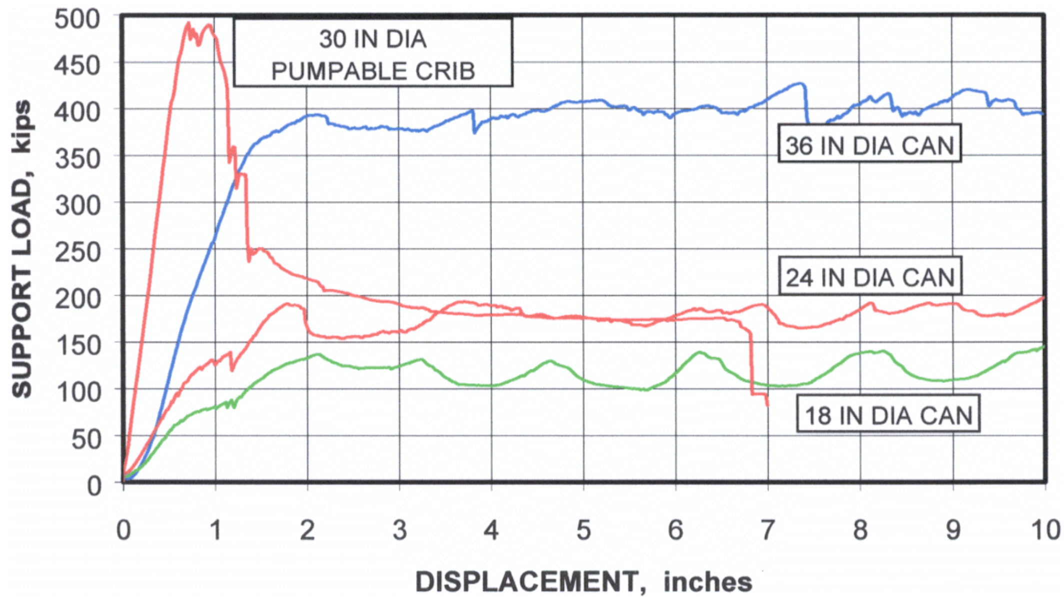


Figure 6.–Load-displacement data for deformable concrete supports.

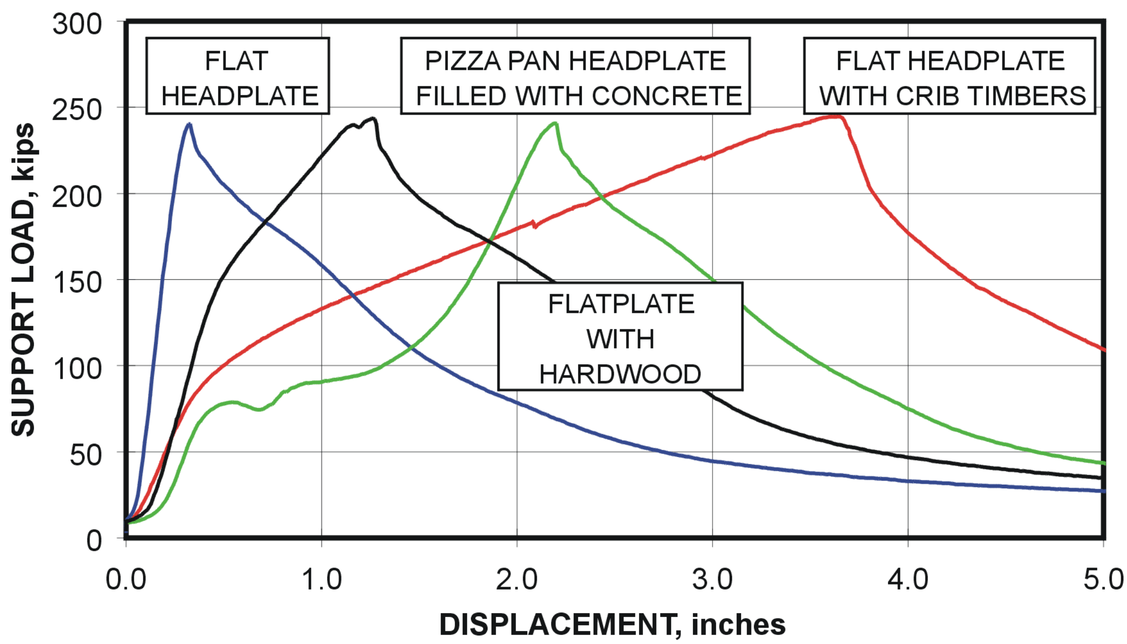


Figure 7.–Load-displacement performance data for yielding steel supports.

high enough load density to keep convergence below the failure point for any one support.

The ideal support is one in which stiffness can be controlled through the support design so that the support capacity at a given displacement can be engineered to match ground reaction behavior. Furthermore, the ideal support would be able to maintain this capacity through a wide range of displacements without shedding load or failing prematurely and therefore would provide a margin of safety in the event that ground conditions worsen unexpectedly.

MEASUREMENT OF THE GROUND REACTION CURVE AT A WESTERN PENNSYLVANIA MINE

Studies were made at a longwall mine operating in the Pittsburgh coal seam in western Pennsylvania. The mine has historically used a staggered double row of four-point wood cribs on 8-ft spacings (figure 8) for longwall tailgate support. Due to uncertainties and inconsistent timber qualities, the mine also employed concrete cribs constructed from normal mine ventilation stopping blocks. These were also arranged in a staggered double row, as shown in figure 9. Since the stiffness of these two supports are significantly different (figures 2 and 5), and the supports were employed in a similar arrangement, a ground reaction curve was determined for this particular mine by measuring the load on individual supports and the associated roof-to-floor convergence in the vicinity of the supports. The results are shown in figure 10.

First, it is important to note where these measurements were obtained. Since abutment loading changes dramatically in longwall mining as the face approaches, the ground reaction curve should always be established at the most severe load condition, which generally will be just behind the tailgate shield. In this case, the mine wanted to maintain sufficient control of the tailgate entry to maintain a ventilation airway back to the next open crosscut. Hence, measurements of support loading were obtained to distances of 50 to 100 ft inby the face. For reasons previously explained, a different ground reaction was measured through the intersections than in the entries (figure 10). These measurements were also made under the deepest cover, again to establish the "worst-load" condition.

The following analysis applies to the entries at positions where there was no influence from the crosscuts. The load on the four-point wood cribs was estimated at 40 tons with a roof-to-floor convergence of 4 in. Qualitatively, it was noted that the integrity of the immediate roof was showing signs of deterioration at this convergence, which suggests that 4 in is approaching critical convergence where failure of the roof is inevitable. Some of the wood cribs in the area were also showing signs of premature failure, probably due to poor-quality timber. Conversely, convergence in the concrete-crib-supported area was only 0.5 in, and the measured load on the cribs was 62% greater at 65 tons per crib. These support loads were converted into a support system load density of 0.625 tons/ft^2 for the wood cribs (equation 1) and 1.35 tons/ft^2 for the concrete stopping block cribs (equation 2).

$$\text{Load density (wood cribs)} = \frac{4\text{ cribs} \times 40\text{ tons/crib}}{16 \times 16\text{ ft}} = 0.625\text{ tons/ft}^2 \tag{1}$$

$$\text{Load density (concrete cribs)} = \frac{4\text{ cribs} \times 65\text{ tons/crib}}{16 \times 12\text{ ft}} = 1.3\text{ tons/ft}^2 \tag{2}$$

These two data points were then used to establish the ground reaction curve for the tailgate entry inby the longwall face as depicted in figure 10. Since measurements were made on only these two support systems, a linear approximation to the ground reaction curve was made using these data as end points with a straight line connecting them. This curve could then be used to determine the support load density required to control the convergence inby the longwall face from 0.5 to 4.0 in. For example, if the goal were to limit convergence to 2 in, then drawing a line from 2 in vertically upward until it intersected the ground reaction curve and then drawing a line horizontally to the y-axis would reveal that a support load density of 1.04 /ft^2 must be provided. An algebraic solution to the problem can also be found by determining the slope and y-intercept for the ground reaction curve. Once the algebraic equation for the line is determined, the support load density at any displacement can be calculated.

$$\text{Load density (2-in convergence)} = -0.20171 \times 2.0 + 1.45 = 1.04\text{ tons/ft}^2 \tag{3}$$

The next requirement is to transform the required support load density into a support system design. Support load density is determined primarily by two factors: (1) the stiffness of each support and (2) the spacing of the supports. Continuing with our example, if we want to increase the support load density of the four-point wood crib support system from its current 0.625 to 1.04 tons/ft^2 in order to reduce convergence in the entry from 4 to 2 in, then we would need to decrease the spacing of the wood cribs from the current 4-ft spacing. The question is by how much? The required center-to-center spacing to provide a support load density of 1.04 tons/ft^2 can be determined by first identifying the load capacity of a wood crib at 2 in of convergence, which is found from the performance data developed from the laboratory tests conducted in the Mine Roof Simulator (figure 2). As shown in figure 2, the capacity of a four-point wood crib is 27 tons at 2 in of displacement. The spacing of a single row of cribs is then determined by dividing this capacity by the product of the required support load density and entry width. As the following analysis shows, the required spacing of 4-point wood cribs to achieve a support load density of 1.04 ton/ft^2 1.6 ft. Such a tight spacing can only be achieved through a staggered double row arrangement, where the center-to-center spacing in each row is 3.2 ft.

$$\text{Spacing}\,(2\text{-in convergence}) = \frac{\text{Capacity}\,(2\,\text{in})}{\text{Load density} \times \text{entry width}} = \frac{27}{1.04 \times 16} = 1.6\ \text{ft} \tag{4}$$

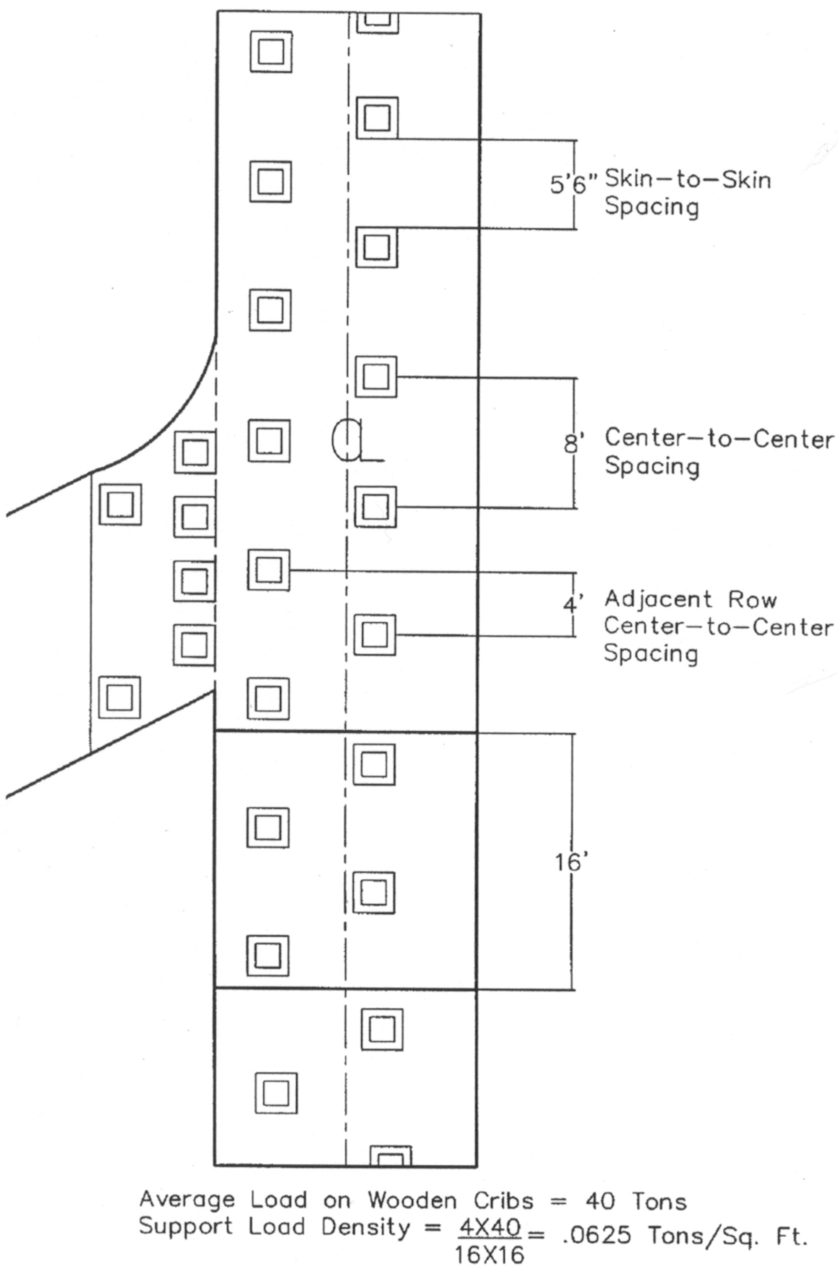


Figure 8.–Arrangement of four-point wood cribs in a western Pennsylvania mine.

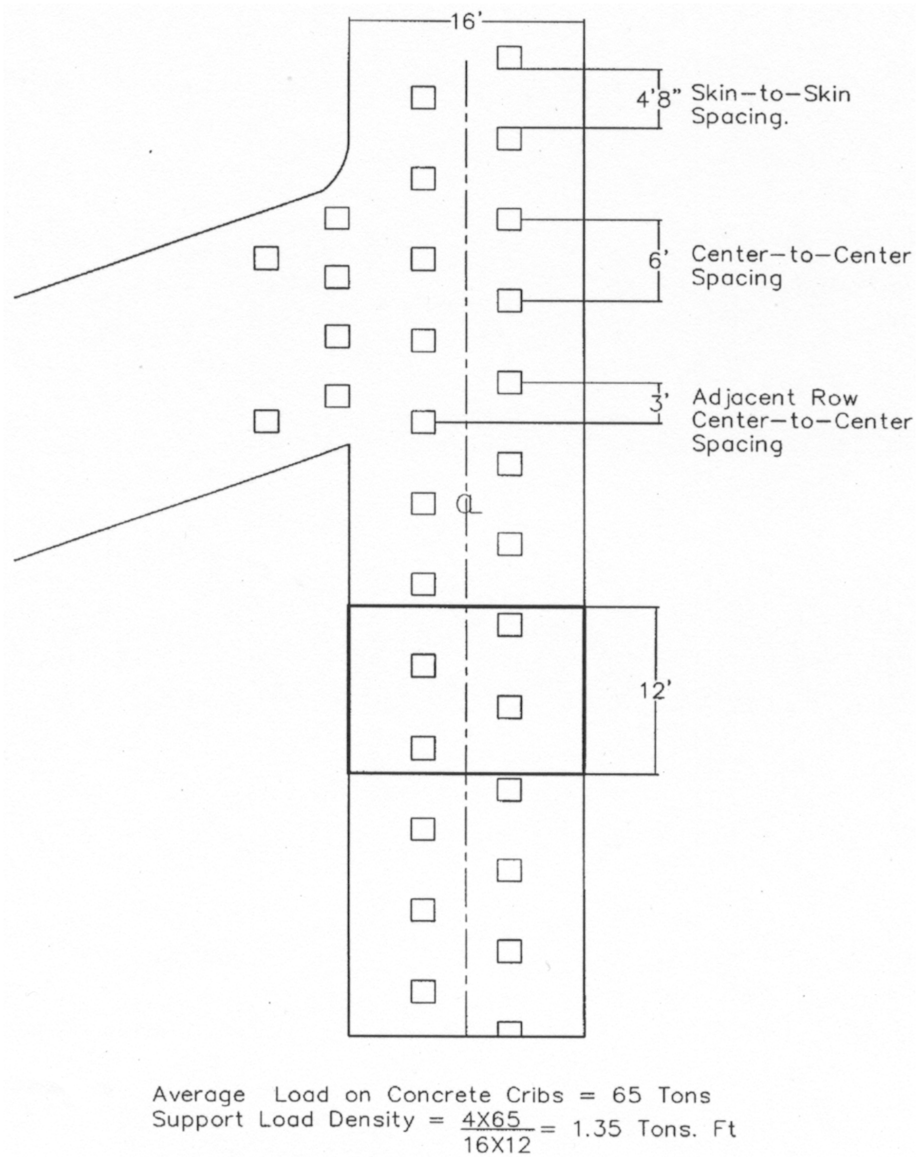


Figure 9.–Arrangement of stopping block concrete crib support in a western Pennsylvania mine.

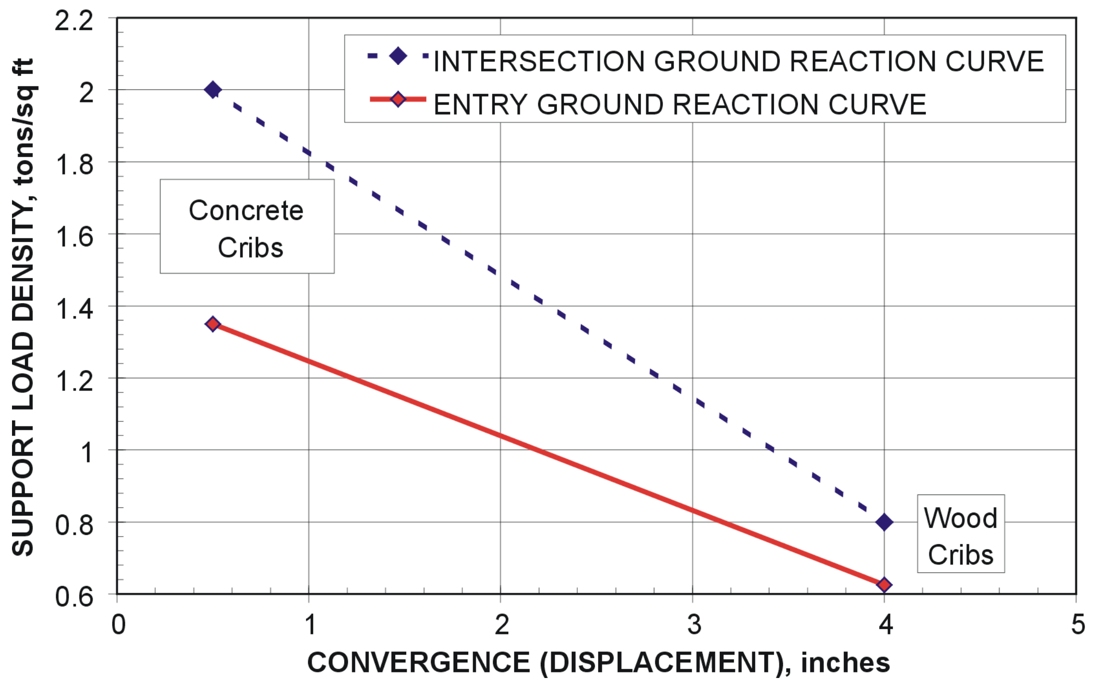


Figure 10.–Measured ground reaction curve for a western Pennsylvania coal mine.

DESIGN METHODOLOGY FOR LONGWALL TAILGATES

The concept examined for the western Pennsylvania mine can be generalized and employed at any mine to optimize secondary support design and application. The first step in the design process is to determine the ground reaction behavior. This can be done by installing at least two, and preferably three, support systems of varying support system stiffness and measuring load on and corresponding convergence of the mine roof and floor. It is important that these supports cover a wide range of stiffness to provide a full picture of ground reaction behavior. As part of this process, an effort should also be made to determine the critical roof-to-floor convergence where roof failure occurs, since this will be a critical design value for the support design. However, in order to do this, a low density support system must be installed that will allow considerable convergence to take place. Since this poses a risk of inadequate ground control, precautions should be taken. One possibility is to set additional wood cribs through this trial zone with the top layer removed so that they could be reinstalled to provide additional support if necessary, or in the worst case, provide ground support after the roof deforms (5-6 in) to the cribs if they cannot be topped off.

Altering support system stiffness can be done several ways. One way is to utilize the same support and same spacing down the entry, but increase the number of rows of support across the mine entry from one to two to three, which would proportionally increase system stiffness by the same factors. Another way is to keep the spacing and number of rows of support constant, but use supports of varying stiffness. This would eliminate the impact of both span and roof coverage which could be limiting factors in support placement (load deformation) strategies. For example, using conventional wood cribs, the support construction could be varied from a 4-point to a 9-point to a 16-point crib. The support load density would increase in direct proportion to the increase in support stiffness.

Adjusting the support spacing down the entry could also be considered. Support load density is proportionally increased as the spacing is decreased. Care should be taken to avoid excessive spacing that will cause span-related problems. A good rule of thumb is that the support spacing should not exceed half the entry width. The load-bearing area (quality and extent of the contact area) of the support also an important factor to consider. The pressure exerted by any support should not exceed the strength of the mine roof or floor.

Ideally, the loads on the various support systems should be measured underground. Support load measurements are typically made through a hydraulic flat jack. This will be more difficult to do on some support designs than others. Supports with a single contact area, such as concrete cribs, are easier to work with than something like a conventional wood crib, which has multiple loading paths. Theoretically, if loads cannot be measured, they can be estimated directly from laboratory load-displacement data provided convergence (support displacement) measurements are made. However, this estimate may not be accurate for materials such as wood, where creep can occur and distort the approximation of the load that produced the measured displacement. Measurements of convergence are essential to this design methodology. Roof-to-floor convergence measurements can be made in several ways, but it is important that these measurements correlate to displacements induced in the support structure.

Readings of load versus displacement (convergence) should be made under the most severe load conditions that occur. For most longwall mines, this will be at the back of the tailgate shield. If it is critical that the tailgate be kept open inby the face for ventilation reasons, then ideally the load-displacement readings should be made inby the face. Of course, this may not be as easy to do, but it is important to realize that the design methodology assumes worst-case load conditions, and safety factors will need to be employed if measurements are taken under less severe load conditions. Another good rule of thumb to follow is that the face should be retreated a distance approximating the width of the face before the full ground reaction behavior is established. In other words, if the face width is 1,000 ft, then the face should be retreated at least 1,000 ft before ground reaction behavior is measured. Of course, the first requirement for any tailgate design is proper pillar design. While in theory ground reaction behavior can be determined for any pillar design and roof geology, the methodology proposed here assumes that the pillar design falls above the ALPS design line for a given CMRR [Mark et al. 1994].

The load-displacement data are then used to generate a plot of support load density as a function of convergence (ground reaction curve). Each support type with different stiffness represents one data point on the ground reaction curve. The support load density is determined as the measured load in the support at the observed convergence times the number of supports per unit area of mine entry (equation 5). For a single row of supports employed on a constant center-to-center spacing, the support load density can be determined by equation 6.

$$\text{Support load density} = \frac{\text{No. of supports} \times \text{support load}}{\text{Area of support coverage}} \quad (5)$$

$$\text{Support load density} = \frac{\text{Support load}}{\text{Center-to-center spacing} \times \text{entry width}} \quad (6)$$

Once the ground reaction curve is developed, the center-to-center spacing (down the mine entry) of alternative support systems arranged in a single row required to achieve ground control (equilibrium) at a desired convergence can be determined from equation 7. The center-to-center spacing of a double row of supports is simply twice that of a single-row arrangement.

$$\text{Spacing (displacement)} = \frac{\text{Capacity (displacement)}}{\text{Load density} \times \text{entry width}} \quad (7)$$

Where spacing (displacement) = center-to-center spacing of a single row of supports in feet,

Capacity (displacement) = Individual support capacity in tons at a specified displacement (obtained from laboratory performance data) equal to the desired convergence control, tons

Support load density = Support load density in tons/ft^2 at the required convergence (obtained from ground reaction curve) in tons per square feet, and

Entry width = width of the entry in feet.

This design methodology is a valuable tool in optimizing the utilization of standing secondary roof support technology. However, as previously described, it is still up to the mining engineer to decide how close to the critical convergence he/she wants to operate based on knowledge of the particular ground conditions. A margin of safety is provided by designing for a convergence that is less than the critical convergence (minimal acceptable support load density). To make equivalent comparisons of alternative support systems, a safety factor can be quantified by comparing the design support load density to the minimal acceptable support load density that will be representative of the maximum allowable (critical) convergence. This is referred to as the ground reaction curve (GRC) safety factor (equation 8).

$$\text{GRC safety factor} = \frac{\text{Design load density}}{\text{Minimal acceptable load density}} \quad (8)$$

Another factor to consider is whether the support is being fully loaded and how much reserve capacity is left in the support at the design load. In the event that load conditions worsen beyond expectations, this reserve support capacity may be needed to support the mine roof. If the support characteristic is such that the support sheds load quickly after reaching its peak load, such as the nonyielding concrete supports (figure 5), then consideration must be given to avoid designing near the peak loading capability of the support. A safety factor for the support can be defined based on support loading at the required support load density in relation to the peak loading capability of the support (equation 9). Hence a safety factor of 1 indicates that there is no reserve capacity available, and a safety factor of 2 indicates that the support is loaded to only 50% of its maximum support capacity.

$$\text{Safety factor (support)} = \frac{\text{Peak load capability}}{\text{Load at installed load density}} \quad (9)$$

ALTERNATIVE SUPPORT STUDIES AT THE WESTERN PENNSYLVANIA MINE

Four different standing support systems and one cribless system were installed in the longwall tailgate entry of this mine. In addition to conventional four-point wood crib supports and concrete stopping block supports, the alternative standing supports were (1) Heintzmann Corp.'s Alternative Crib Supports (ACS's), (2) HeiTech Corporation's Pumpable Crib Supports, (3) Strata Product's Propsetters, and (4) Burrell Mining Products' The Can. Cable trusses were used in the cribless area. The alternative supports were assigned to sections of the tailgate between the standard wood cribbing. This was done to ensure that there was no interaction between support installations, thereby allowing a fair evaluation to be made under equivalent conditions (i.e., normal cover, no excessive roof or floor damage, normal geology).

Table 1 shows the installed spacing of the alternative support systems and the typical 4-point wood crib system and concrete stopping block crib system. The support load density of the alternative supports is calculated by matching system performance to the ground reaction curve. Essentially, this requires working backward through the design methodology. The following steps can be used.

1. Pick an arbitrary convergence within the bounds of the ground reaction curve.
2. Determine the required support load density that matches the ground reaction curve for this convergence.
3. Identify the individual support capacity at this displacement from the laboratory performance data at this convergence.
4. Determine the support load density from equation 6.
5. If the support load density is greater (falls above the curve) than the required support load density, a lower convergence should be chosen. If the support load density is less (falls below the curve) than the required support load density, then higher convergence, should be chosen and steps 1 through 4 repeated until the support load density matches the ground reaction curve.

An analysis of table 1 reveals that all four alternative support systems were installed with sufficient support load density to control the convergence well below the critical level of 4.0 in provided by the 4-point wood cribs. The high safety factor utilized in these alternative support applications was to provide a margin of safety in anticipation of a tailgate horizontal stress concentration on the next panel.

The HeiTech pumpable support had the highest load density at 1.35 tons/ft^2, which limited convergence to approximately 0.5 in. Conditions both inby and outby the face as shown in figure 11 were excellent with the HeiTech pumpable support system. However, it should be noted that the HeiTech support also had the lowest support safety factor (1.3) of the four alternative support systems utilized, meaning that load development approached the peak loading capability of the support. If the maximum loading capability of the support was exceeded due to unexpected additional roof loading or variability in the peak strength of the support, convergence would increase to approximately 4 in at the installed spacing based on the residual support capacity of approximately 90 tons. Since this is the critical convergence for this mine (at which point roof conditions deteriorate significantly), it is critical that the peak pumpable support capacity not be exceeded through the zone where it is desired to maintain full roof control, which means that the spacing must be properly maintained during installation so as not to overload the support past failure.

Table 1. Assessment of standing alternative support technologies utilized in study at a western Pennsylvania. mine.

Support system	Installed spacing, ft	Installed load density, tons/ft^2	Conv. control, in	Safety factors		Observed roof condition	
				GRC[1]	Support[2]	Outby face	Inby face
Four-point wood crib	8.0 (DR[3])	0.625	4.0	1.00	1.8	Good	Marginal
Concrete stopping block crib	3.0 (DR)	1.35	0.5	2.16 (0[4])	1.0	Good	Marginal
Heintzmann ACS	5.0 (DR)	1.20	1.24	1.92	2.5	Excellent	Good
HeiTech Pumpable concrete crib	9.2 (SR)	1.35	0.5	2.16	1.3	Excellent	Excellent
Strata Products Propsetter	4.0 (DR)	1.12	1.6	1.79	1.7	Excellent	Good
Burrell Mining Products Can support	7.0 (DR)	1.19	1.25	1.90	1.8	Excellent	Excellent

[1]Ground reaction curve safety factor is determined from equation 8 as the ratio of installed support load density to minimum allowable support load density.
[2]Support safety factor is determined from equation 9 as the ratio of peak loading capability of the support to load developed at installed spacing.
[3]All double rows of supports were installed in a staggered fashion. The spacing here refers to the spacing of one row of supports. With the staggered arrangement, the spacing between adjacent supports of both rows is half of that of the individual row (see figure 8 or 9).
[4]The roof condition was good prior to failure of the support. Hence, the installed support load density actually dropped to zero once the support failed, which accounted for deterioration in the integrity of the roof.
DR = Double row. SR = Single row.

The Propsetter support had the lowest margin of GRC safety at 1.79, but even this system was conservative in that the convergence was limited to 1.6 in. Conditions outby the face were excellent, as shown in figure 12A, and relatively good inby the face (figure 12B). Some of the Propsetter supports well inby the face (mostly in the intersection areas) (figure 12C) appeared to be in a state of post-yield deformation where "brushing" (yielding) caused the props to tilt from a vertical orientation, which is normal for this load condition. It does not mean that the prop is shedding load. Another possibility is that the props were being dislodged or moved laterally by flushing of the gob material, floor heave, and/or lateral displacements of the roof relative to the floor by the cantilevered roof beam. Despite these occasional abnormalities in the support condition, the Propsetter was able to maintain an effective air way beyond the first open crosscut inby the face. It was also reported by the mine that five or six Propsetters were dislodged from the mine roof and floor outby the face. Since convergence was minimal outby the face, the cause of these props "falling over" was never definitively determined. The same props were reinstalled and performed well throughout the duration of the test.

Figure 11a.–Outby area supported with pumpable crib.

Figure 11b.–Inby area supported by pumpable crib.

The Heintzmann ACS support had the most limited yield capability of the four alternative support technologies used at this mine. The ACS also shed load rather quickly after reaching its peak loading capability at about 2.2 in (figure 7). However, the installed spacing provided the highest support safety factor (2.5), meaning the loads were kept well below the peak capacity of the support. Likewise the installed load density limited the convergence to 1.2 in, which is considerably less than the yield point of 2.2 in. Hence, this is a good example of how a stiff support with limited yield capability can provide effective ground control in a longwall tailgate, provided that a sufficient number of supports are installed per unit area to establish a high enough load density to minimize the ground movement. Figure 13 shows the condition of the entry both outby and inby the face in the area supported by ACSs. Similar to the Propsetter support, a few of the ACS props were tilted inby the face, but continued to provide support capability in this condition without becoming unstable.

The Burrell Can support installation had a GRC safety factor (1.92) almost identical to that of the area supported by the Heitzmann ACS. The 1.92 GRC safety factor means that the installed load density was almost twice that needed to prevent roof failures from occurring. The entry conditions, both inby and outby the face, were excellent with the Burrell Can support, as shown in figure 14. Inby, the conditions were slightly better than in the area supported by the ACS. This improvement is attributable to the larger surface coverage and improved stability of the Burrell Can support compared to the ACS.

Figure 12a.–Outby area supported by Propsetter support.

Figure 12b.–Inby area supported by Propsetter support.

Figure 13a.–Outby area supported by ACS supports.

Figure 12c.–First open crosscut intersection inby the face supported with Propsetter supports.

Figure 13b.–Inby area supported by ACS supports.

Figure 14a.–Outby area supported by Burrell Can support.

Figure 14b.–Inby area supported by the Burrell Can support.

USING THE GROUND REACTION CURVE TO OPTIMIZE THE USE OF ALTERNATIVE SUPPORTS

Once the ground reaction curve is determined, use of any other support technology can be defined by strategic employment strategies relative to the ground reaction curve. Table 2 shows alternative placement strategies for these and other alternative support systems for this particular mine site based on measured ground reaction behavior with conventional wood and concrete support systems.

Examining table 2 reveals that other support technologies could be used favorably. One example is the Link-N-Lock crib support developed by Strata Products. A Link-N-Lock crib 24 in long could be installed in a staggered double row with a center-to-center spacing of 11 ft per row (5.5 ft between cribs in opposite rows); this support could limit convergence to 2 in. Another alternative would be a single row of 36-in Link-N-Lock cribs on an 8-foot center-to-center spacing, which also would limit convergence to 2 in. In contrast, a single row of nine-point cribs would have to be installed on a 5.2-ft center-to-center spacing to provide equivalent ground control capability.

Three other points can be made by examining the data in table 2. First, the spacing of stiff, high-capacity support systems can become excessive at large displacements. It is important to remember that ground reaction behavior was measured in the immediate vicinity of secondary support. It is assumed that support loading is sufficiently transferred to control the roof and floor between the supports. Obviously, there are some limitations to this capability, which largely depend on the strength of the immediate roof. Generally, stronger roofs can span greater distances between supports than weaker roof. Currently, this capability is best obtained from empirical data within the mine, but a good rule of thumb to follow in the absence of specific information is that the span between supports should not exceed half the entry width, particularly in weak roof conditions such as those observed in the Pittsburgh coal seam. Using this criterion, it is seen from table 2 that nine-point cribs and Link-N-Lock cribs must be employed at a load density greater than 0.63 tons/ft^2 to avoid an excessive spacing where failure might occur between the cribs. Surface control of the immediate roof is another issue. Surface control refers to failure of the immediate skin of the roof. This is different from the excessive spacing issue discussed above in that there is no major failure of the roof rock. If surface control is necessary to prevent flaking of the roof skin between cribs, then methods such as wire meshing can be effectively employed as a control measure.

Second, it is seen from table 2 that stiff concrete supports must be installed at a high enough support load density to limit convergence to less than 2 in; otherwise, the supports will fail prematurely, resulting in no support capability and unstable ground conditions after failure of the support. Third, it is seen that required load density to limit convergence below 1 in is not practical with passive wood crib supports, including the stiffer Link-N-Lock cribs. Wood is just too soft to generate meaningful loads at such small displacements.

Table 2. Recommended placement and safety factors for alternative support technologies.

Support system	Center-to-center support spacing and individual support capacities at convergences of 0.5,1, 2, and 4 in.							
	0.5 In (LD =1.35), (SF =2.16)		1 IN (LD = 1.24), (SF = 2.0)		2 In (LD = 1.04), (SF = 1.7)		4 In (LD = 0.63), (SF = 1.0)	
	Load (tons)	Space (ft)	Load (tons)	Space (ft)	Load (tons)	Space (ft)	Load (tons)	Space (ft)
Four-Point cribs	8	0.7	17	0.9	27	1.6	39	3.9
Nine-Point cribs	14	0.6	55	2.8	86	5.2	115	11.4
24-in Link-N-Lock	10	0.5	45	2.3	92	5.5	115	11.4
27-in Link-N-Lock	11	0.5	49	2.5	102	6.1	127	12.6
36-in Link-N-Lock	13	0.6	63.5	3.2	132	8.0	162	16.1
Propsetter (8.5 in dia)	12	0.6	25	1.3	42	2.5	57	5.7
Stopping block cribs	65	3.0	0	N/A	0	N/A	0	N/A
SFR donut cribs	86	4.0	280	14.1	0	N/A	0	N/A
SFR block (2 per layer) crib	85	4.0	210	10.6	0	N/A	0	N/A
HeiTech Pumpable Crib . .	190	8.8	240	12	112	6.7	90	8.9
Burrell Can (24-in dia)	40	1.9	65	3.3	90	5.4	90	8.9
Heintzmann ACS (100 ton)	39	1.8	46	2.3	102	6.1	36	3.6

N/A - Indicates that support would fail prior to the designated convergence and would not have sufficient post-failure (residual) capacity to be considered for use in this condition. LD designates the support load density of a single row of cribs in the designated center-to-center spacing. SF refers to the GRC safety factor as computed by equation 8 for a single row of cribs. If two rows of supports were used, the designated spacing at a specific displacement would be reduced by a factor of 2, and the safety factor would be increased by a factor of 2.

OTHER CONSIDERATIONS IN TAILGATE SUPPORT SELECTION AND APPLICATION OF THE PROPOSED DESIGN METHODOLOGY

While the primary consideration in support design is obviously the prevention of roof falls through proper ground control, there are other factors to consider. These include (1) cost of the support, (2) material handling requirements and ease of installation, and (3) impact of the support structure on ventilation. These issues are beyond the scope of this paper.

While this paper is focused on standing roof support applications, several mines have explored the application of intrinsic secondary support, such as trusses, to replace conventional standing support in longwall tailgates [Mucho 1998]. Two points need to be made in reference to truss supports. First, it should be noted that the design methodology proposed in this paper applies only to standing roof support. While some of the basic rock mechanics principles used here may apply to intrinsic support, the support mechanisms are different, and these have not been examined in this study.

Another caveat of the design methodology pertains to application in yield pillar gate roads. While in theory a ground reaction behavior can be established for yield pillar systems, the mechanisms of ground behavior and support interaction are different. In particular, the yield pillar system is a high deformation environment by design. Secondary support should ideally allow the ground to yield in accordance with pillar deformation and not interfere to the point where the secondary support develops sufficient capacity to damage the roof while it is yielding. Hence, a stiff, high-density support may not be desirable in this environment, and the important secondary support design consideration may well be the stability and yield capability of the support.

CONCLUSIONS

Several alternatives to conventional wood and concrete cribbing have been developed in recent years. These new support technologies provide improvements in supporting capability as well as material handling advantages. However, since their supporting characteristics are all different, a design methodology must be developed so that for mines can employ these technologies and safely maximize their benefits without increasing the overall cost of support. Conservative applications or trial-and-error assessments are no longer practical nor prudent for state-of-the-art longwall mines.

The design methodology proposed in this paper and examined through a field trial at a western Pennsylvania coal mine embodies a fundamental concept of rock mechanics, that being the "ground reaction curve." Measurement of the ground reaction curve at this mine indicated that support capacity had a significant impact on the ground behavior in the longwall tailgate. Increasing the support load density by a factor of 2 from 0.625 to 1.25 tons/ft^2 decreased convergence in the entry from 4 to 1 in. Conventional four-point wood cribs installed in a double row with an 8-ft center-to-center spacing in a staggered arrangement (4-ft center-to-center spacing between adjacent rows) resulted in marginal ground control. Concrete cribs constructed from concrete stopping blocks reduced convergence to 0.5 in, but some failed under this amount of deformation, resulting in localized poor ground control resulting from support failure.

Four alternative standing support technologies were installed at the western Pennsylvania mine: (1) Can (Burrell Mining Products), (2) Alternative Crib Support (Heintzmann Corp.), (3) Pumpable concrete support (HeiTech Corp.), and (4) the Propsetter (Strata Products USA). These alternative support technologies were installed at a support load density ranging from 1.12 to 1.35 tons/ft^2, providing ground control safety factors of 1.79 to 2.16. Ground conditions for all these support applications were generally very good, which is consistent with the measured ground reaction behavior and installed support density.

The NIOSH Support Technology Optimization Program (STOP) has been developed to facilitate the use of this design methodology and allow mines to optimize the use of any support technology once a ground reaction curve for that particular mine has been identified [Barczak 2000].

REFERENCES

Barczak TM [1994]. Assesment of wood and alternative materials for secondary roof support construction. In: Peng SS, ed. Proceedings of the 13th International Conference on Ground Control in Mining. Morgantown, WV: University of West Virginia, pp. 190-201.

Barczak TM [2000]. Optimizing secondary roof support with the NIOSH Support Technology Optimization Program (STOP). In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Deere DU, Peck RB, Monsees JE, Schmidt B [1970]. Design of tunnel lining and support systems. Highway Research Record, Washington, DC, No. 339, pp. 26-33.

Mark C, Chase FE, Molinda GM. [1994] Design of longwall gate entry systems using roof classification. New Technology for Longwall Ground Control. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, Special Publication 01-94, pp. 5-17.

Mucho TP [1998]. In focus—cable bolts: a “new” support. Falls Church, VA: U.S. Department of Labor, Mine Safety and Health Administration, Holmes Safety Association Bulletin *Mar*:3-4.

OPTIMIZING SECONDARY ROOF SUPPORT WITH THE NIOSH SUPPORT TECHNOLOGY OPTIMIZATION PROGRAM (STOP)

By Thomas M. Barczak[1]

ABSTRACT

The 1990s brought an unprecedented increase in the development of innovative technologies to provide more effective and easier-to-install roof support in underground mines. To facilitate the application of these technologies in improving mine safety, the National Institute for Occupational Safety and Health (NIOSH) developed the Support Technology Optimization Program (STOP). STOP is a Windows-based software program that provides mine operators with a simple and practical tool to make engineering decisions about the selection and placement of these various roof support technologies. The program includes a complete database of the support characteristics and loading profiles obtained through safety performance testing of these supports at the NIOSH Safety Structures Testing Laboratory. A support design criterion in the form of the required support load density at a specified convergence can be established from four options: (1) a database of measured ground reaction obtained from various mines or ground behavior information input by the user, (2) load requirements based on a detached roof block or rock failure height, (3) criteria based on the current roof support system, and/or (4) arbitrary criteria set by the user. Using these design criteria, the program will determine the installation requirements for a particular support technology that will provide the necessary support load density and convergence control. Optimization routines are also available to determine the most efficient support design for a user-specified support installation. In addition to these performance measures, STOP can be used to compare material handling requirements and installation costs. Comparisons among the various support technologies are easily made, including a graphical analysis of relevant support parameters. This paper describes STOP and its application to optimizing standing secondary roof support systems.

[1]Research physicist, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

Secondary support provides additional roof support in the event of failure of the primary support system. When properly designed, secondary support will also assist the primary support in controlling the integrity of the immediate roof beam. Thus, roof support should be thought of as a three-part system: (1) the remaining coal pillars, which provide control of overburden weight, (2) the primary support system consisting of roof bolts, which help form a more competent roof beam and, in the case of mechanical bolts, attach the immediate roof beam in suspension to the more stable main roof rock, and (3) the secondary roof support system, which consists of standing roof support and intrinsic support elements designed to control deformation of the immediate roof and handle additional abutment loads during retreat mining. The latter occurs in longwall mining where the tailgate is frequently supported with various secondary roof support systems. It should be noted that the Support Technology Optimization Program (STOP) in its present form is limited to the evaluation of standing roof support systems.

Engineering design is applied primarily to size the pillars to account for load variations due to depth of cover, active mining zones, and the quality of the roof rock. Computer programs such as the Analysis of Longwall Pillar Stability (ALPS) and Analysis of Retreat Mining Pillar Stability (ARMPS) programs are valuable tools in designing pillars for various mining scenarios [Mark 1992; Mark and Chase 1997]. There are no universally accepted design criteria for primary support (roof bolts), although recent research by the National Institute for Occupational Safety and Health (NIOSH) indicates that there may be some fundamental criteria to define conditions where current primary support densities are inadequate. The Wood Crib Performance Model [Barczak and Gearhart 1993] was developed by the former U.S. Bureau of Mines in 1994 to provide engineering design for conventional wood cribs, which were the predominant form of secondary support at the time. The Wood Crib Performance Model was used to determine the supporting capability of various conventional wood crib configurations and match this capability to user-defined load and convergence criteria. More recently, a design methodology for standing secondary roof support in longwall tailgates was developed that incorporates measured ground reaction data into the formulation of the load and convergence design criteria for standing roof support systems [Mucho et al. 1999; Barczak et al. 1999].

In the past 5 years, there have been 16 new standing roof support systems developed for use as secondary roof support. These new support systems not only provide superior roof support, but many provide material handling advantages as well. STOP was developed by NIOSH to allow mine operators to compare these various support systems and to optimize the application of both new standing roof support technologies and conventional wood and concrete crib support systems. Although STOP can be considered as an upgrade of the original Wood Crib Performance Model, it is built on a Windows-based architecture and has several enhanced features that were not available in the previous Wood Crib Performance Model. These include—

(1) Selection from a database of currently available standing roof support systems for evaluation;

(2) A synopsis of pertinent design and installation criteria for each support system;

(3) A description of performance characteristics, including photographs of the support loading profile showing the condition of the support as it deforms;

(4) Name and telephone numbers for support manufacturers;

(5) Ground reaction curve support design criteria where the laboratory support performance can be matched to a curve corresponding to ground behavior, as opposed to a single (load and convergence) data point, as was done in the original Wood Crib Performance Model;

(6) Enhanced optimization algorithms that determine the most sufficient support design for user-specified spacing limitations and/or the user-defined load density and convergence requirements;

(7) Material handling and cost information for each support; and

(8) graphical displays of support system capabilities.

It is important to understand that although there are now a wide variety of support choices, each of these support systems has a unique performance profile. Simply replacing one support system with another will not provide equivalent ground control. Most of the new support technologies provide superior supporting capability, which may allow wider spacings of the support to be used if the goal is to provide support capability equivalent to that of a conventional wood crib support system. STOP will determine the spacing requirements that will provide equivalent support capacity. This is one way that STOP can optimize the use of a particular support system. STOP can also provide important information regarding the benefits of increasing support load density. Using measured ground reaction data, STOP will determine either the convergence that can be expected from a certain support design installed on user-selected spacings or the support spacing required to limit convergence to a certain level.

This information can be very useful when petitioning the Mine Safety and Health Administration (MSHA) for approval to use an alternative support technology. Without this information, MSHA will typically require a trial section where the alternative support system can be observed before full approval is granted. In longwall mining, this means that a mine operator might have to wait for a full panel of mining before implementing a new support technology. Likewise, without an engineering basis to justify a change, variations in placement strategy or implementing a change in support design can be delayed until a trial section is observed. Thus, STOP can be included as another part of the overall process that MSHA may use in approving a roof support plan. While it may not be the sole deciding factor, STOP can provide critical engineering data that will facilitate a decision regarding the implementation of these new support technologies.

This paper introduces STOP, describes the architecture of the program, and provides several examples of how it can be used to optimize the design and use of secondary roof support technologies. STOP can provide an engineering foundation to ensure that inadequate support designs, as well as ultra-conservative support applications, are avoided. Safety will be improved by proper matching of support performance to mine conditions, which will reduce the likelihood of roof falls and blocked escapeways. Material handling injuries associated with support construction are known to account for about 5,000 lost workdays per year in underground coal mines. STOP can help to define the material handling advantages of alternative support technologies that use lighter weight materials or systems that can be installed with mechanical assist. The use of these support technologies can significantly reduce material handling injuries.

PROGRAM ARCHITECTURE

STOP is a Windows-based architecture. The Main Menu allows the user to control the flow through the program if desired. This window can be accessed through each of the primary program segments. The Main Menu contains six modules: File, Design Criteria, Support Evaluation, Comparison, Information, and Help.

File: The *File* module contains file management subroutines that allow the user to create new files, open and close existing files, and exit from the program. The File menu also allows the user to set the path for storing several photographs that are incorporated into the program to allow visual display of support performance.

Design Criteria: The *Design Criteria* module is where the load and convergence design criteria for the support system are formulated. The requirement is to define the required support load density in terms of tons of support capacity per linear foot of entry advance and at what convergence this support capability is to be provided. There are four different ways to establish these design criteria in the program: (1) ground reaction curve, (2) detached block, (3) current support system, and (4) arbitrary criteria.

1. *Ground Reaction Curve* (figure 1) allows the user to define support load density and convergence criteria from in-mine measurements of the ground behavior (convergence) associated with various support systems [Mucho et al. 1999; Barczak et al. 1999]. Essentially, the ground reaction concept implies that convergence in the mine entry is controlled by the magnitude of support resistance. Generally, convergence decreases with increasing support load density. Thus, if measurements of convergence are made with two or more support systems of varying stiffness, then a ground reaction curve can be established for that particular mine. The user can define a ground reaction curve or use one from the database established from various mine sites maintained in the program. Once a ground reaction curve is defined, the program will determine the required spacing for a particular support system that will provide the support load density consistent with the ground reaction curve at a specified convergence.

2. *Detached Block* is shown in figure 2. The support load density requirements are established by calculating the weight of a detached block of roof rock above the mine entry. The failure height can be inputted by the user or estimated from the quality of the roof rock (Coal Mine Roof Rating) using an approximation developed by Unal [1986]. The volume of the block is also influenced by the shape of the failure. Options include either an arch or a vertical shear failure at the pillar boundaries. Two options are available for determining the convergence criteria. If ground reaction information is available, this information can be used to help define the convergence criteria. In terms of the ground reaction curve, there is a critical convergence where failure of the roof occurs. This could be used to define the convergence criteria for the detached block, the idea being that the support should put the roof rock mass into equilibrium before the critical convergence is reached. If this option is selected, convergence is defaulted to the maximum convergence on the ground reaction curve, but the user can change this input if desired. In the absence of ground reaction information, the user can simply input a convergence criteria (allowable displacement before roof weight is supported in equilibrium).

3. *Current Support System* allows the user to define design requirements based on the performance of the current support system (figure 3). Two options are available. The first one is that if a ground reaction curve is available, then the program will determine where the current support system falls on the ground reaction curve and set the support load density and convergence requirements to that point. For the second option, the user must define an allowable convergence, which should be based on some in-mine measurements. The support capacity and resulting load density for support spacing will be calculated from the load-displacement profile for the support, as

determined from tests in the Pittsburgh Research Laboratory's (PRL) unique Mine Roof Simulator load frame or from other laboratory data.

4. *Arbitrary Criteria* simply allows the user to set the support load density and convergence criteria to any arbitrary set of values.

Support Evaluation*:* The *Support Evaluation* module is the heart of the support design process. Any of a variety of support systems contained in the program database can be selected for design and analysis (figure 4). The *Design* algorithm calls up a subroutine that allows the user to control relevant design parameters and/or pick from the available design (models) for a particular support type (figure 5). The user must also input the number of rows to be used in the support placement. The program will then calculate the required spacing of the supports to achieve the desired support load density at the designated convergence, or the user can select a support spacing and the achieved convergence will then be calculated for the user-defined support installation. An optimization algorithm is also included in which the program will determine the support design or model that most closely matches the design criteria (support load density at designated convergence). Also included in the *Support Evaluation* module are analyses of installed support costs and material handling requirements for the support.

Comparison: The *Comparison* module compares the support systems chosen for analysis. There are three windows in the Comparison module: (1) *Comparison Assessment Table*, (2) *Support Description Summary*, and (3) *Graphical Data Analysis*. The *Comparison Assessment Table* describes the support layout (number of rows and support spacing) and various design parameters for each support system in a tabular format. The parameters are grouped into six categories: (1) support layout, (2) ground control, (3) unit support costs, (4) normalized support cost, (5) installation assessment, and (6) material handling. The user can pick any one of the selected support systems as a baseline system for comparison purposes. The *Support Description Summary* summarizes the support design parameters for each support system. The *Graphical Data Analysis* window allows the user to plot support performance (unit support load or support load density) as a function of convergence and graphically compare the various support systems, as shown in the example in figure 6. Ground reaction data can also be displayed on the plots to show the convergence control provided by the various support systems relative to the ground reaction curve.

Information: The *Information* module can be thought of as a general information center. The various support technologies are categorized in six groups: (1) conventional wood (crib) supports, (2) engineered timber supports, (3) conventional

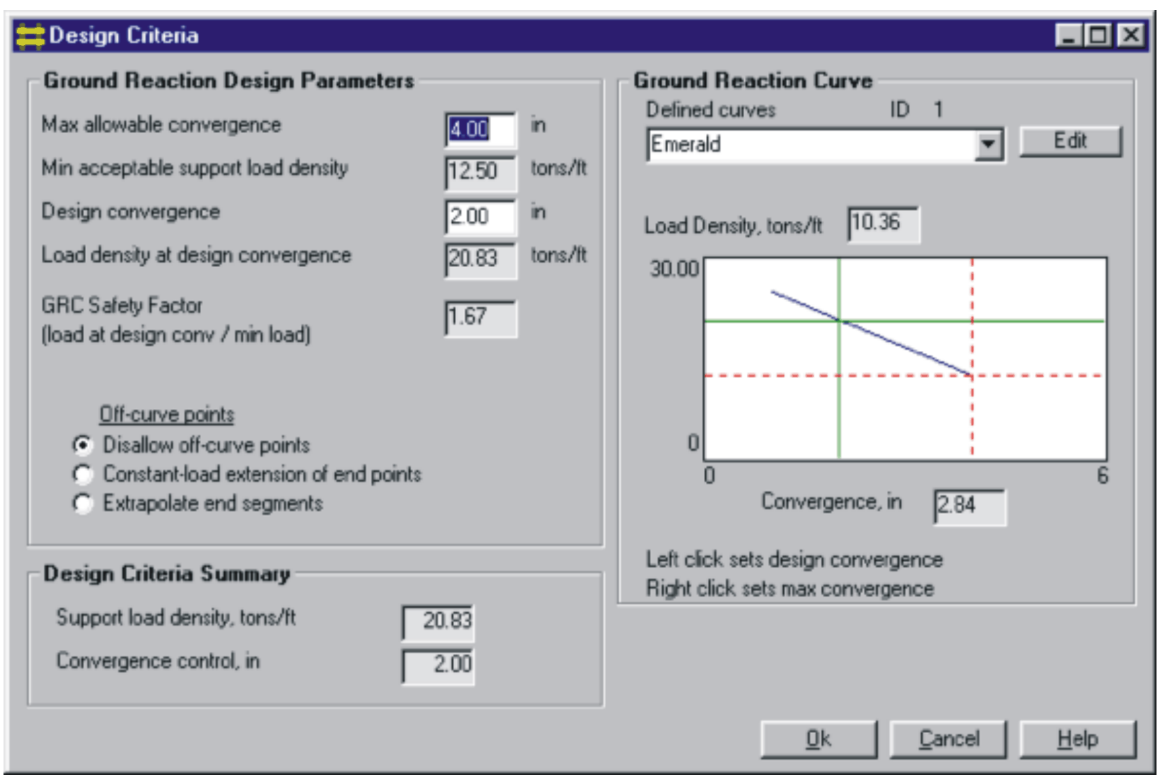


Figure 1.—Window for establishing design criteria based on ground reaction curve .

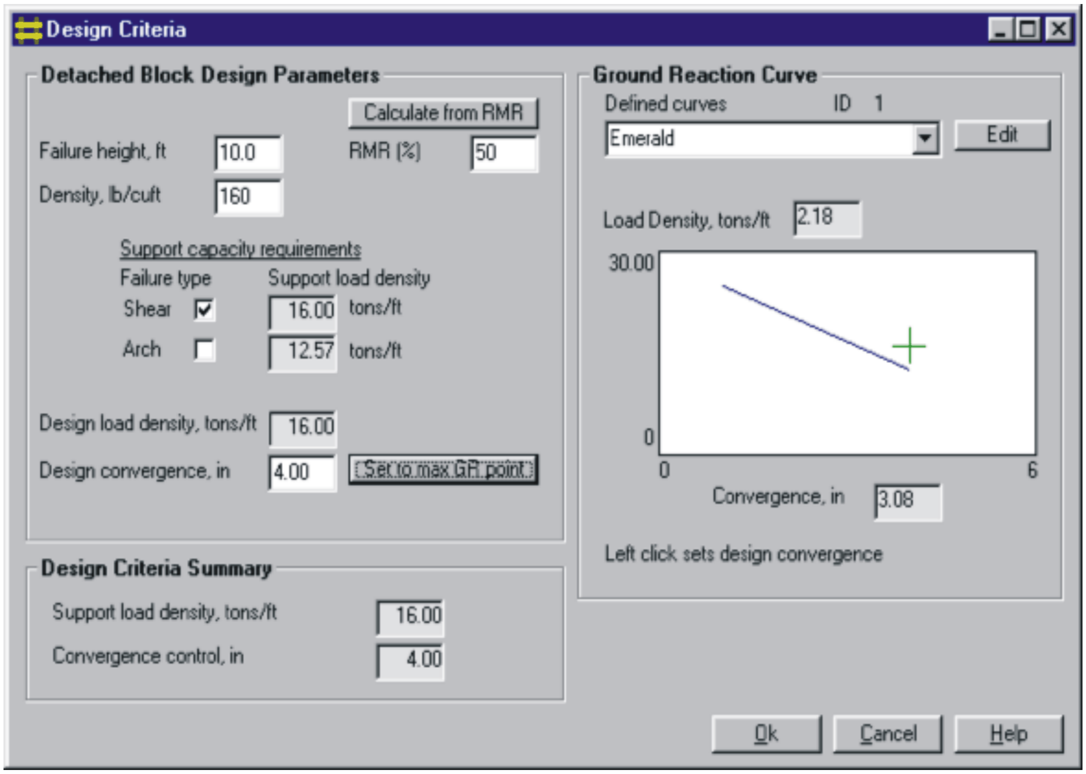


Figure 2.—Window for establishing design criteria based on detached roof block.

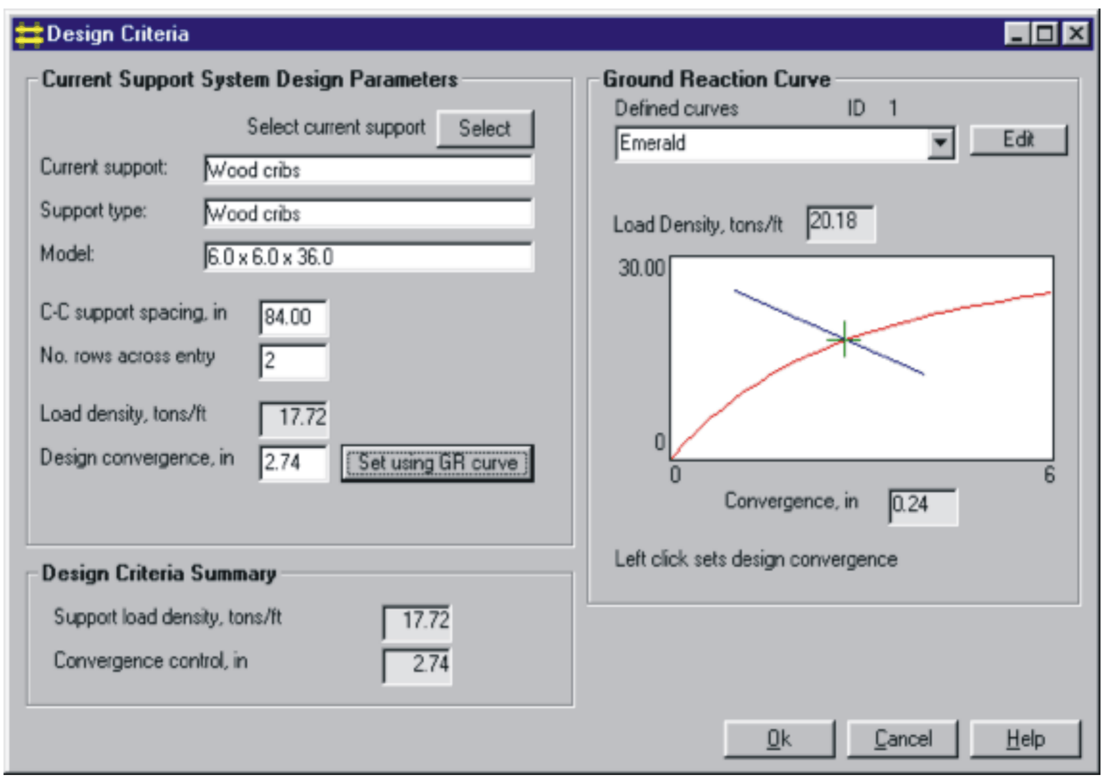


Figure 3.—Window for establishing design criteria based on current roof support system.

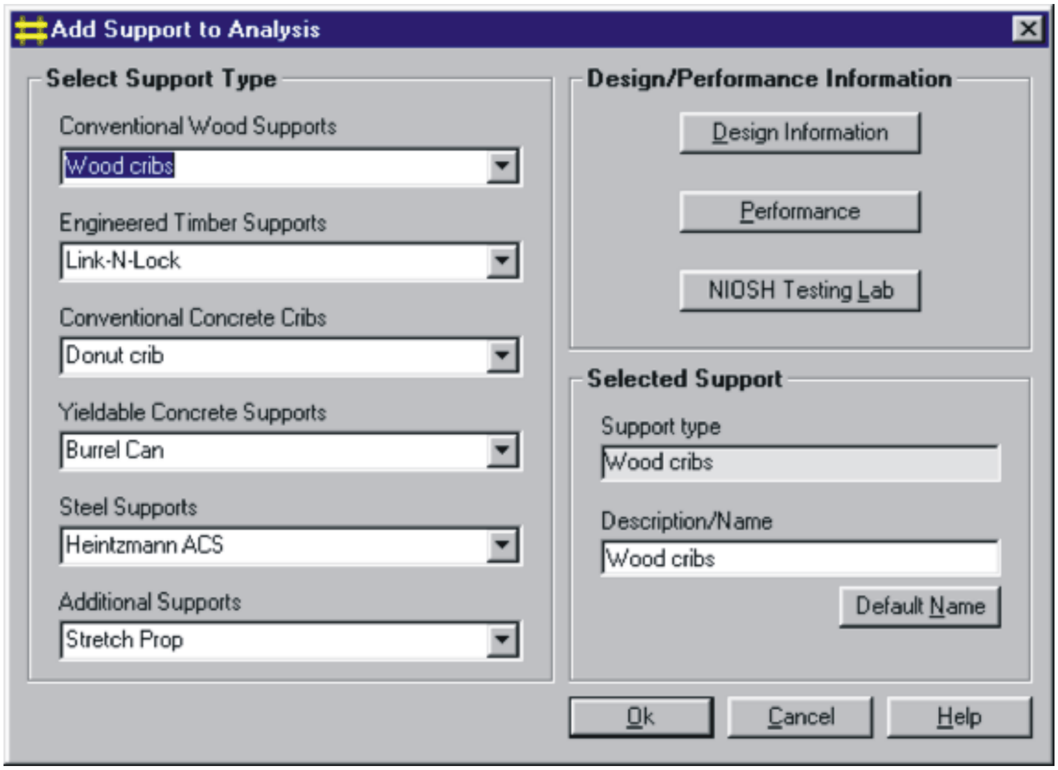


Figure 4.—Window for selecting support types for design and analysis.

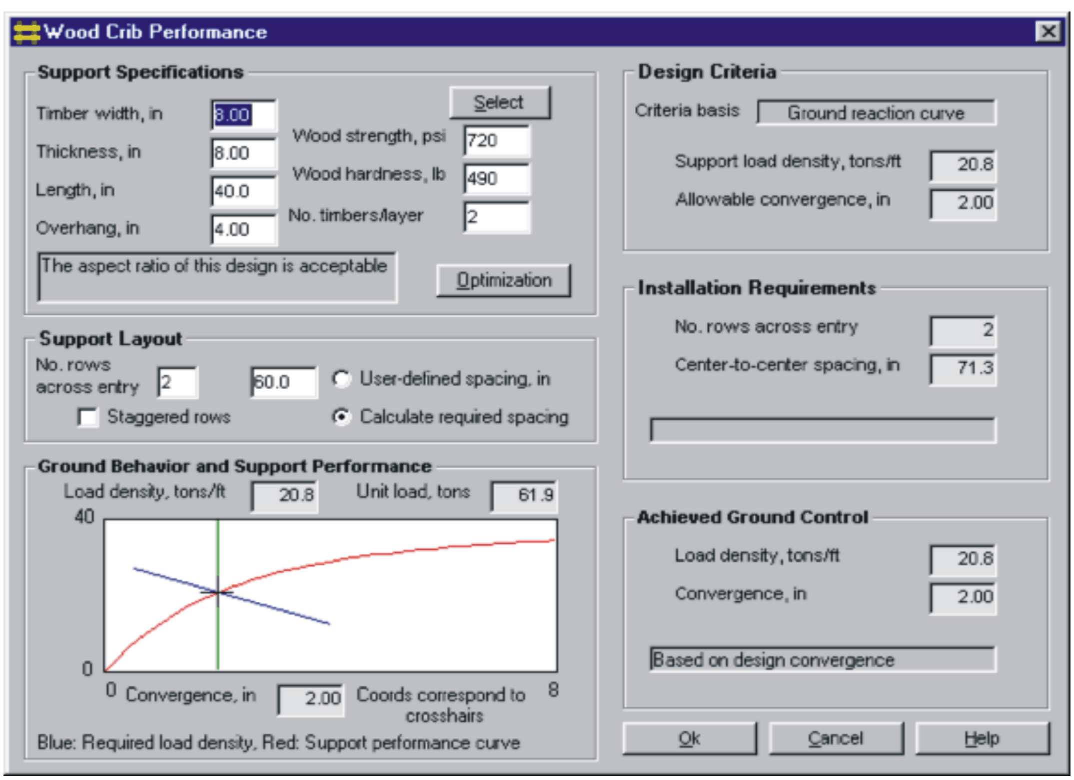


Figure 5.—Window for designing support system and determining installation requirements to achieve design criteria objectives.

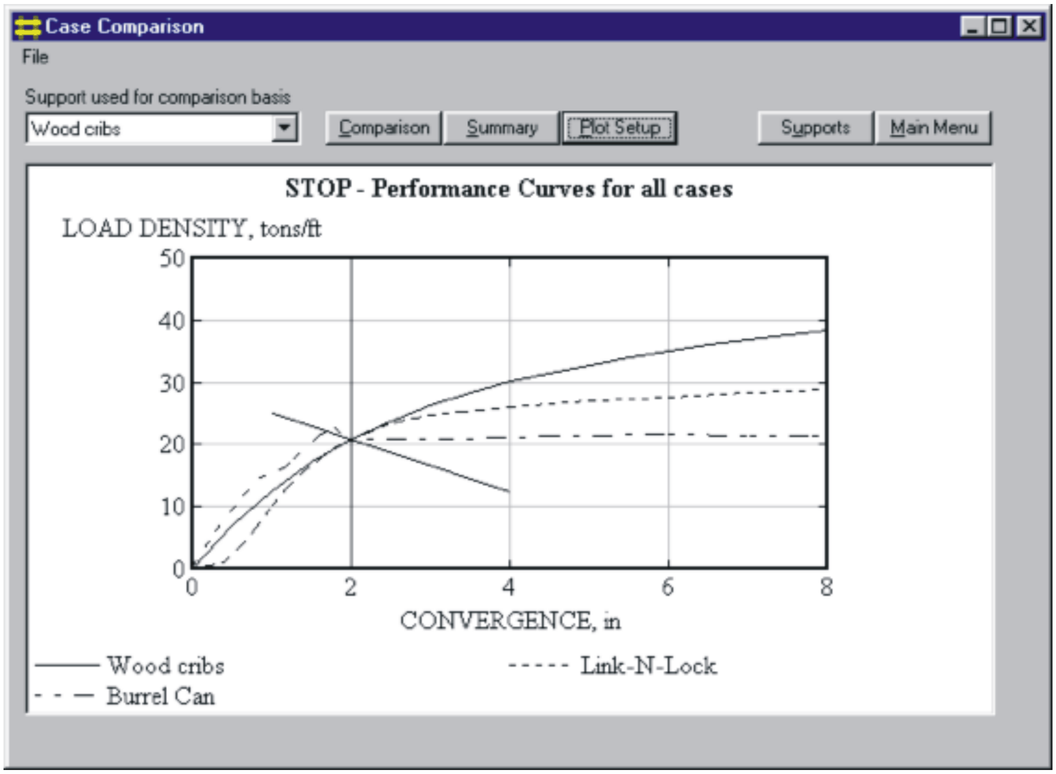


Figure 6.—Graphical analysis of selected support systems showing support load density as a function of convergence.

concrete crib supports, (4) yieldable concrete supports, (5) steel supports, and (6) additional supports. From this list and the embedded submenus, the user can select a specific support and learn more about the support through several other program buttons. *Design Information* provides a description of the support, design and installation considerations, performance characteristics, and manufacturer or supplier contact information. *Performance* displays the loading profile of the support with photographs that depict deformation and associated support loading. *NIOSH Testing Laboratory* describes PRL's Mine Roof Simulator and refers to the safety performance testing protocols through which the performance characteristics of the support were determined. *Reference/Bibliography* contains relevant reference material pertaining to the selected support system.

Help: A context-sensitive *Help* file is available to facilitate operation of the program and interpretation of the results. The *Help* file can be called from each window or from the main menu.

HOW TO USE STOP

Generally, the program control guides the user through a logical sequence of operations to facilitate the design and implementation of a roof support system (figure 7). A General Program Flow window is shown on startup. This window shows the basic program flow and recommends using the *Next* buttons to assist the user in following this recommended procedure for support design and analysis. The *Next* button transfers control to the *Design Criteria* module, where the user must select the basis for establishing the design criteria by choosing one of the following options: (1) ground reaction curve, (2) detached block, (3) current support system, or (4) arbitrary criteria. Control is then transferred to the appropriate window for the chosen design criteria and the user then defines the support load density and convergence design criteria in that window.

Once the design criteria are established, control is transferred to the *Select Supports* window. Here the user picks the supports to evaluate. Several options are available: (1) *Add* allows the user to select a new support and review the NIOSH database on support performance and design considerations, (2) *Delete* deletes a support from consideration, (3) *Duplicate* duplicates the choice of a support, which can be helpful when the user wants to reevaluate a support design with a few minor changes, and (4) *Rename* simply renames the support.

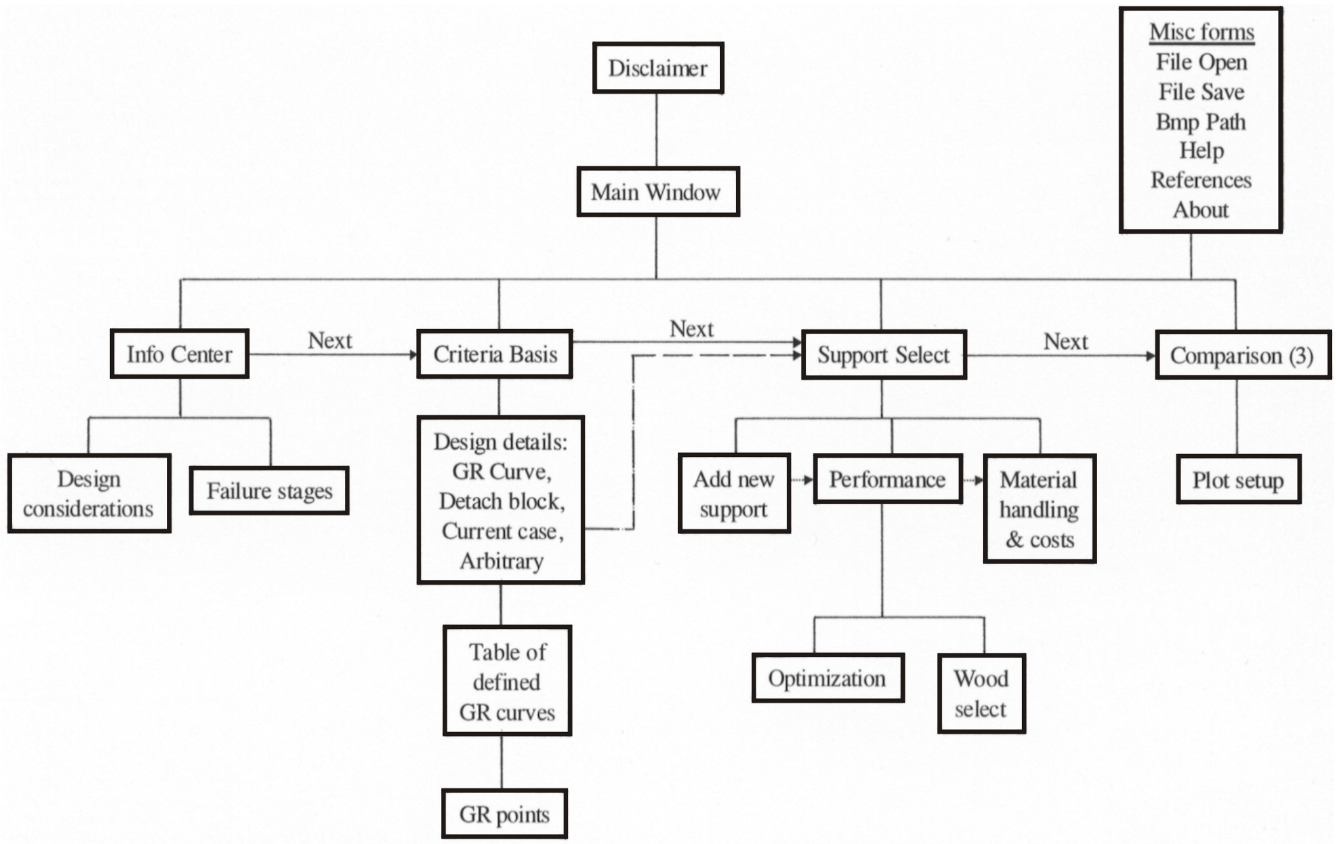


Figure 7.—Flow diagram showing program control.

After a support type is selected, control is transferred to a window where the user can define the appropriate support design parameters. Once the user defines the support design, the program calculates two outputs: (1) *Achieved Ground Control,* where support load density and the convergence control provided by the support system are displayed, and (2) *Installation Requirements*, where the number of rows and required support spacing are provided. For most supports, under *Support Specifications,* there is also an *Optimization* button. Clicking on the *Optimization* button causes the program to transfer to an optimization window where the user can select a support spacing and number of rows and the program will calculate the support model or design that most closely matches the required convergence and support load density previously established in the design criteria. The output of the optimization algorithm depends on which spacing option was selected in the support specifications window. If the "Calculate the Required Spacing" option is selected, the optimization routine will select a support design that will meet the design criteria at less than the specified maximum spacing. If the "User-Defined Spacing" option is selected, the program will determine a support design that will meet the load density requirement at a convergence less than the design convergence.

Once the support system is defined and the installation requirements (number of rows and spacing) are determined, control is transferred to the *materials handling* window where the support costs and material handling requirements can be defined and examined. A set of default values are included in the program that are considered to be representative of the various support technologies at the present time; however, the program allows the user to modify any of these parameters. In particular, the cost parameters may be mine-specific and time-dependent to some degree. These default values will be updated periodically when STOP is eventually placed on the NIOSH Web site (www.cdc.gov/niosh); however, the user should contact the support manufacturers to receive the latest cost information. Finally, the *Next* button transfers control to the *Comparison* module, where the various support systems can be compared to one another.

EXAMPLES OF APPLICATIONS OF STOP

EXAMINING THE LOADING PROFILE OF A SUPPORT SYSTEM

In the *Information Center* of STOP, photographs of each support during its loading phases are shown when the *Performance* button is clicked. The load-displacement curve for the support is shown with a vertical line to designate the displacement that corresponds to the photo in the window. Photos are typically shown at 2-in increments in support displacement. An example for the Propsetter support is shown in figure 8.

OPTIMIZING THE USE OF CONVENTIONAL WOOD CRIBS

Historically, conventional wood cribs have been used extensively for secondary roof support. A common support system is a double row of 4-point cribs constructed from 5- by 6-in (cross-sectional dimensions), 30-in-long, mixed hardwood timbers. STOP can be used to evaluate alternative designs and show that 9-point cribbing can be more cost-effective. The procedure to conduct such an analysis would be as follows:

1. Choose *Current Support Systems* as the basis for selecting the design criteria. Change entry height to 84 in for this example. Since no supports have been defined yet, click on the *Add* Option to transfer control to the *Add Supports to Analysis* window, where wood cribs can be selected and relevant information on the design and performance of wood cribs can be reviewed. *OK* then transfers control to the *Wood Crib Specifications* window.

2. In the *Wood Crib Specifications* window, enter the wood crib specifications (timber width, timber thickness, timber length) and select the mixed hardwood species to establish wood strength. Input the number of timbers per layer. After confirming the support design, *OK* transfers control to the *Select Current Support* window. *OK* then transfers control to the *Design Criteria* window, where the current wood crib design is featured.

3. Enter a value for the spacing of the supports, number of rows, and a convergence to establish the support load density design criteria. In this example, a spacing of 81 in for a double row of cribs and a convergence of 4 in were chosen. *OK* transfers control to the *Select Basis for Design Criteria* window, where the support load density requirements of 10.6 tons/ft and convergence control of 4 in are displayed. When these values are confirmed by pressing *Next*, the *Select Support* window is recalled. The user is required to update this design by activating the *Design and Cost* button before proceeding. When the *Design and Cost* button is pressed, control transfers to the *Wood Crib Performance* window, where the *Installation Requirements* and *Achieved Ground Control Parameters* are displayed.

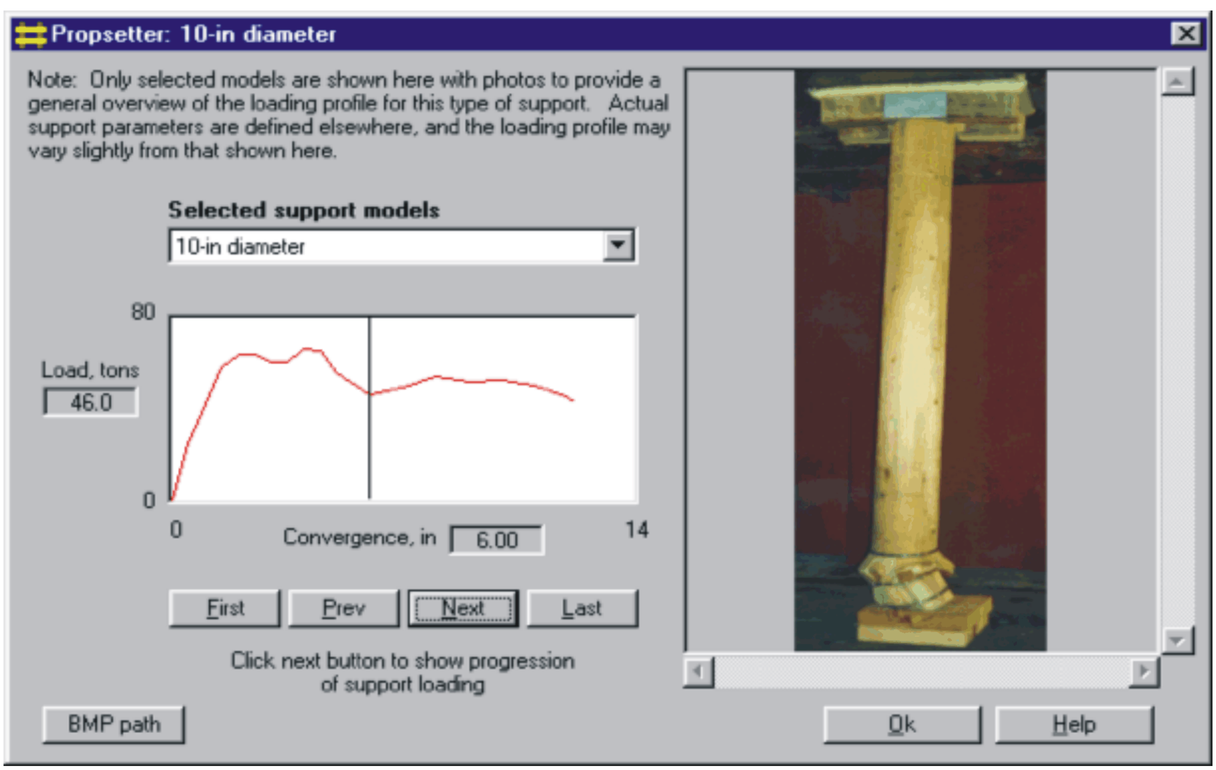


Figure 8.—Example of load profile display available in Information Center module.

4. The support material handling and costs for this crib design are then examined by pressing the *OK* button.

5. When control transfers back to the *Select Supports* window, the 9-point crib can be analyzed. One way to accomplish this is to duplicate the current support (press *Duplicate* button and then the *Design and Cost* button) and simply change the number of timbers per layer from 2 to 3 and the number of rows from 2 to 1. The program will then calculate the required spacing of single row of 9-point cribs that will provide the same load density as that of the double row of 4-point cribs.

Figure 9 documents the result of one such analysis and shows a comparison of the installed cost of a double row of 4-point cribs on an 81-in center-to-center spacing with that of single row of 9-point cribs on a 92-in spacing. Both support systems, using cribs constructed from 5×6×30-in mixed hardwood timbers, provide 10.6 tons/ft of support capacity at 4 in of convergence. Also included in this analysis is a double row of 4-point cribs constructed from all oak timbers instead of mixed hardwoods. Note that in this analysis, the narrow (5-in) side of the timber was placed down to establish the interlayer contact.

Figure 10 illustrates the same comparison, except that the cribs are constructed with the wide (6-in) side down instead of the narrow (5-in) side down, as was done in the previous example. The results clearly show the benefits of maximizing support capacity by increasing the contact area using the wide-side-down construction.

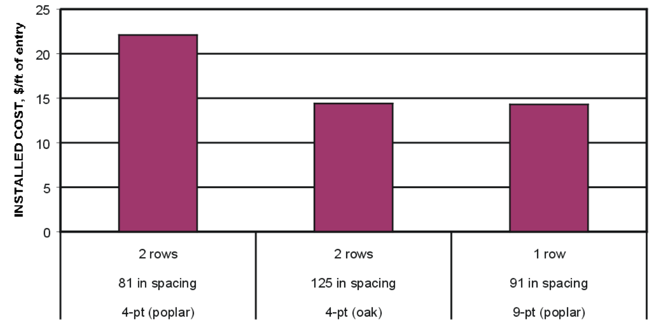


Figure 9.—Comparison of installation costs of various wood crib designs (timbers are 5×6×30 in and placed with 5-in side down).

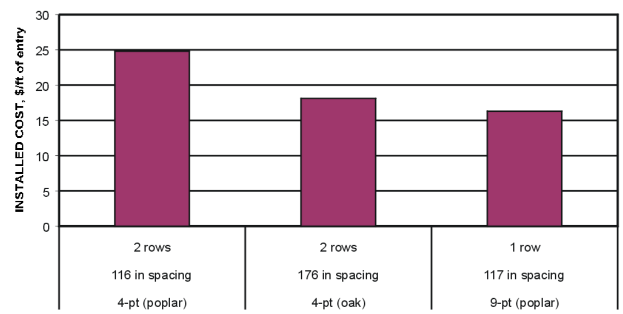


Figure 10.—Same analysis as shown in figure 9 except wide (6-in) side of timber placed down.

REPLACING CONVENTIONAL WOOD CRIBBING WITH ENGINEERED TIMBER SUPPORTS

In recent years, numerous alternative timber supports have been developed. These supports are engineered to provide improved loading characteristics compared to conventional wood cribbing. For this example, the goal is to replace a conventional wood crib design with engineered timber supports and provide equivalent support capability in terms of support load density at a specific convergence. The procedure for designing these engineered timber supports is essentially the same as in the previous example, except that alternative supports are chosen for analysis instead of conventional wood cribs.

The baseline case for this example is the same as that chosen in the previous example: a double row of 4-point, mixed hardwood cribs constructed from 6×6×36-in timbers on 116-in spacing providing 10.52 tons/ft support capacity at 4 in of convergence. The alternative supports chosen for this example were (1) 24-in Link-N-Lock, (2) 30-in Link-N-Lock, (3) Hercules HM-9(308) crib, and (4) 30-in Tri-Log crib. Figure 11 shows the installed support cost per foot of entry for support systems designed by STOP to provide equivalent support loading to that of the conventional wood crib support system chosen as a baseline. The installation requirements are also shown. As seen in figure 11, all four of the engineered timber support systems are able to reduce the installed support cost considerably without sacrificing support capability.

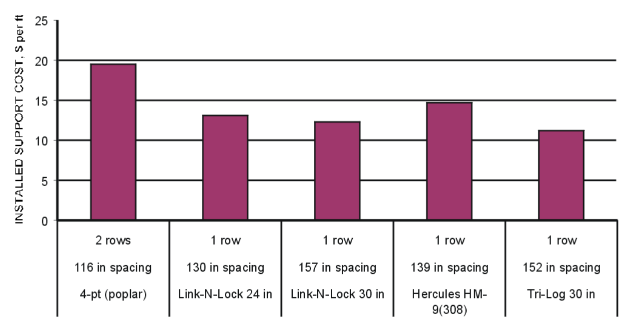


Figure 11.—Evaluation of engineered timber support systems as a replacement for conventional wood cribbing.

INCREASING SUPPORT LOAD DENSITY TO REDUCE ENTRY CONVERGENCE

The objective of increasing support load density is to improve ground control by allowing less roof movement. If the ground reaction at a particular mine is known, then support systems can be designed to provide any measure of convergence desired. The following example (figure 12) is based

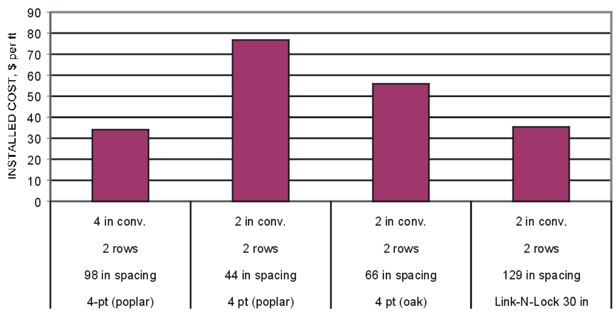


Figure 12.—Analysis of support system options when convergence requirement is reduced from 4 to 2 in.

on a ground reaction curve selected from the program's database. Using this curve, convergence is reduced from 4 in, achieved with two rows of conventional 4-point, mixed hardwood cribs on an 87-in spacing, to 2 in with alternative supports.

1. Choose *Ground Reaction Curve* as the basis for selecting the design criteria. Change the default entry height from 96 to 84 in. In the Ground Reaction form, the Emerald ground reaction curve is chosen. Input 4 in for the design convergence, and the program will determine the required support load density consistent with the ground reaction curve. In this case, it is 12.50 tons/ft.

2. Control will then be transferred to the *Select Supports* window. Pressing the *Add* button brings up the *Add Support to Analysis* window. Choose wood cribs as the support type.

3. Control will then be transferred to the *Wood Crib Performance* window, where parameters for a 4-point wood crib are defined (6×6×36-in timbers, mixed hardwood species, two rows, two timbers per layer). The program will then determine the installation requirements for this crib design as a 98.5-in spacing of the support. The program proceeds to the *Costs and Material Handling-Wood Cribs* window. Review the default settings for wood crib. The program then computes an installed cost per foot of entry for this support design.

4. Control is then transferred to the *Case Comparison* module. Review the performance parameters for this support system.

5. The *Comparison* window is then closed by transferring control back to the *Main Menu*. Activate *Design Criteria* and edit the *Ground Reaction Curve* design criteria (Emerald Mine) by changing the design convergence to 2 in. A warning message will be displayed, which indicates that the design criteria have changed. The 4-point wood crib system must then be updated by pressing the *Design and Cost* button. The installation requirements for this crib support system are changed to 44 in for the new design criteria.

6. Control will then be transferred back to the *Select Supports* window. Select one of the various alternative support technologies for analysis and enter the appropriate design parameters in the *Performance* window.

7. When the support designs are completed, the *Case Comparison* window will show baseline wood crib performance at 4 in of convergence and the alternative support performance at 2 in of convergence.

REPLACING TIMBER SUPPORTS WITH OTHER ALTERNATIVES

There are several alternatives to timber supports. STOP can be used to evaluate these various alternatives and make comparisons based on equivalent support capability or show the advantages of alternative placement strategies with superior roof support systems. The following example shows how these alternative support technologies can be designed relative to the current roof support system using available ground reaction data.

The process begins by selecting the *Current Support System* for design basis. The entry height is left at the default setting of 96 in. In this example, the current roof support system is a double row of 4-point wood cribs. Thus, the user selects wood cribs as the support type and enters the appropriate data in the *Wood Crib Specifications* form to define a 4-point crib constructed from 6×6×36-in timbers oak timbers. The center-to-center support spacing (108 inches in this case) and the number of support rows (two in this case) are entered. Design convergence and support load density are determined by clicking on the *Set Using GR Curve*, where it is shown that the current support system intersects the chosen ground reaction curve at 3.34 in of displacement and provides a support load density of 15.25 tons/ft. Table 1 compares several alternative support systems that provide equivalent or improved support capability. It is noted that with some supports, the support may shed load prior to the design convergence. The program logic is set to use the design convergence, but the user can determine from the ground behavior and support performance curves the convergence at which these supports will provide the required load density (see example in table 1).

USING OPTIMIZATION ROUTINES TO SELECT BEST SUPPORT DESIGN

The previous examples have shown how STOP determines the required support spacing needed for a user-specified support design. The optimization routines allow the user to specify support spacing and number of rows of support elements, and the program will determine which support design best fits the load and convergence design criteria. In the example shown in table 2, the design criteria of 16.67 tons/ft of entry at 3 in of

Table 1.—Comparison of alternative support technologies as replacements for conventional wood cribbing

Support type	Design specifics	Installation requirements: No. of rows	Installation requirements: Spacing	Achieved convergence control, in	Achieved support load density, tons/ft
Wood cribs	4-point (6×6×36-in oak timbers)	2	108	3.34	15.2
Pumpable crib	30 in	1	73	3.34 [1](0.24)	15.2
ACS	Pizza headplate	2	92	3.34 [1](1.51)	15.2
Can	24-in-diam	2	138	3.34	15.2
Stretch prop	Timber ft/hd boards	3	61	3.34	15.2

NOTE: Design requirements: support load density = 15.25 tons/ft; convergence = 3.4 in.

[1]The required support load density of 15.2 can also be achieved at less displacement since the support sheds load prior to the design convergence of 3.34 in. Using the mouse coordinates on the ground behavior and support performance curve in the appropriate support design window, the convergence that produces the required 15.2 tons/ft of loading can be determined.

Table 2.—Support systems determined by the optimization routine for user-defined support installation parameters

Support type	Optimized design	User-specified installation requirements: No. of rows	User-specified installation requirements: Spacing	Achieved convergence control, in	Achieved support load density, tons/ft
Wood cribs	9-point (5×5×30-in timbers)	2	96	2.8	17.6
Link-N-Lock	36 in	1	96	2.6	18.5
Tri-Log cribs	30-in standard	2	96	1.5	23.0
Propsetter	10-in-diam	2	96	2.6	18.4
Can	24 in	2	96	1.6	22.5

NOTE: Design requirements: support load density = 16.67 tons/ft; convergence = 3.0 in.

convergence were established from a ground reaction curve chosen from the program database. A 96-in spacing was chosen for the analysis by entering 96 inches in the *Support Layout* section for the user-defined spacing option. Then various support types can be selected for evaluation. When the *Optimization* button in the *Performance* form is selected, the user will define the installation requirements (number of rows and support spacing), and the program will determine the support model that most closely matches the design criteria (16.67 tons/ft at 3 in of convergence). Since the installation spacing and number of rows are specified, the achieved convergence will vary depending on the support type chosen. Table 2 documents some examples of optimized supports as determined by STOP.

EVALUATING MATERIAL HANDLING ASPECTS OF SUPPORT DESIGN

Surveillance data show that material handling injuries are common in support construction, resulting in several thousand lost workdays each year. Thus, part of the support selection process should be material handling requirements. Figure 13 is an example of data derived from STOP for four support systems: (1) pumpable crib, (2) Alternative Crib Support (ACS), (3) Propsetter, and (4) conventional 4-point crib. As seen from this analysis (figure 13), there are significant material handling advantages in using the alternative support technologies instead of conventional wood cribbing.

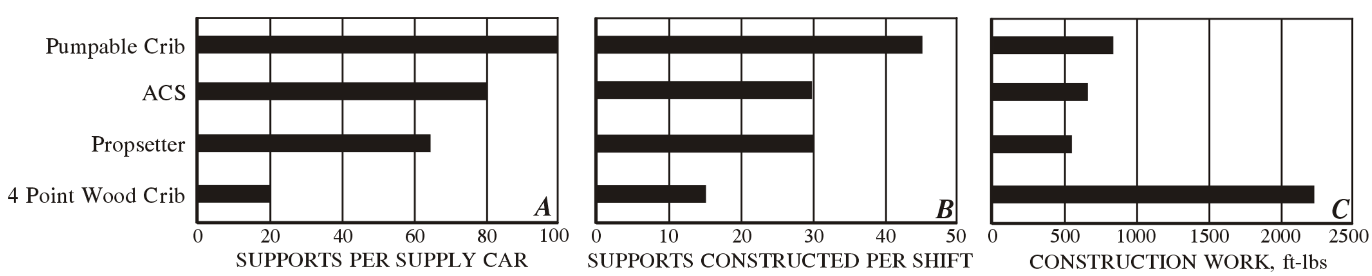


Figure 13.—Comparison of conventional wood supports with three alternative support systems. *A*, Number of supports per supply car; *B*, number of supports constructed per shift; and *C*, amount of construction work in foot-pounds.

CONCLUSIONS

Historically, wood cribs have been used as secondary roof support in underground mines. The Wood Crib Performance Model was developed in 1994 to provide an engineering foundation for the design and applications of these supports. In the past few years, many new support technologies have been developed by various roof support manufacturers. In many cases, these innovative support technologies provide superior roof support and reduced material handling requirements. However, each has its own performance characteristics; thus, they all need to be employed differently to provide equivalent roof support.

STOP was developed to provide a more comprehensive support design program than that provided by the original Wood Crib Performance Model. Not only does STOP include new support technologies, it also allows for application of a new design methodology based on a measured ground reaction curve at a particular mine site. Also included in STOP are a comprehensive material handling assessment and cost evaluation for each support system. STOP is a Windows-based program that is user-friendly and very flexible and provides engineering solutions for various secondary support applications.

STOP can be used to determine installation requirements for an alternative support technology that will provide equivalent support compared to a mine's current support system or an installation that will provide a specified support load density at a designated roof convergence. The optimization routines in the program will select the most efficient support design for the user-specified criteria.

STOP uses performance data developed by NIOSH through safety performance testing in the Mine Roof Simulator. Each support system has been evaluated through a rigorous testing protocol that simulates in-mine service conditions. Photographs of support conditions at various stages of loading are also included in STOP. These photos help the user gain an understanding of the limitations of the support and can be used to assess general loading conditions when these profiles are observed underground.

STOP can provide some much-needed engineering for secondary roof supports. This can be very helpful when petitioning MSHA for approval to use an alternative support technology or changing applications, such as increasing support spacing. By proper engineering of the support relative to ground reaction, convergence can be controlled to a predetermined level. This will allow an operator to optimize the support application and provide a margin of safety in roof stability that will reduce the likelihood of roof failure without the need for excessive roof support. Likewise, proper engineering will remove uncertainty in support design and prevent the application of inadequate support that can lead to roof falls. Finally, STOP will allow mine operators to consider fully the material handling aspects of support design in the selection process, thereby reducing the incidence and severity of material handling injuries.

Copies of the STOP software program can be obtained from Thomas M. Barczak, NIOSH Pittsburgh Research Laboratory, P.O. Box 18070, Pittsburgh, PA 15236-0070; phone: (412) 386-6557; e-mail: TBarczak@cdc.gov. It is also anticipated that STOP will eventually be available through the NIOSH Web site (www.cdc.gov/niosh).

ACKNOWLEDGMENT

The author acknowledges Dave Conover, NSA Engineering, Inc., who developed the Windows-based program for STOP. The STOP architecture evolved over a year of effort through several painstaking revisions to form the final product. It is to Mr. Conover's credit that this program was completed within the budget constraints of the project and with such a high quality of workmanship.

REFERENCES

Barczak TM, Gearhart DF [1993]. Engineering methods for the design and employment of wood cribs. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9361.

Barczak TM, Mucho TP, Dolinar DR, Bryja J, Bower J [1999]. Longwall tailgate support, consideration, design, and experience. In: Proceedings of Longwall U.S.A. Pittsburgh, PA, pp. 79-105.

Mark C [1992]. Analysis of longwall pillar stability (ALPS): an update. In: Proceedings of the Workshop on Coal Pillar Mechanics and Design. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9315, pp. 238-249.

Mark C, Chase FE [1997]. Analysis of retreat mining pillar stability (ARMPS). In: Proceedings—New Technology for Ground Control in Retreat Mining. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 97-122, IC 9446, pp. 17-34.

Mucho TP, Barczak TM, Dolinar DR, Bower J, Bryja JJ [1999]. Design methodology for standing secondary roof support in longwall tailgates. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 136-148.

Unal E [1986]. Empirical approach to calculate rock loads in coal mine roadways. In: Peng SS, ed. Proceedings of the Fifth International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 234-241.

CABLE SUPPORT IN LONGWALL GATE ROADS

By Dennis R. Dolinar[1] and Lewis A. Martin[2]

ABSTRACT

Cable bolt technology used by the U.S. coal industry was developed to a large extent in the 1990s. Today, these cable systems include both cable bolts and cable trusses to provide supplemental and secondary support in gate roads. This cable technology is significantly different than the cable systems in use in either U.S. hard-rock mines or Australian coal mines. Development of this technology was initiated and spurred by research efforts of the U.S. Bureau of Mines, and has continued under the health and safety programs of the National Institute for Occupational Safety and Health (NIOSH). It was also followed up by work of roof support manufacturers to create an essentially new support system. In this paper, the important support characteristics of both cable bolts and cable trusses are discussed. The design of cable systems and the basis for that design for tailgate and headgate situations are reviewed and explained. Case histories are presented on the application of these support systems based on experience gained at a number of in situ test sites.

[1]Mining engineer, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.
[2]Mechanical engineer, Spokane Research Laboratory, National Institute for Occupational Safety and Health, Spokane, WA.

INTRODUCTION

Cable bolting was introduced into the U.S. coal industry in 1992 through the research efforts of the U.S. Bureau of Mines. This original work was conducted at a longwall operation in western Colorado for the purpose of finding alternative secondary tailgate support [Tadolini and Koch 1993]. For the initial test, cable design and installation were based on cable technology developed for the hard-rock mine industry [Goris et al. 1993; Goris 1990; Goris 1991; Goris et al. 1994]. This involved the installation of the cables by hand and using a fully grouted cable with a pumpable grout system. This technology was certainly adequate for the hard-rock industry, but not for a high-production longwall. Initially, a crew could install 8 to 12 cables per shift, but with experience this number increased to 30 cables per shift.

From this beginning, the cable bolt system used today quickly evolved until an essentially new product was created. The result was a headed cable bolt installed by machine and anchored with a resin grout cartridge and a partial grout column [McDonnell et al. 1995]. Furthermore, resin manufacturers developed special resins for cable bolt anchors. With these new cable bolt systems, it is now possible to install up to 70 or more cables per shift with a double-boom bolter.

A driving force behind the development and use of cable bolts as secondary support in the tailgate was to replace wood cribs. Wood cribs, especially in the West, were becoming expensive because of a shortage of timber and a lessening of timber quality. A typical longwall gate road in the West supported by wood cribs requires about 248 acres of timber [Tadolini and Koch 1993]. Cutting this amount of timber has an environmental impact. The installation of wood cribs is labor intensive and materials handling is a significant problem, not only from a logistic consideration, but also from an injury standpoint. Especially in the West, where seam are high, a large number of injuries result from crib installation. The high density of the wood cribs necessary in a gate road restricts ventilation and impedes the use of the tailgate as an escapeway [Kadnuck 1994]. All of these considerations can put constraints on the development of super longwall systems.

From a ground control standpoint, wood cribs are far from ideal. Four-point wood cribs are regarded as a soft support system. Improperly built cribs can result in a wide variation in the performance of individual cribs, thus exacerbating ground conditions. In many western mines, yield pillars are used, and by design the tailgate will be subjected to large amounts of deformation, much of which is the result of roof-to-floor convergence. Yet crib systems will resist this deformation, taking up a large portion of crib capacity. Wood cribs also have problems handling the large lateral movements associated with yield pillar designs.

Today, cable bolts are competing against other newly developed types of standing support developed to replace the wood cribs [Barczak et al. 1996; Mucho et al. 1999]. These new systems are definite improvements over wood cribs. Still, from a ground control standpoint these standing support systems use a significant portion of their capacity to resist main roof-to-floor convergence and limit full access by equipment to the face.

Cable bolts are used not only as tailgate support but also for support in bleeders [Tadolini et al. 1993; McDonnell et al. 1995]. They are used as supplemental support, not only in longwall operations, but also in room-and-pillar mines. There are also efforts to adapt cable bolts as primary support because they can be installed so they are much longer than the height of the opening, a factor that could improve roof support in thin-seam mines. Some consideration is also being given to using a single-pass system in gate roads, where the cable bolts will be used both as the primary support for development and secondary support when the panel is mined.

The Australians had used cable bolts in their coal mines as both supplemental and secondary support [Gale 1987; Gale et al. 1987]. The cables were normally 10 m (33 ft) long, fully grouted, and installed with a pumpable grout system. Because the Australians used a two-entry system with large abutment pillars and 60 to 120 m (200 to 400 ft) between entries, the design and use of their cable system offered little that could be adapted to use in a U.S. longwall tailgate.

The development of cable bolts also spurred the development and use of cable trusses. Cable slings had been used on a limited basis since the 1970s [Mangelsdorf 1982; Scott 1989]. However, anchoring the cables was a problem until a system was devised that used anchorage systems based on a resin cartridge inserted into the drill hole. These truss systems can be installed with the assistance of a roof bolting machine, though installation can still be accomplished in the traditional manner with small drills. Cable trusses are now used as supplemental support in headgate entries, especially where horizontal stress causes damage. Cable trusses have been used in open entry recovery rooms and to a limited extent, as secondary support in tailgates.

This paper will discuss cable bolts and trusses and the design of cable systems for the support of longwall gate road entries.

CABLE BOLT

Cable bolts are made from a high-strength steel cable. The most common cable used is seven strands 1.52 to 1.59 cm (0.6 to 0.625 in) in diameter (Goris et al. 1994; McDonnell et al. 1995). The cable consists of six outer strands wrapped around a middle or king wire strand (figure 1). The cross-sectional area of the steel for the cable is 0.55 cm^2 (0.217 in^2). Cable bolts can be of any length, but typically range from 2.4 to 6.1 m (8 to 20 ft) for use in a coal mine. The cables bolts are anchored in the roof with resin grout cartridges using only a partial grout column. This leaves a free cable length in the lower portion of the hole. Cable diameters range from 1.27 to 2.29 cm (0.5 to 0.9 in), but only the properties of the 1.52-cm (0.6-in) in diameter cable bolts will be discussed in the following sections.

Figure 2 shows the components of a typical cable bolt. A cable bolt consists of a cable head that ties the cable strands together and allows the bolt to be installed and rotated with a roof bolter. For ground control, the head is necessary for the ungrouted portion of the cable to take load and resist rock movement. The head also permits the installation of bearing plates and other surface control devices. A barrel-and-wedge system is used to attach the head on the cable.

A stiffener is necessary to install the cable bolt and insert it through the resin cartridge with a roof bolter. Without the stiffener, the cable is too flexible to be pushed through the resin cartridge and will bend outside the hole. If possible, the stiffener should be long enough to be in the hole before the cartridge is punctured by the cable and yet short enough to be installed at a given mining height. A stiffener that is as long as the resin cartridge will allow this to occur. Another function of the stiffener is to prevent the cable from being nicked by the bearing plate during installation, which reduces the potential for corrosion of the cable.

To assist in anchoring the cables and mixing the resin, anchor buttons, "birdcages," nut cases, or bulbs are used in the upper or anchor portion of the cable. These systems are designed for specific hole diameters, usually for holes 2.5 to 3.5 cm (1 to 1-3/8-in) in diameter. An end button holds the cable end together and assists in inserting the cable through the resin. A resin keeper or dam keeps the resin confined along the section of cable to be anchored and also compresses it. The necessity of the resin keeper will depend on resin viscosity, hole diameter, and cable bolt design.

Other designs allow the cables to be tensioned. This is usually done at the head of the bolt through the wedge-and-barrel-head or by a threaded bar attached to the end of the cable. Tensioning the head and wedge is done by hand while the threaded rebar can be tensioned by the roof bolter. Cables with yieldable heads are available where large roof deformation is expected, and loads will exceed cable strength [Tadolini and McDonnell 1998; Vandekraats et al. 1996].

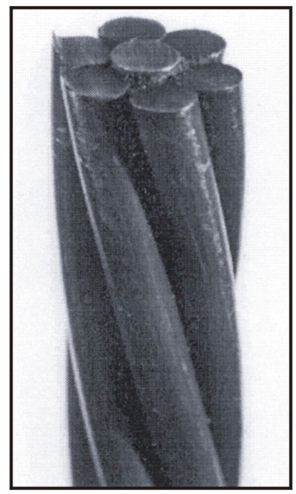


Figure 1.–Seven strand steel cable used to make cable bolts.

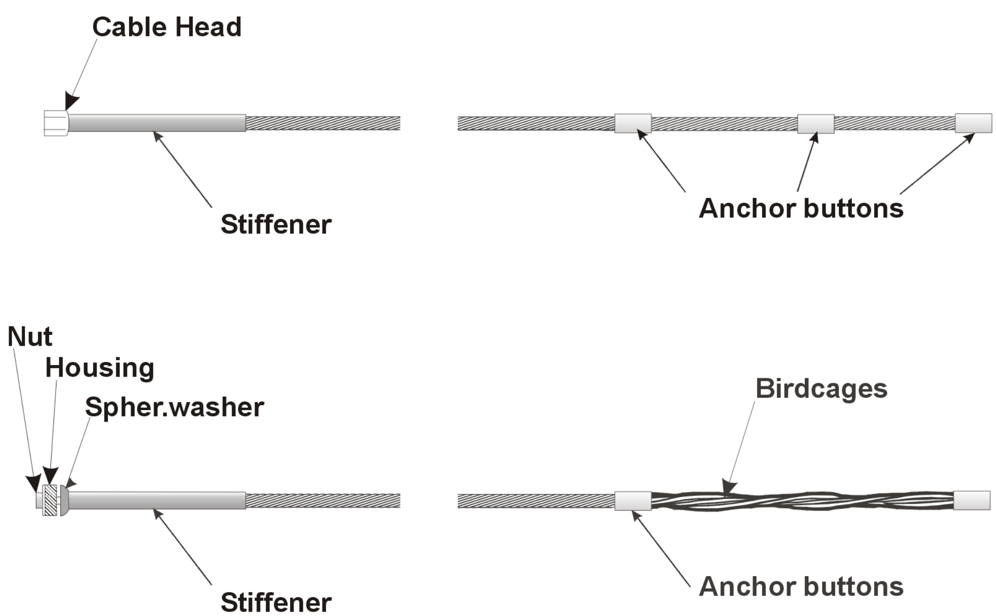


Figure 2.–Cable bolt and cable bolt components.

CABLE SYSTEM CHARACTERISTICS

The cable system consists of a cable bolt with a ungrouted or free length of cable and a resin grout anchor. The bearing plate and other surface control devices held in place by the cable bolt are also part of the system. The performance of the cable support will depend on how well these components act together as a system. Figure 3 shows a cable system installed in a tailgate. In general, the cable should be the weakest part of the system where the other components should be designed to reach the ultimate strength of the cable. This includes the head, anchor, and bearing plate. An exception is the specially designed cable head that allow for controlled yield below cable capacity.

Figure 3.–Cable bolt system installed in a tailgate.

CABLE BOLT CAPACITY

Cable ultimate strength will usually be between 244.7 and 266.6 kN (55,000 and 60,000 lbf) and will normally exceed 260 kN (58,600 lbf), while elongation of the cable at failure can range from 3.5% to 8%. Cables will begin to yield at about 1% strain. Figure 4 shows a typical test for a cable conducted in the laboratory where ultimate strength exceeded 260 kN (58,600 lbf). The load deformation curve from an underground pull test of a resin-anchored cable bolt is shown in figure 5. In this pull test, the cable length was 0.3 m (10 ft) and the grout anchor 0.9 m (3 ft). A maximum load of 268.7 kN (30.2 tons) was achieved with the cable failing during the test, indicating that the anchorage exceeded cable strength. In this case, two strands were broken on the cable, which is typical failure for a cable. When a cable breaks, there is a sudden, drop in load, often to near zero. This is then followed by some load recovery, but this is limited by the strength of the remaining strands and can be highly variable. The load will drop again when the remaining intact strands begin to break. Essentially, the final residual strength of the cable will be zero although the cable will have some intermediate residual strength. From pull tests, the elongation at failure is usually less than 4% strain [Barczak et al. 1996].

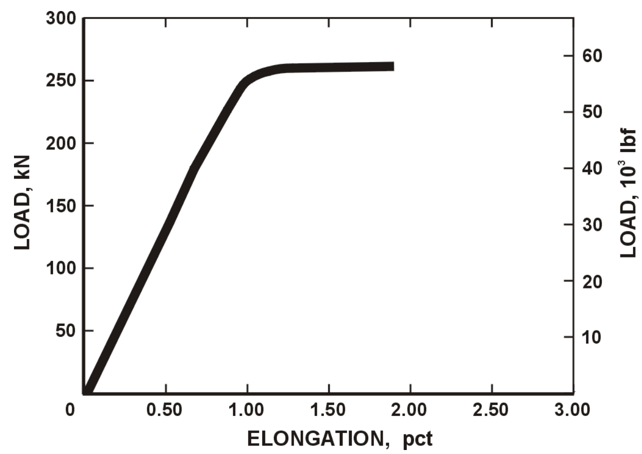


Figure 4.–Load-deformation curve from a laboratory tensile test of a cable.

CABLE SYSTEM STIFFNESS

The stiffness of a cable bolt will be determined by the free cable length in the hole and the elongation properties of the cable. However, elongation in the resin-anchored section of the cable will influence stiffness and must also be considered. The deformation properties of a cable consist of three components-a construction, elastic, and rotational elongation. Construction stiffness is permanent but is usually small. The rotational component is due to the rotation of the cable about the axis during a test or as the cable is loaded. The elastic component is dependent in part on the elastic modulus of the steel composing the cable. The elastic modulus of the steel is 203.4 GPa (29.5 million lbf/in^2). However, the elastic modulus of the cable is

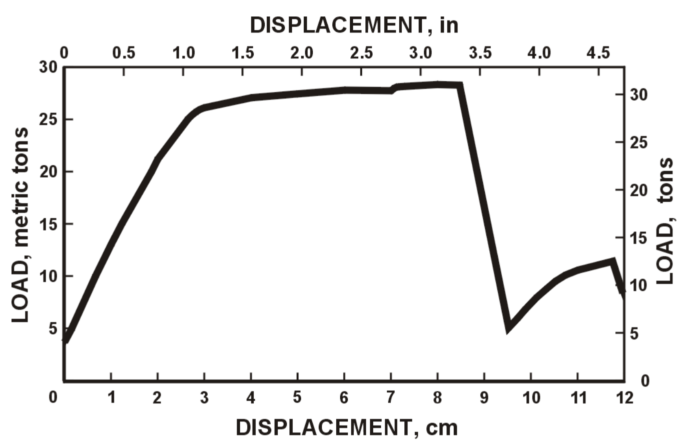


Figure 5.–Load-deformation curve from a pull test of a cable bolt underground.

also dependent on the construction of the cable, which involves the lay length. The lay length is the distance one strand takes to make a complete revolution around the cable [Maryland and American Iron 1985]. The stiffness can be calculated from the following equations.

$$K = E \times A / (L) \tag{1}$$

where K = stiffness, kN/cm (lbf /in),

A = area of cable, cm^2 (in^2),

L = free cable length, cm (in),

and E = elastic modulus of the cable, GPa (lbf/in^2).

Knowing the elastic modulus, length, and area, the cable stiffness can be calculated for a given load. Cable stiffness has been measured underground using a pull test on cables installed in a limestone roof [Zelanko et al. 1995]. For a 3-m-(10-ft-long) cable bolt with a 1.5-m (5-ft) resin anchor and therefore a 1.5-m (5-ft) free length of cable, the initial cable stiffness below the system yield was 106 kN/cm (30.4 tons/in) for cables installed in a 2.5-cm (1-in) in diameter hole, and 98 kN/cm (28 tons/in) for a cable installed in 3.5-cm (1-3/8-in) in diameter hole.

Based on these stiffness values, the deformation modulus of the cables can be calculated from the stiffness equations. However, a correction must be applied to the free length of cable to allow for elongation of the cable in the anchor. From the load transfer characteristics and distances determined experimentally for grouted rebar, the elongation of the cable in the anchor can be approximated by an additional 20 cm (8 in) of free cable length [Serbousek et al. 1987]. Although the anchors will affect the load transfer, any error in determining the additional free cable length from cable stretch in the anchor portion of the cable will have only a small effect on stiffness calculations. From the above test results, for the 106-kN/cm (30.4-tons/in) stiffness, a free cable length 1.72 m (5 ft 8 in) , and an area of 0.55 cm^2 (0.217 in^2), the calculated cable modulus is 132 GPa (19.1 million lbf/in^2).

Using this calculated elastic modulus and the stiffness equation, the stiffness of cables bolts with different free lengths can be determined. For a 4.3-m (14-ft) cable with a 1.2-m (4-ft) anchor and 3.0 m (10 ft) of free cable length, cable stiffness would be 56.4 kN/cm (16.1 ton/in). The assumption is made that the anchor has sufficient length where the anchor will not slip and a portion of the anchor will have little or no load below the yield of the system.

The stiffness of the support will determine how quickly the support will develop resistance and load as the roof deforms. The cable bolt stiffness can be compared to the stiffness of other support systems. For a 1.5-m (5-ft), long No. 6, fully grouted rebar bolt, the stiffness is 700 kN/cm (200 tons/in), and for a 1.8-m (6-ft) long, 1.9-cm (3/4-in) in diameter point-anchor system , the stiffness is 175 kN/cm (50 tons/in) [Karabin and Hoch 1979].

Cable bolts have much less stiffness than most primary support systems. Although the cables are more flexible, the lower stiffness indicates that they will not resist movement as much as other primary support for a given load. For secondary support, a four-point poplar wood crib is 1.8 m (6 ft) high will have a stiffness of 75.3 kN/cm (21.5 tons/in) [Mucho et al. 1999]. This is equivalent to a cable bolt with a 2.3 m (7.5 ft) free length. However, in a tailgate support system, at least two or three cable bolts would be used in place of a single crib. In this case, the cable system would be two or three times stiffer than a crib.

SHEAR CHARACTERISTICS

Resistance to shear and lateral movement can be developed along the free length of the cable and result in cable loading. To determine the shear characteristics of the cables, a series of laboratory tests were conducted where both ungrouted and grouted cables were installed across the block boundary in pairs of concrete blocks [Goris et al. 1995; Goris et al. 1996]. The blocks were sheared parallel to the contact surface and perpendicular to the installed cable bolt. The results showed that the initial peak shear strength was not changed, but that the residual shear strength at 3.8 cm (1.5 in) of displacement was doubled (figure 6). The cables were not immediately activated, but required about 1.0 cm (0.4 in) of displacement before resisting the shear for a 3.5-cm (1-3/8 in) in diameter hole and about 1.52 cm (0.6 in) before significant resistance occurred. Essentially, shear is resisted only when sufficient movement has occurred and the roof has already been mobilized.

In this series of tests, the maximum lateral displacement on the cables was about 3.8 cm (1.5 in). None of the cables failed as a result of this level of displacement. At this point, the cables were loaded to about 60 kN (13,500 lbf), still well below the ultimate strength. However, in a field study, where about 0.3 to 0.46 m (1 to 1.5 ft) of lateral movement occurred across the entry, several cable bolts had failed. It is estimated that the cable bolts failed at between 5 to 10 cm (2 to 4 in) of lateral movement [Dolinar et al. 1996]. The failures occurred from a combination of shear and tension.

RESIN ANCHOR

Several factors influence the effectiveness of the resin anchor, including anchor length, type of resin, and hole diameter [Zelanko et al. 1995]. Figure 7 shows a cross section of a cable installed in resin. The cables will transfer the load through the anchor to the rock with all the load transfer taking place within the anchor length. However, if the anchor is too short, the anchor could slip (rock-grout interface failure), especially in weak rock. With a longer anchor, the anchor

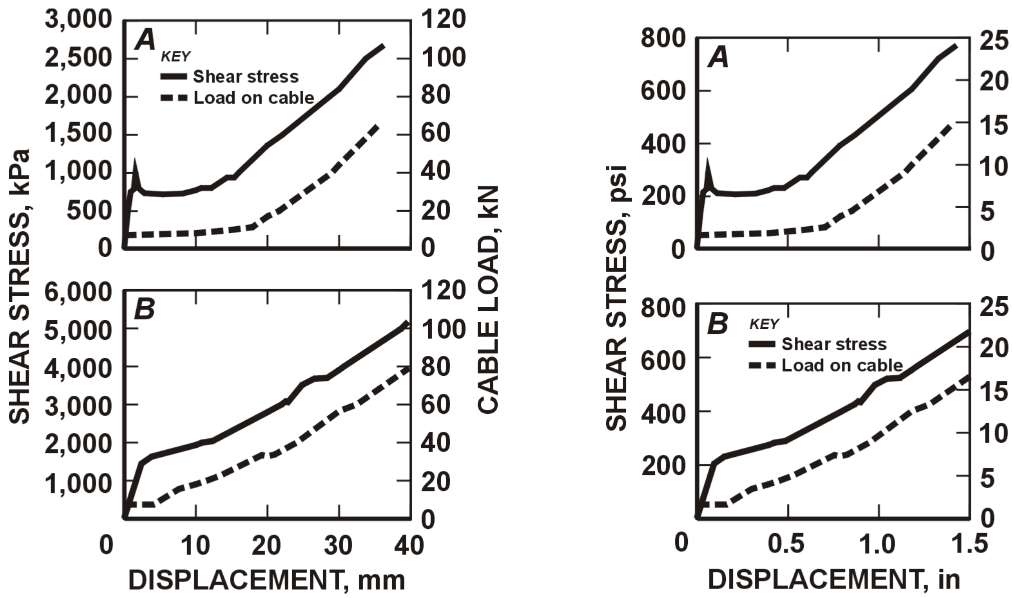


Figure 6.–Typical cable load and shear stress versus shear displacement A, smooth joint with cable bolt reinforcement and B, rough joint with cable bolt reinforcement.

could still fail along the grout-cable interface as the cable is loaded and yields where the anchorage will not exceed the strength of the cable. The anchor failure mechanisms are discussed in more detail elsewhere [Goris 1990; Goris 1991].

An important aspect to the development of an adequate anchorage is the addition of buttons, nut cases, garford bulbs, or birdcages to the anchor portion of the cable. During cable installation, these anchor components assist in mixing the resin, which should provide for an improved quality and consistency of the resin anchor, especially in the larger-diameter holes (3.5 cm [1-3/8 in]). Further, laboratory pull tests on short (76.2 cm [30 in and less) column grout anchors have shown that anchor components embedded in the grout significantly increase anchorage capacity over that of a cable without embedded anchors [Goris 1990, 1991]. With an increase in the resin column length to 0.9 m (3 ft), the conventional cable without anchors can achieve the ultimate strength of the cable, although test results are somewhat inconsistent. In laboratory pull tests, only 60% of the tested cables reached the ultimate cable strength while 40% of the cable anchors failed at significantly lower loads [Martin et al. 1996a]. Without the addition of anchor components embedded in the resin, there is a high probability that the cable anchor will be significantly weaker than the cable.

Both laboratory and field investigations using pull tests have shown that 1.2 and 1.5 m (4 and 5 ft) of resin anchor will achieve the ultimate capacity of the cable if properly grouted [Martin et al. 1996a; Zelanko et al. 1995]. Although laboratory tests have shown that a length of 0.9 m (3 ft) or less of anchor can result in the cable reaching ultimate strength, this was not achieved on a consistent basis. In the elastic range, below the yield of the cable, most of the load is transferred within the first

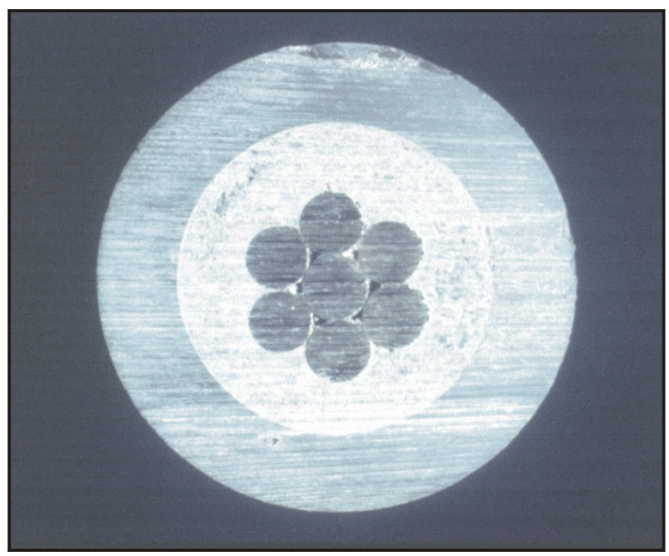


Figure 7.–Cross section of 1.52 cm (0.6 in) cable set in resin, installed in a 2.5 cm (1 in) diameter hole.

0.6 m (2 ft) of the anchor [Goris 1990; Goris 1991; Serbousek et al. 1987]. Essentially, the cable load in the anchor decays exponentially with distance along the anchor. Beyond about 0.6 m (2 ft) there will be little load in the anchor. However, once the cable yields, the lower portion of the resin anchor may begin to fail or the cable may debond from the grout, resulting in the loads being transferred further up the anchor. Essentially, the anchor becomes shorter as the cable is loaded beyond yield. Therefore, a minimum of 1.2 or 1.5 m (4 or 5 ft) of resin will provide a margin of extra length in allowing the cable to reach the ultimate strength. Also, this margin of extra length gives some degree of safety for improper grout installation although this is no guarantee that even a 1.2- or 1.5-m (4- or-5 ft) long anchor not properly installed will result in the cable reaching ultimate strength. Cable systems using longer resin anchor columns of 2.1 to 2.7 m (7 to 9 ft) have also been installed. However, depending on the resin used and the thrust capacity of the bolter, problems may be encountered with inserting the cable through that length of resin column.

An investigation was conducted underground using pull tests on 3-m (10-ft) cable bolts with 1.5-m (5-ft) resin anchors to evaluate parameters other than length that could affect the anchor performance [Zelanko et al. 1995]. Parameters varied in this study included hole diameter, resin type, and use of a resin keeper. The 0.6-in cable is generally installed in either a 2.5 or 3.5-cm (1-or 1-3/8-in) in diameter hole. Overall, the capacity and stiffness of the installed cable bolts were lower in the larger-diameter holes (3.5 cm [1-3/8 in] as compared to a 2.5 cm [1 in]. Although an adequate anchor can be achieved in holes of either diameter, there is less consistency in performance with the larger holes.

Higher-viscosity resins gave a better anchor performance than lower-viscosity resins. Cable resins are less viscous than the standard resins and allow for easier installation with the roof bolter. However, besides decreased performance, resin loss could occur more easily with these types of resin, thus requiring longer column lengths. More viscous bolt resins can be used, but with longer resin lengths, older bolting machines may not be able to thrust the cable through the cartridge. Furthermore, with the standard resins, another installation problem that can develop is loosening of bearing plates or plates not being in contact with the roof. In such cases, the higher- viscosity resins prevent the end of the cable from being pushed up completely through the cartridge even though the roof bolter was able the push the plate against the roof. When this occurs, the ungrouted section of cable bends and with the release of pressure, the cable will spring back, resulting in a loose plate. Resin keepers were also found to be important for the larger-diameter holes, but not a factor in 2.54-cm (1-in) in diameter holes. However, the need for a resin keeper will depend on the overall design of the anchor section of the cable bolt.

From this study, the system with the thinner annulus, the 1.52-cm (0.6-in) in diameter cable in a 2.5-cm (1-in) hole, performed better with more consistent behavior than the system with the larger annulus. In this regard, the button, birdcage, or nut case diameters must be matched with the proper hole size. This study highlights not only the necessity for a properly installed cable anchor, but also a properly designed anchor system as well.

CORROSION

Corrosion is a issue, although the extent of the problem is not completely known. Cables are more susceptible to corrosion and failure than other types of support. A nick in a cable strand that corrodes has a much greater impact than corrosion in a roof bolt. Some observations suggest that bright or black cables have about a 10% decrease in area of the strands six months after installation [Martin et al. 1996b]. Both galvanized strand and epoxy coated cable can be used to minimize the potential for corrosion [Goris et al. 1994]. Manufacturing techniques are now available that do not adversely effect either the strength or flexibility of galvanized cable [Tadolini et al. 1994].

Figure 8.–Surface control in the form of a "Monster Mat" installed with cable bolts.

SURFACE CONTROL

Bearing plates are necessary for the functioning of a cable bolt with a free length and allow the cable to load and resist rock movement while transferring this load thorough the anchor to rock deeper into the roof. Therefore, the bearing plate must be designed for the cable to reach ultimate strength, or a minimum of 260 kN (58,600 lbf). "Monster Mats" and T 5 channel are often installed with the cables to provide additional support and surface control across the row of cables (figure 8). A Monster Mat is a steel pan 0.48 cm (3/16 in) thick and 33 cm (13 in) wide, while a T 5 channel is 0.5 cm (0.2 in) thick and has a 10-cm (4- in) wide bearing surface. Both systems can add significantly to surface control and also provide some structural support. These systems are installed in conjunction with the high-capacity bearing plates.

DESIGN CONCEPTS FOR TAILGATE SUPPORT

In the tailgate, the primary support is designed to withstand development mining, but may not be able withstand the longwall environment and control the lower roof. Therefore, cable bolts can be installed as secondary support to maintain the entry. The cable bolts must keep the roof from falling and the entry open during panel mining. As the lower roof moves and deforms, the cables will distribute the forces that develop below a given failure horizon deeper into the roof through the cable and anchor support. Although there is primary support, it is not normally taken into account when designing the cable support system or, for that matter, another type of secondary support system.

The basic design concept in using partially grouted cable bolts to support the roof is suspension. Essentially, the cable bolt system must maintain and control the dead weight load of rock or rock movement below a potential failure horizon in the mine roof. This in part determines the spacing of the cable bolts. Furthermore, an adequate cable anchorage length must be obtained above a given failure horizon and, combined with the location of the failure horizon, determines cable length. Experience based on test sites in tailgates have further refined and established a basic design for cable spacing and row spacing. Although the cable systems are designed for the full dead weight of the rock, this is seldom seen and is somewhat an oversimplification of conditions, but it provides a starting point for design and designing to a worst-case scenario. Also, lateral roof movement, as well as vertical expansion from lateral roof movement, can cause significant loads to develop on the cables even beyond the weight of the rock.

CABLE LENGTH

The selection of cable length is the probably most crucial aspect of the design of a cable system. Depending on geologic conditions, selecting a length may be simple and straightforward, while in other cases, it may require an iterative process using a range of information. The key is to identify the location of potential failure horizons in the roof that may develop when the panel is mined.

Once the deepest potential failure horizon is identified, the cable length will be the depth of this failure horizon plus the length of an adequate anchor. Typical cable lengths in gate roads are between 3.7 and 4.9 m (12 and 16 ft). However, a minimum length in general should be for the cable bolt to be long enough to be anchored above the primary support. In this case the primary support zone is being suspended by the cables. However, there may be failure planes that develop above the primary support and require a longer length of cable bolt. This potential failure zone may be a flat or arched surface, depending on how the roof may fail. In a gate road situation, much deeper movements may occur that are not relevant to the stability of the immediate roof or the opening.

The initial step in designing an adequate support system requires gathering detailed information on ground conditions and the underground mining environment. To determine a potential failure horizon will require examining the roof and roof geology or evaluating roof performance to determine an adequate cable length. Such information may include a general estimate of rock mass strength or rating, geologic structure, and strengths of the immediate and main roof members. This information can be obtained from roof core samples and supplemented by observations from a borescope or camera to evaluate test holes in the roof. If the rock overlying the immediate roof is stronger or more competent, this may be an obvious place to locate the anchorage and is the easiest situation for determining cable length. However the geology may not be that clear-cut or the depth of the stronger unit may be too deep to be of practical use for supporting the immediate roof. Actual mining experience, test sites, and examination of roof falls can provide more data to help in the design of the cable system. Tests sites with instruments such as multi point extensometers can also be used to locate and evaluate these potential failure horizons. Such instrumented test sites can be used to confirm the adequacy of cable's length and design.

DESIGN FOR SUSPENSION

For cables, to consider that the rock is being supported through suspension may be an oversimplification, but does provide a basis for establishing the initial design of the system. Designing for suspension requires that the cables carry the weight of the rock under the potential failure zone, which, in many situations, is the worst case scenario [McDonnell et al. 1995]. In some situations, there will be loads that actually exceed rock load because of geology, horizontal stress, lateral rock movement, and mining-induced loads.

For suspension, the simplest approach is to identify a parting plane or a flat-lying, potential failure plane above the bolted roof horizon where the roof will shear at the pillar edge of the opening and the entire weight of the rock must be supported as a detached block (figure 9). The weight of the material can be determined from the following equations.

$$F_w = W_e H_p \gamma , \qquad (2)$$

where F_w = weight of rock per linear length, kN/m (lbf/ft),

W_e = effective width of opening, m (ft),

H_p = distance from coal roof to parting plane, m (ft) ,

and γ = rock density, kN/ m^3 (lb/ft^3).

If an arched roof failure is formed with the pillars carrying some of the weight, the cables need only support the weight of the rock under the arch (figure 10). The height of the arch will be determined by a combination of the geology, as well as by the vertical and horizontal stresses acting on the roof and the induced mining stresses. Obviously, the length and the number of cables will depend on the height of the arch, and therefore this requires the identification of the failure surface. The weight of the material within the arch can be estimated from the following equation.

$$F_a = \frac{\pi}{4} w_e H_a \gamma , \qquad (3)$$

where F_a = weight of rock under pressure arch per linear foot, kN/m (lbf/ft),

and H_a = height of pressure arch, m (ft).

The behavior of the pillar under different loading conditions will affect the width of the opening and therefore the weight of the rock that must be supported (figure 11). The depth of the yield zone can be determined from equations developed by Wilson and depend on the strength of the coal pillar [Wilson 1972]. The following equations can be used to estimate the depth of the yield zone. w = pillar width in meters (feet).

(1) Rigid floor conditions–

$$w = 2 \frac{m}{F} \ln\left(\frac{q}{p + p'}\right) \qquad (4)$$

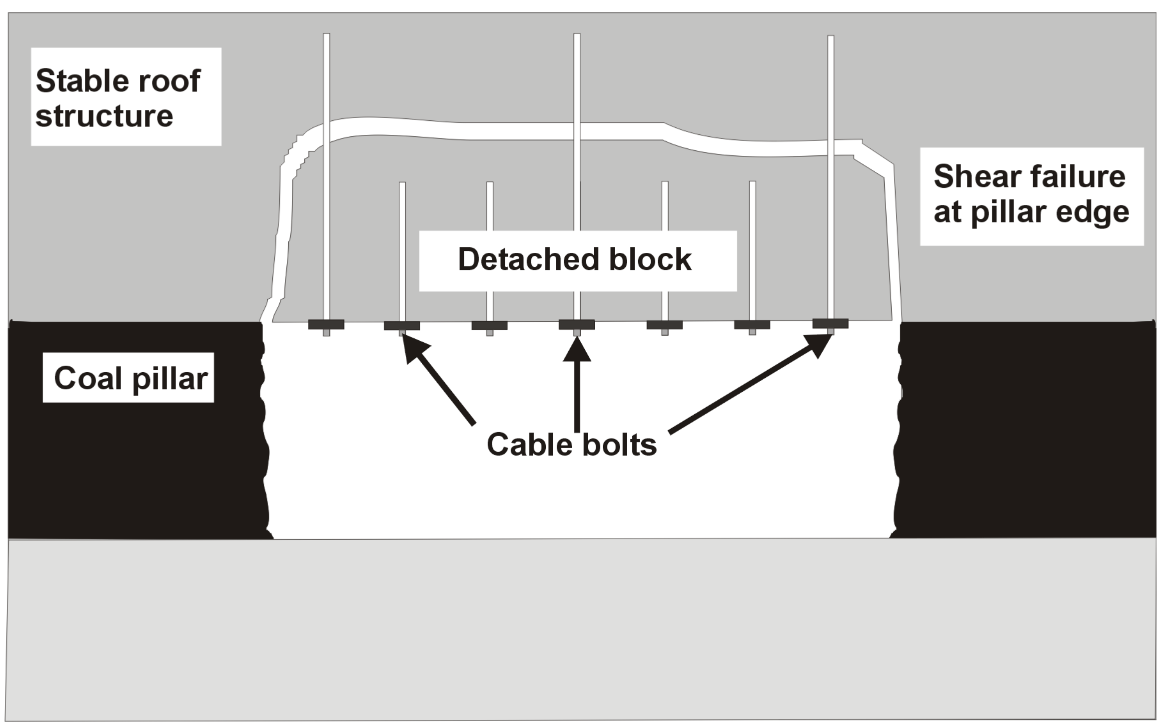


Figure 9.–Detached block of failed roof supported by cables.

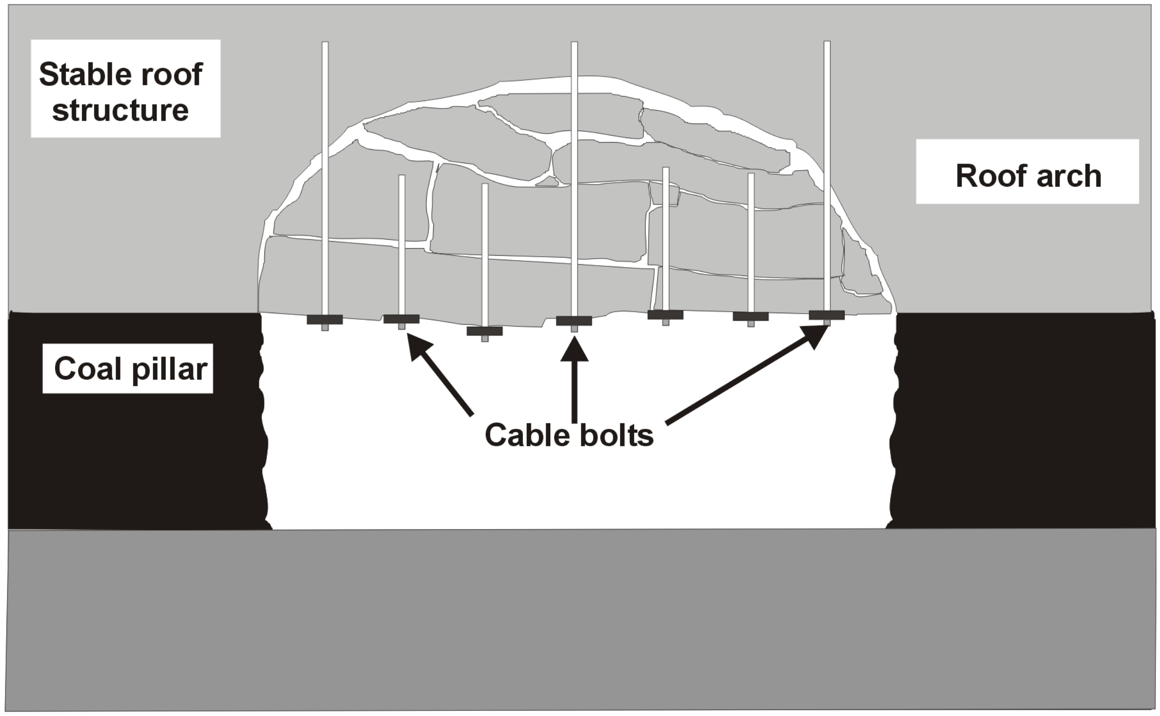


Figure 10.–Formation of pressure arch of failed mine roof material.

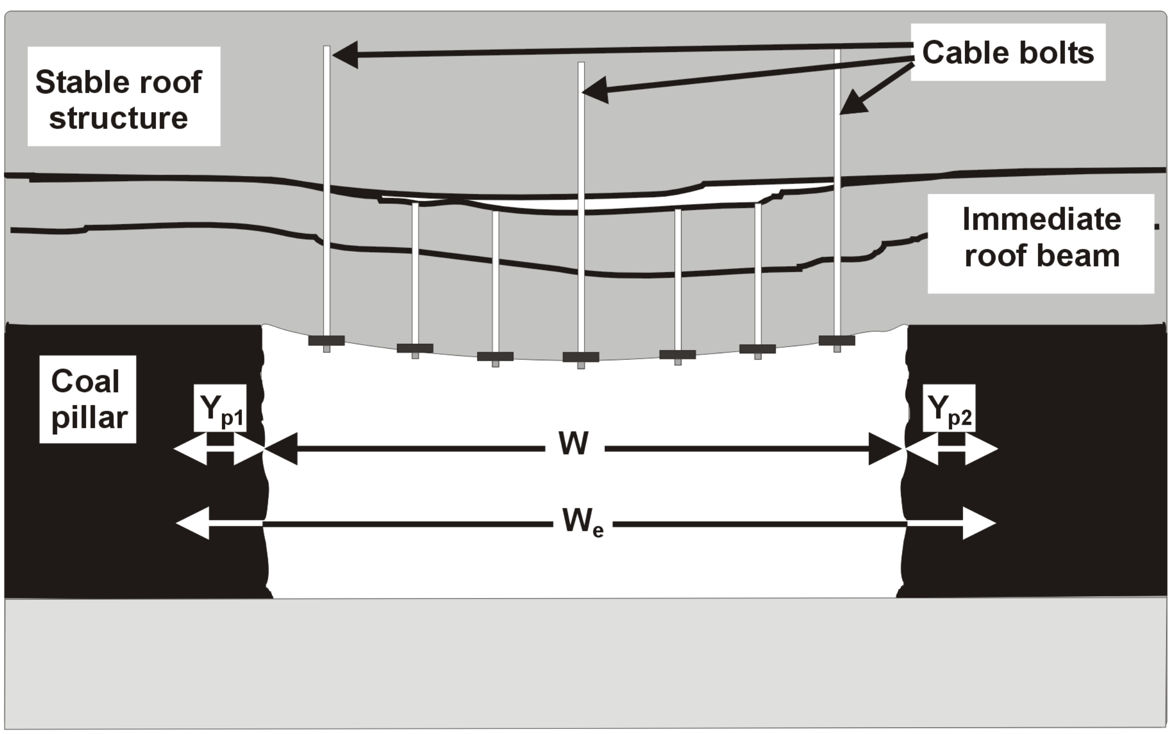


Figure 11.–Formation of yield zone in coal pillar, (W_e = effective width of opening; yp_1 = yield zone of pillar 1; yp_2 = yield

(2) Yielding roof-floor conditions–

$$w = m\left[\left(\frac{q}{p + p'}\right)^{\frac{1}{k} - 1} - 1\right] \tag{5}$$

$$F = \frac{k - 1}{\sqrt{k}} + \frac{(k - 1)^2}{k} \tan^{-1}\sqrt{k}\ , \tag{6}$$

where $\tan^{-1}\sqrt{k}$ is expressed in radians,

m = seam height, m (ft),

q = overburden load, t/m^2 (st/ft^2),

p = artificial edge restraint, 0 t/m^2 (st/ft^2),

p′ = uniaxial strength of fractured coal, $1/m^2$ (st/ft^2),

and k = triaxial factor = $\frac{1 + \sin\varphi}{1 - \sin\varphi}$, where ϕ = angle of interval friction, deg.

Figure 12 shows charts developed from these equations to calculate the depth of the yield zone. The charts were created using a angle of internal friction of 35°. The effective opening or roof width can then be determined from the following equation:

$$W_e = W + Y_{p1} + Y_{p2}, \tag{7}$$

where W = mined width of opening, m (ft),

Y_{p1} = yield zone for pillar 1, m (ft),

and Y_{p2} = yield zone for pillar 2, m (ft).

Based on the weight of material that must be supported, the spacing of cable bolts across the opening, as well as row spacing, can be calculated. Using a cable with a capacity of 260 kN (58,600 lbf) and varying the number of cables across the opening and row spacing, a design can be determined for different thicknesses of rock that must be supported. Figure 13 shows this design chart for an effective width of 7.6 m (25 ft) and a rock density of 2,403 kg/m^3 (150 lb/ft^3). This chart is based on a flat failure surface developing at the given horizon with the additional weight of material for the yield zone. A separation at 2.4 m (8 ft) would require four cables per row with 2.4-m (8-ft) row spacing. However, there are no safety factors calculated into these charts.

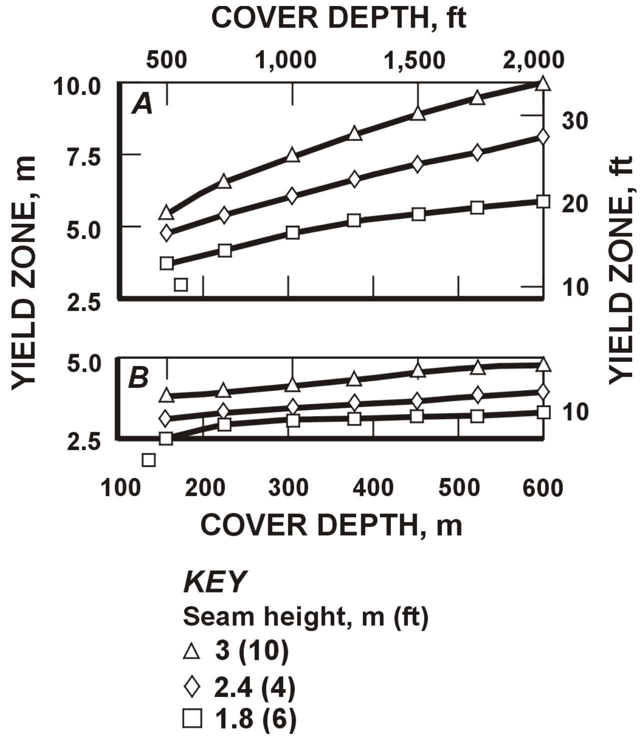


Figure 12. –Design chart to determine yield zone width in coal pillars. A, yielding roof; B, rigid floor.

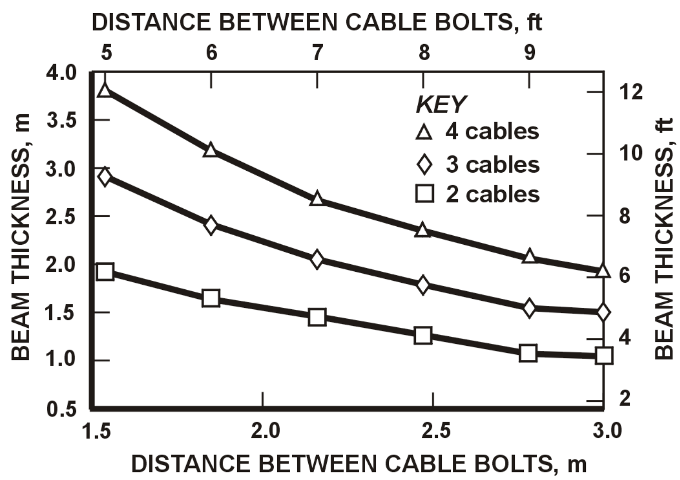


Figure 13.–Cable support design chart, where the effective width of opening (W_e) = 7.6 m (25 ft) and rock density (γ) = 2,403 kg/m^3 150 lb/ft^3 [McDonnell et al. 1995].

PERFORMANCE OF CABLE BOLTS WITH RESPECT TO TAILGATE INTERACTION ZONES

When using cable bolts for secondary support in the tailgates, there are three zones that must be considered in evaluating the design and performance of the cable systems. These zones are the outby abutment zone for both vertical and horizontal stress, the shield zone from the face to the back of the shield, and the cave zone. Each zone has different performance requirements, and therefore, the cable system much be designed to meet these requirements. If problems do occur in the tailgate that results in the shutdown of the face, the cost to the operation in both downtime and clean-up can be high.

In the forward abutment zone, the cable support must maintain an open tailgate entry and prevent any major roof falls that impede the use of the tailgate as a secondary escapeway and for ventilation (figure 14). In the abutment zone, the cable loads will depend in part on the geology, depth, and pillar design. This zone receives the most support from the pillars and the panel and may be up to 45 to 60 m (150 to 200 ft) wide. The depth will control the pillar yield zone that develops along the tailgate entry and at the face, where an increase in the entry or intersection span will, in general, result in more roof separation and movement. This yield zone will obviously increase near the face. With pillar design, abutment pillars will offer the most support to the tailgate entry. In many situations, little load or roof movement will be seen. With a yield pillar adjacent to the tailgate, significant roof movements and cable loads can develop when the pillar yields. Often this will include lateral movement that the cables must withstand.

Geologic structures such as joints, faults, and sand channels, can cause locally high loads to develop on the cable support and can result in cable failures and even small roof falls. These roof control problems will usually begin with a sudden increase in the rate of vertical loading from the abutment. In such cases, some additional support may be required locally if the cables fail. Although horizontal stress is not typically a problem in the tailgates because of the adjacent caved panels, horizontal stress damage to the roof may have been caused by a previously mined panel and the damage may have been transmitted through the crosscuts to the tailgate entry. Lateral roof movement may occur just outby the face and result in additional cable loads or even cable failure in shear. Furthermore, damage done to the roof in this zone and subsequent loads on the cables will impact performance in the other zones. In general, in the abutment zone, the highest loads and roof movements will be seen in the intersections although with yield pillars, this may occur at mid-pillar.

In the zone from the face to the back of the shields, performance requirements are very similar to those for the abutment zone —the area must remain open as an escapeway and for ventilation (figure 15). However, support of the panel has been removed and replaced by the shields, and this creates an opportunity for the roof to move because of the loss of support. Therefore, higher cable loads will develop here than in the abutment zone. This is the situation for which the cable system should be designed. The degree of roof movement and separation will depend to a large extent on the geology and any previous damage done to the roof. Cables often begin to load in this zone when there was little movement in the abutment zone, especially when abutment pillars are adjacent to the tailgate. Maximum loading and roof movement are seen just as the cables go behind the shields.

In many operations, there is no need to maintain the tailgate behind the shields and the performance of the cable bolts in this

Figure 14.–Tailgate abutment zone outby the face supported with cable bolts.

Figure 15.–Tailgate shield zone supported by cable bolts.

area is not a factor. However, in some mines, ventilation requirements necessitate that the gateroad be kept at least partially open to the nearest crosscut behind the face, a distance of usually 30 to 45 m (100 to 150 ft). The maintenance of this section of the tailgate by the cables is dependent to a large degree on the geology and the cave and only to a limited degree on cable system design. In the tailgate adjacent to the cave, the roof develops into a cantilever that must be supported. If the roof is not strong enough and the cave goes above the cables then the cantilever could fail and close most of the entry (figure 16). If the roof is strong enough to maintain the cantilever, then the cables will help to maintain any lower weaker roof. The critical factors are whether the cave develops above the cables and if the zone is strong enough to maintain the cantilever. The cables probably add little overall strength to the cantilever. However, there are cases where the entry has stayed open more than 45 m (100 ft) behind the face [Koehler et al. 1996; Martin et al. 1996; Mucho et al. 1996] (figure 17).

Geologic structures such as joints can cause periodic failure of even a competent roof behind the face. Essentially, there is no guarantee with cables that this zone can be maintained to the next crosscut. If the tailgate must be kept open, then other types of support should be considered. However, even if the roof fails, a portion of the tailgate alongside the pillar will usually remain open, although this is a restricted area [Molinda et al. 1997].

DESIGN BASED ON TEST SITES

Test sites have been used to establish, evaluate, and confirm cable system designs. Besides being a good practice, test sites may be required by MSHA when cable systems are used for the first time at a mine. Test sites can also be used to modify existing cable system designs. Although observation can be used to judge the successes of the design, instruments that monitor both roof movement and separation and cable loads to quantify the results and confirm the design are preferred. Monitoring of roof movement is especially useful when evaluating cable length. Final cable system designs should be based on evaluation of test sites.

The design most used in tailgates has been one in which there are four cable bolts per row. Although three bolts can provide adequate support with the same safety factors, four bolts per row have certain advantages. This number provides good coverage across the entry, thus maintaining an effective support front, especially as the cable row goes behind the face. Also, in a given row, the failure of a single cable represents a loss of support of only 25% with four cables per row and 33% for three cables per row. Although the cable support is designed on the basis of an area of support, as the support goes behind the face, the performance of a single row or the line of support becomes important. Finally, with the use of double-boom bolters, it is usually more efficient to install four bolts than three bolts per row. Row spacing has varied from 1.2 to 1.8 m (4 to 6 ft). With row spacings wider than 1.8 m (6 ft), interaction between rows can be lost and the effectiveness of the reinforcement as a system reduced.

Additional support to the crosscuts must also be considered when using cable bolts because of the increased spans in the intersections and any damage in the crosscuts from previous panels. Generally, this support can consist of one or two rows of cables installed in the crosscuts. Instead of (or in conjunction with) the cables, cribs can also be set in the crosscuts. Another modification to the design is to angle the outside cable bolts toward the pillars and panel. This angle is usually about 10° from vertical and will allow the anchorage to be in a more stable roof zone.

DESIGNS FOR LATERAL MOVEMENT

Cable bolts will offer resistance to lateral movement, although shear is resisted to a large degree only after the peak rock strength has been exceeded. Essentially, the rock has failed and is now mobilized where the cables will offer significant post-failure resistance by significantly increasing the residual shear of the rock [Goris et al. 1995, 1996]. However, in some cases, because of large lateral deformations, the cables may not be able to stop or limit this displacement prior to failing. At a mine in western Colorado, a tailgate supported with cable bolts was subjected to large lateral deformation. This occurred as the adjacent panel was being mined, with the horizontal stress abutment in the headgate causing roof damage not only to the headgate, but also to the tailgate of the next panel through the crosscuts [Dolinar et al. 1996]. This panel was supported with cable bolts and rigid trusses. About 0.3 to 0.45 m (1 to 1.5 ft) of lateral movement occurred in places along this entry. All the rigid truss cross bars had been thrown from the anchor bolts while about 20% of the cable bolts failed. It is estimated that the cables withstood about 5 to 10 cm (2 to 4 in) of lateral movement before failure. These are very tough ground conditions where few support systems could be expected to prevent movement of this magnitude. With shear or lateral movement, the flexibility of the cable bolts is not fully utilized.

Figure 16.–Tailgate behind the shields has caved though supported with cable bolts.

Figure 17.–Tailgate behind the shields kept open with cable bolts.

Obviously, it may be difficult or impossible to stop such large movements with support, and other approaches may need to be considered to prevent support failure. If the support does not fail, then it can support the damaged roof by suspension. However, there are some alternative approaches that can be used to minimize the impact of large lateral movements on cable supports. One approach is to keep the cable bolts out of the highest zones of shear or differential lateral movements that occur near the edge of the pillar. To do this, cables can be positioned 0.6 to 0.9 m (2 to 3 ft) from the rib. Another successful approach is to use cable bolts with a yielding head. These heads will allow the cable system to yield in a controlled manner at loads below the ultimate capacity of the cables [Tadolini and McDonnell 1998; Vandekraats and Watson 1996]. Some of these heads will allow up to 50 cm (20 in) of controlled movement, thus letting the cable deform with the roof. With nonyielding cable heads, the head will lock in the bolt, and stretch in the system must take place as the bolt goes into a yield condition.

CASE HISTORIES OF CABLE BOLTS AS SECONDARY SUPPORT IN TAILGATES

The following section gives case histories for tailgates supported with cable bolts either as the main or only secondary support system. In each of these cases, a 1.52-cm (0.6-in) in diameter cable bolt with an ultimate capacity of 260 kN (58,600 lbf) was used. At these test sites, cable loads were monitored usually with hydraulic U-cells and pressure pads, while differential roof sag measurements were made within and above the cable horizon. Roof-to-floor convergence measurements were also obtained at some sites. Usually, several intersections as well as midpillar locations were monitored.

CASE HISTORY 1

This mine is located in western Colorado and used a yield-abutment pillar configuration. Three different cable bolt system designs were tested in a 274-m-(900- ft-long) section of the tailgate. They included a passive system, a stiff passive system (increased grout anchorage length), and a tensionable system [McDonnel et al. 1995; Tadolini and Koch 1993; Tadolini and Koch 1994]. The roof geology consisted of 1.2 m (4 ft) of coal overlain by 0.6 m (2 ft) of silty shale and 1.2 m (4 ft) of interbedded shale, silty shale, and sandstone. After evaluating the geologic data on the roof, it was thought that roof separation would most likely occur in the silty shale although separation might also develop higher in the interbedded shale and sandstone. Above the immediate roof was a 4.9-m-(16- ft-) thick massive sandstone. This sandstone provided a good anchorage from which to suspend the lower roof. The entry width was 5.8 m (19 ft), but with pillar yield, the effective width for design was assumed to be 7.9 m (26 ft).

Figure 18 shows a tailgate entry cross section with the cable configuration where four cables per row were installed on 1.5-m (5-ft) row spacings. The cable bolts were 4.9 m (16 ft) long. With this configuration, the cables would have just enough capacity to hold up 3 m (10 ft) of rock if the separation occurred at this level and the full weight had to be supported by the cables. For surface control, bearing plates, monster mats, and wire mesh were used. For the passive site, the cables were installed with a resin grout length of 1.7 m (5.7 ft), which assured adequate anchorage in the sandstone and resulted in a free cable length of 3.1 m (10.3 ft). For the stiff passive system, the resin anchor length was 3.7 m (12 ft), leaving only 1.2 m (4 ft) of free cable length. In the tensional section, the resin length was again 1.7 m (5.7 ft) with a free cable length of 3.1 m (10.3 ft). These cables were tensioned to 35 kN (8,000 lbf). Because of the thrust from the roof bolter, the cables in the passive sections were installed with 6.7 to 22.2 kN (1,500 to 5,000 lbf) of load.

With panel mining in the passive area, the maximum total roof separation was about 0.6 cm (0.25 in) in an intersection. In the stiff and tensional areas, the maximum total separation was between 3.2 and 3.8 cm (1.25 and 1.5 in) in both sections. The movement and separation took place within 30 m (100 ft) of the face and did not affect functioning of the tailgate or load the support beyond the cable's strength. Cable loads in the passive section ranged from 0 to 107 kN (0 to 24,000 lbf) and averaged 21.3 kN (4,800 lbf). In the stiff section, cable loads alongside the shields ranged from 71 to 116 kN (16,000 to 26,000 lbf). For the tensioned cable site, the loads ranged from 18.2 to 151 kN (4,100 to 34,000 lbf). However, in the tensioned test site area, several geologic features, including coal spars and a clay dike, were observed in the roof. In one small area, the cables were loaded to over 133 kN (30,000 lbf), while the roof was broken and fractured. In this area, some cables appeared to have failed or the cable heads had slipped. Nine wood posts were set to provide additional support, although the section through the area was mined without incident. In the passive area, the roof remained open 30 to 45 m (100 to 150 ft) behind the face, while for the stiff system, the entry remained open about 30 m (100 ft) behind the face.

All three systems worked extremely well and were able to keep the tailgate open through the abutment zone, alongside the shields, and even for a distance behind the shields. However, from these test sites, it could not be determined if there were any difference in performance among the systems. In the tensional area, localized geologic structure in the immediate roof did induce higher cable loads and possibly cable failures, but no significant problems were apparent in controlling the roof.

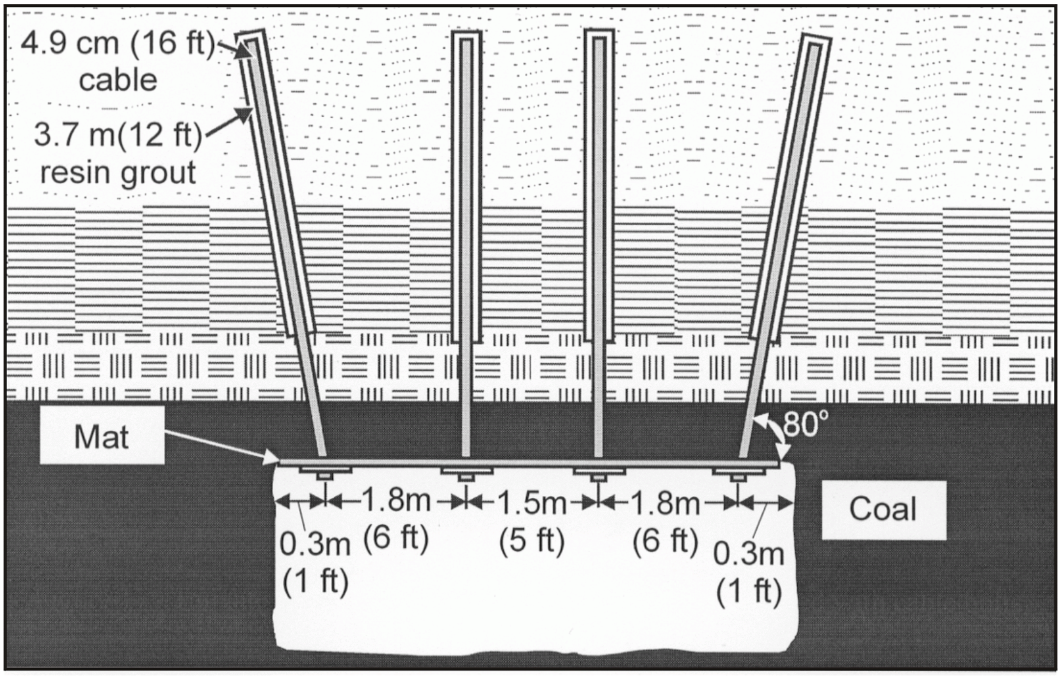


Figure 18.–Cross section of tailgate entry showing cable bolt design for case history 1.

CASE HISTORY 2

Case 2 was a mine located in Utah with a double entry yield pillar configuration in the tailgate [Tadolini and Trackemas 1995]. The depth of cover at the mine averaged about 460 m (1,500 ft). Because of the yield pillar, the effective opening width was estimated at 9.7 m (32 ft). The geology of the immediate roof consisted of thinly bedded siltstones, sandstone, and mudstones along with carbonaceous material to a depth 1.2 to 1.8 m (4 to 6) ft. This was overlain by a sandstone containing bands of carbonaceous material. Sand channels cut into the immediate roof, but not into the coal, and affected roof quality locally.

Figure 19 shows the geology as well as a cross section of the entry with the cable system design. The cable bolts were 4.9 m (16 ft) long with four bolts per row and a row spacing of 1.5 m (5 ft). The resin anchor length was 1.5 m (5 ft), resulting in a free cable length of 3.4 m (11 ft). This free cable length allowed for greater cable elongation in the high-stress and deformation environment caused by crushing of the yield pillar. This cable system design would support a roof thickness of up to 3.3 m (10.9 ft) based on a dead weight load.

The installed cable loads averaged 15.1 kN (3,400 lbf). With panel mining, cable loads ranged from 0 to 178 kN (0 to 40,000 lbf) during the life of the test site. Loading and unloading of cables occurred in the same row, while shearing in the roof was observed at different depths. This shearing action resulted in differential lateral movement between roof layers and could explain the loading and unloading of the cables. In the area of the sand channels, several cables failed because of this differential movement, and some standing support was added. Separations were observed in the mine roof, but never above the anchor horizon. Up to 10 cm (4 in) of overall roof separation was seen, most of which occurred between the mudstone-sandstone layers within the lower 1.2 to 1.8 m (4 to 6 ft) of the roof. This was within the elongation capacity of the cables. However, cable loads up to 275 kN (40,000 lbf) and cable failures indicated that this level of elongation was approaching the limit of the cable system especially as it was developed by shear. Despite this high-deformation environment, the tailgate was kept open and functional with the cable support even under the sandstone channels (figure 20).

In a series of initial experiments with cable support, the mine installed a double row of wood cribs with spacing that was increased from 1.8 to 6.1 m (6 ft to 20 ft) through the tailgate test area. Finally, a section with no cribs and only cable support was tested. The results of these trials indicated that the best roof conditions were when there few or no cribs. The hypothesis was that the standing support damaged the roof as it resisted the main roof-to-floor convergence. The roof damage and subsequent hazardous conditions resulted as the cribs were compressed against the roof with such force that it caused the immediate roof to break.

CASE HISTORY 3

Case 3 is a mine in western Colorado using a three-entry system with two abutment pillars [Dolinar et al. 1996]. The roof generally consists of a thinly bedded siltstone (stack rock) and massive, fine-grained sideritic siltstones that grade laterally

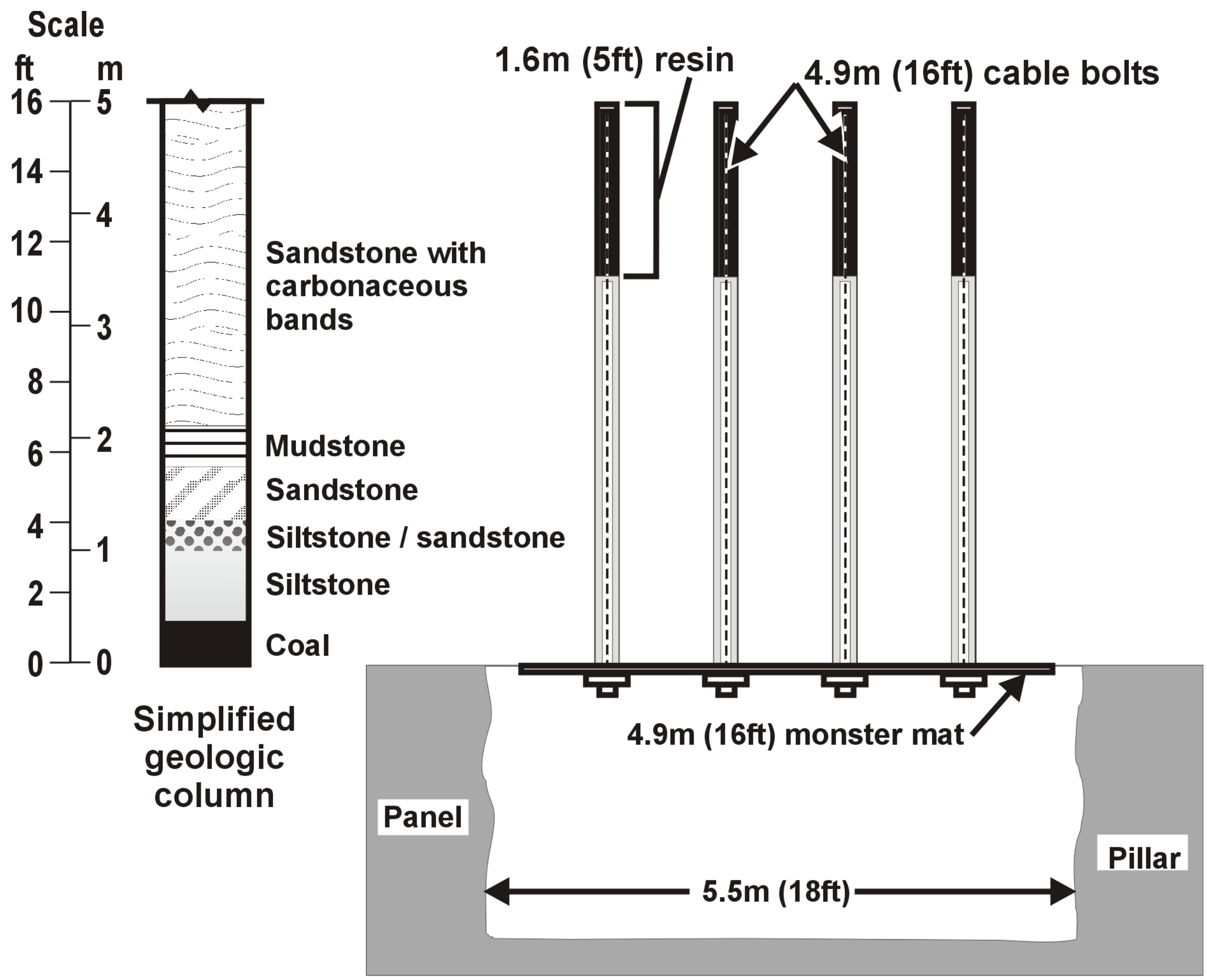


Figure 19.–Cross section of tailgate entry showing cable bolt design for case history 2.

to a dark gray limestone and to sandstone. Another seam overlies the mined seam at distances ranging from 0.9 to 5.5 m (3 to 18 ft). The thickness of the interburden is important to the roof control problems that develop at the mine. The mine is also subjected to high horizontal stresses with a ratio of maximum horizontal to vertical stresses of 1.7.

Test sites were established in tailgates of two adjacent panels. Figure 21 shows the geologic column and an entry cross section with the cable support design. The cables were 4.9 m (16 ft) long with four cables per row with a row spacing of 1.5 m (5 ft). Anchorage length was 1.5 m (5 ft), leaving a free cable length of 3.4 m (11 ft). In addition, high-capacity dome-bearing plates as well as monster mats were installed for surface control. At the initial test site, the interburden was 1.8 m (6 ft). Since this was the first use of cables at this operation, a double row of cribs was installed as additional support. Even with the crib support, cable loads averaged 98 kN (22,000 lbf), while a total of nine cables failed in the 122-m (400-ft) test zone. The failure was due to the large lateral movements that occurred in the interburden. Roof separation ranged from 2.0 to 5.3 cm (0.8 to 2.1 in) and occurred between 1.2 to 1.8 m (4 to 6 ft) into the roof. Cribs in the test site were highly deformed by the lateral roof movement.

At the second site in the adjacent panel, interburden thickness was 5.5 m (18 ft). The maximum increase in cable loads was 56.9 kN (12,800 lbf) with an average increase of only 2.2 kN (500 lbf). The different sag stations showed less than 1.8 cm (0.7 in) of movement. Roof conditions remained excellent, and no roof control problems were encountered in the entire cable section. Often the roof would remain standing one or more crosscuts behind the face, a distance of about 45 to 90 m (150 to 300 ft).

The difference between the two sites was the interburden. The thinner interburden consisted of weaker layers rock (stack rock) subject to horizontal stress damage and lateral movement. With the extensive lateral movement at the first site, a combination of cribs and cables did maintain the tailgate. However, the cribs did little to stop the lateral movement, and the cables may have been able to maintain the gate road without the cribs. With less lateral movement, the gate road probably could have been easily maintained with the cable support. In a third tailgate with a thin interburden and supported only by

Figure 20.–Two entry yield pillar tailgate showing the abutment zone outby the face, supported with cable bolts

cables and rigid trusses, large lateral movements were also encountered. Several cable bolts did fail along with all the rigid trusses. This occurred as the adjacent panel was mined. In this tailgate, lateral movement of between 0.3 to 0.46 m (1 to 1.5 ft) occurred and when the panel was mined, a roof fall did occur in the tailgate that resulted in some delays of the longwall. Under these very severe ground control conditions, additional support may be required although these are tough conditions for most support systems to control.

CASE HISTORY 4

Case 4 is a mine located in Utah with a yield-abutment pillar configuration [Koehler et al. 1996]. The geology of the immediate roof consists of 0.3 to 0.6 m (1 to 2 ft) of coal and 0.3 to 0.6 m (1 to 2 ft) of mudstone overlain by a 0.3 to 0.6 m (1 to 2 ft) layer of gray sandstone. Above this was a white sandstone with occasional shale bands to a depth of at least 6.1 m (20 ft). The cable support design consisted of 4.3-m-(14- ft-) long cables with four cables per row on 1.5-m (5-ft) row spacings. The resin anchor was 1.2 m (4 ft) long, leaving a 3 m (10 ft) length of free cable. T5 channel was used for surface control.

The installation loads on the cables averaged 12.9 kN (2,900 lbf). During mining of the panel, load increases on the cables ranged from 0 to 118 kN (0 to 26,500 lbf). In the intersections, the cable loads increased an average of 7.1 kN (1,600 lbf). However, the highest cable loads were associated with a near-vertical joint located near a mid-pillar instrument site. Maximum cable load increase was 118 kN (26,500 lbf) while the average increase was 66.7 kN (15,000 lbf). Higher cable loads were measured along the pillar side that may be attributable to the yield pillar and the roof breaking adjacent to the pillar in reaction to pillar yielding. This may also explain why the largest loads were seen at the midpillar locations. The maximum roof movement measured in the intersection was only 1.0 cm (0.4 in). Through the test section, there was 10 to 15 cm (4 to 6 in) of roof-to-floor convergence because of the yield pillar. Behind the shields, the tailgate would remain open for 15 to 41 m (50 to 135 ft). Then the entry would cave to just behind the shields. This distance was controlled by a near-vertical joint set subparallel to the face. This was the same joint set that resulted in the highest cable loads at the test site. Outby the cave, the tailgate remained open with no ground control or roof problems.

CASE HISTORY 5

Case 5 is a mine in southern West Virginia with a three-entry abutment pillar configuration. The immediate roof at the mine makes a transition from sandstone to shale [Mucho et al. 1996]. In some areas, the sandstone appears to be massive, while in others it appears to be highly laminated, fossilized, and interspersed with coal streaks. The horizontal stress is high enough to cause damage at some locations, especially with a thinly laminated roof. The cable system design consisted of 3.7-m (12- ft-) long cables with four cables per row on a 1.8-m (6-ft) row spacing. These bolts were tensioned by the use of a threaded rebar head at the bottom end of the cable. The resin anchor was 1.5 m (5 ft) long.

When the panel was mined, less than 0.25 cm (0.1 in) of roof separation was recorded and cable loads increased on average only 8.9 kN (2,000 lbf). Maximum loads and roof separations occurred as the instruments went behind the shields. The tailgate roof area was extremely stable outby the cave. Behind the face, the roof stayed up for a distance of 23 m (75 ft) before the roof caved to just behind the shields. This cyclical caving of the tailgate roof occurred throughout the test area. In addition, when the adjacent panel was mined, there was almost no floor heave in the cable section (tenths of inches) as compared to the crib section where several inches occurred.

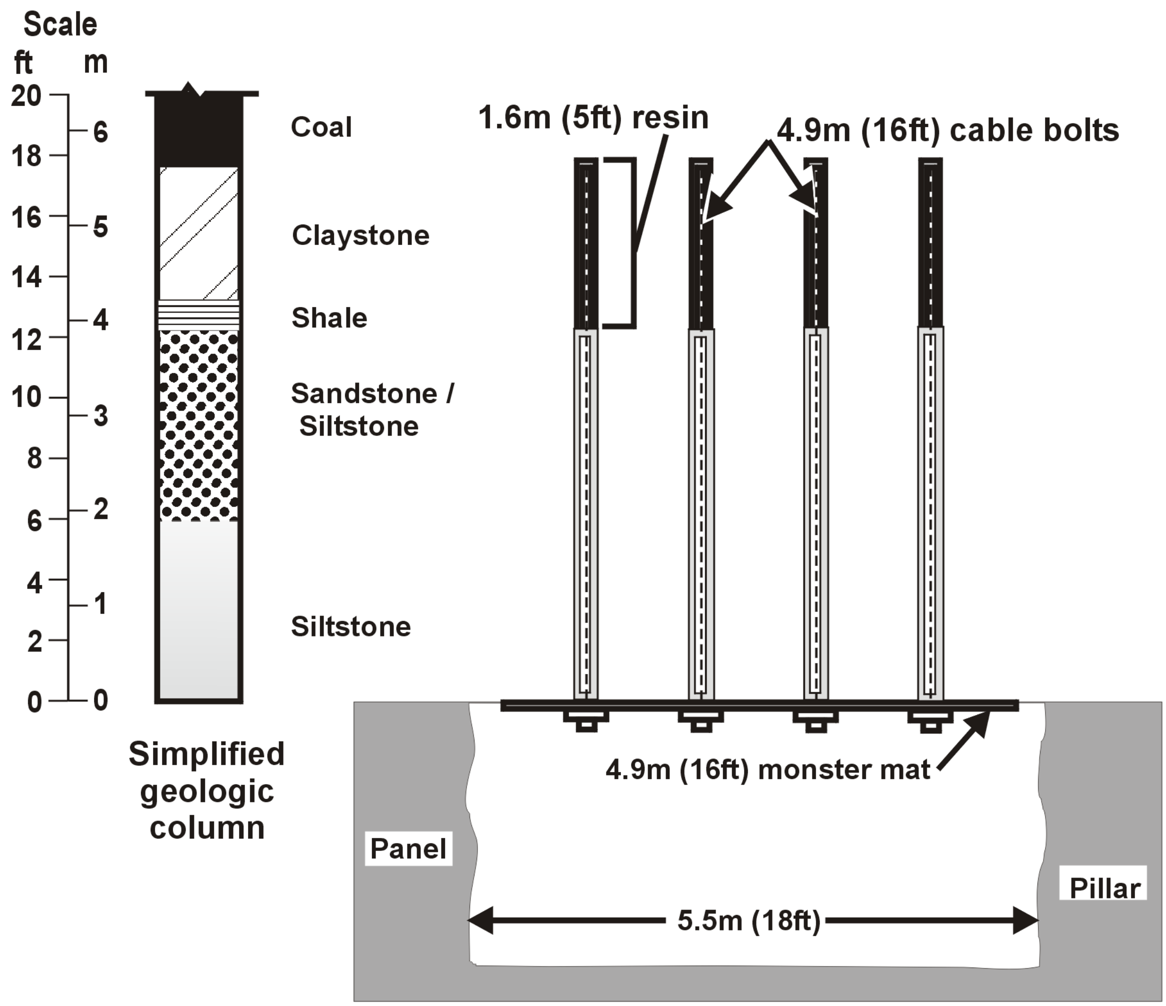


Figure 21.–Cross section of tailgate entry showing cable bolt design for case history 3.

CABLE TRUSSES

Cables trusses are anchored over the pillars and panels outside the potential failure envelope and provide resistance to roof movement along the roof line. Because cable trusses have a high strength and flexibility and a low stiffness, they can survive in a high-deformation and stress environment where other supports would fail. Essentially, cable trusses move and deform with the rock with the truss providing only limited resistance to vertical movement [Scott 1994]. Some systems can be pretensioned, but tensioning is probably not significant to improved ground control, but that assures the truss is tight when installed and therefore can respond immediately to roof movement. Cable trusses have been used in mines since at least the 1970s, but on a limited basis [Scott 1989; Mangelsdorf 1982]. However, in the 1990's with the advent of cable bolting in U.S. coal mines, newly designed cable truss systems that can be installed with roof bolters and anchored with resin grout cartridges are now being used much more extensively as supplemental support, especially in headgate entries.

DESCRIPTION

Cable trusses are constructed from a seven strand cable usually having a diameter of 1.52 cm (0.6 in) and an ultimate strength of 260 kN (58,600 lbf). However, cables with a diameter of 1.27 cm (0.5 in) are also used. Cable trusses are normally installed in a hole 3.5 cm (1-3/8 in) in diameter, although the system can be installed in a 2.5-cm (1-in) hole. The drill holes are typically up 2.4 m (8 ft) deep and drilled 0.6 m (2 ft) from the rib at an angle of about 45° over the coal rib. Domed and grooved bearing plates usually 15 by 40 cm (6 by 16 in) in size are used as bearing surfaces for the rock and cable. This allows for a two-point contact along the roof at installation. At the drill hole-roof interface, the cable will also be in contact with the rock, and a crushed zone may develop as the cable loads. Cable trusses may be composed of either single or multiple pieces, which affect how the systems are installed but not their function.

A one-piece truss consists of a single, continuous cable with anchorage buttons and a resin mixer on each end of the cable in the anchor zone (figure 22) [Dolinar et al. 1996]. The truss uses a no-spin system to mix the resin while a push button on the cable and a special bolter wrench allow for the insertion of the cable into the hole and through the resin with the roof bolter. The procedure is that one end is installed, and the resin is allowed to cure. Then the other end of the cable is placed into the hole on the opposite side of the entry and thrust through the resin with the roof bolter.

With this system, installed cable truss loads ranging from 15.1 to 51.6 kN (3,400 to 8,200 lbf) have been measured. The goal is not to develop large loads in the roof but to simply tighten the truss so that it will provide some immediate resistance to rock movement.

The three-piece cable truss consists of two angle cable bolts and a horizontal cable member (figure 23) [Oldsen et al. 1995]. The angle bolts can be constructed with nuts or birdcages for anchorage as well as resin keepers. The cable bolts are pushed and rotated into the hole and through the resin using a special wrench and a roof bolter. A splice tube assembly is attached to the angle and the horizontal cables, which allows the pieces to be connected and the system to be tensioned. The housing and wedge assembly that form the cable heads are installed in the field and allow the cables to be tensioned against the splice tube. A tensioner powered by the hydraulics of the bolter is used to tension the system and at up to 71.2 kN (16,000 lbf) of preload.

ANALYSIS OF CABLE TRUSS LOADING

The loads developed in a truss can be evaluated by simple statics. Figure 24 shows a simple free-body diagram of the loads for a half of a truss. The following equations can be used to describe the relationship between the reaction force R broken into horizontal and vertical components and the cord tensions.

$$Y_r = T \sin \propto \tag{8}$$

$$X_r = H - T \cos \propto \tag{9}$$

where T = tension in the diagonal member,

H = tension in the horizontal member,

Y_r = vertical reaction force,

X_r = horizontal reaction force,

and $\propto$ = angle of inclination of the cable.

The reaction force R may be a compilation of several forces, especially in the case where the inclined cable bears against the roof at the drill hole. However, these equations are still valid for describing the vertical force Y_r applied to the rock by the truss or to the truss by the rock [Mangelsdorf 1979]. With a cable truss, the tension transfer between the horizontal cord and the diagonal cords becomes more complex. Figure 25 shows a free-body diagram for the more complex loading conditions for a cable truss. Essentially, tension load transfer will take place by slippage of the cable over the bearing block or plate, the bearing block or plate over the rock, or the cable over the rock at the edge of the borehole. These load transfers are dependent on overcoming these frictional forces. Because of these complex loading conditions, the tension in all three legs of the truss must be measured along with roof sag to evaluate field performance.

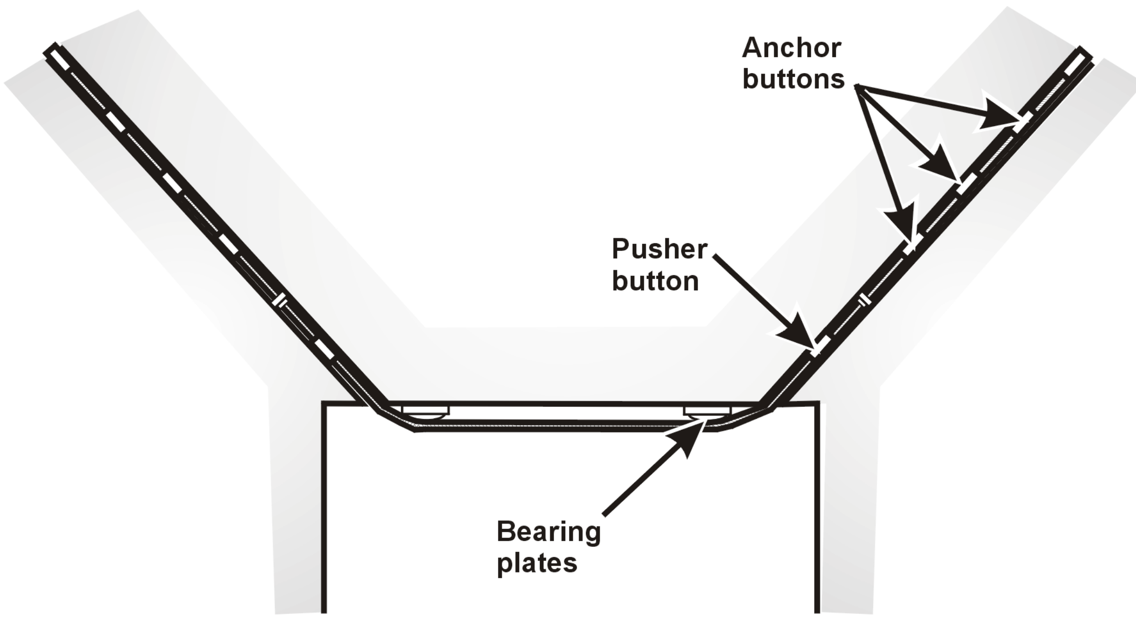


Figure 22.– Single piece cable truss.

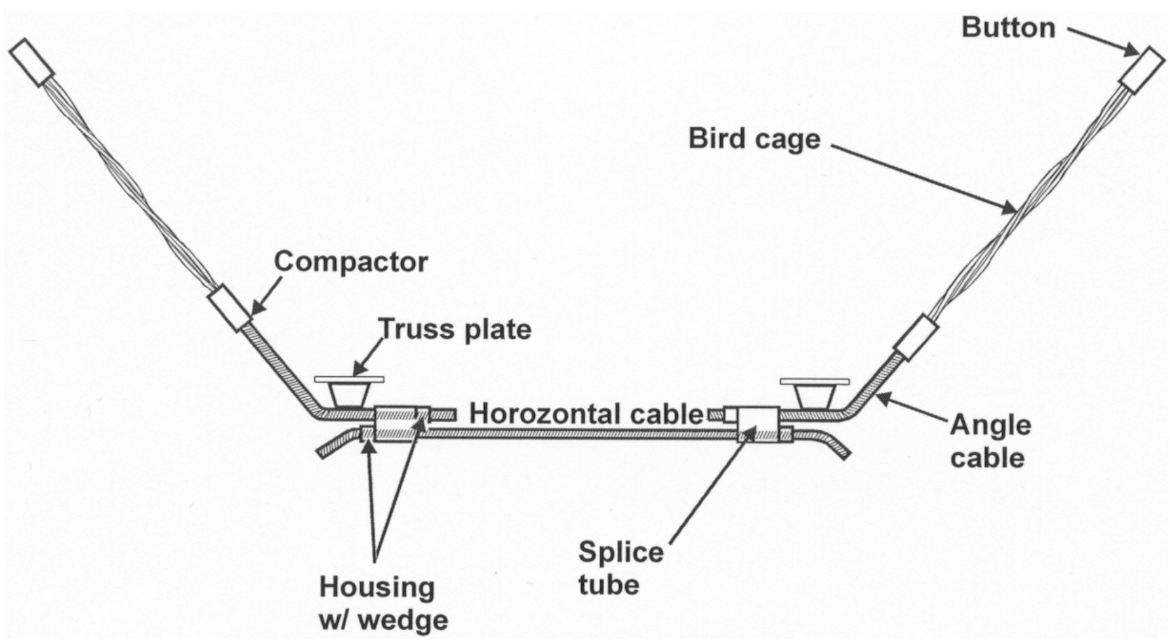


Figure 23.–Three piece cable truss.

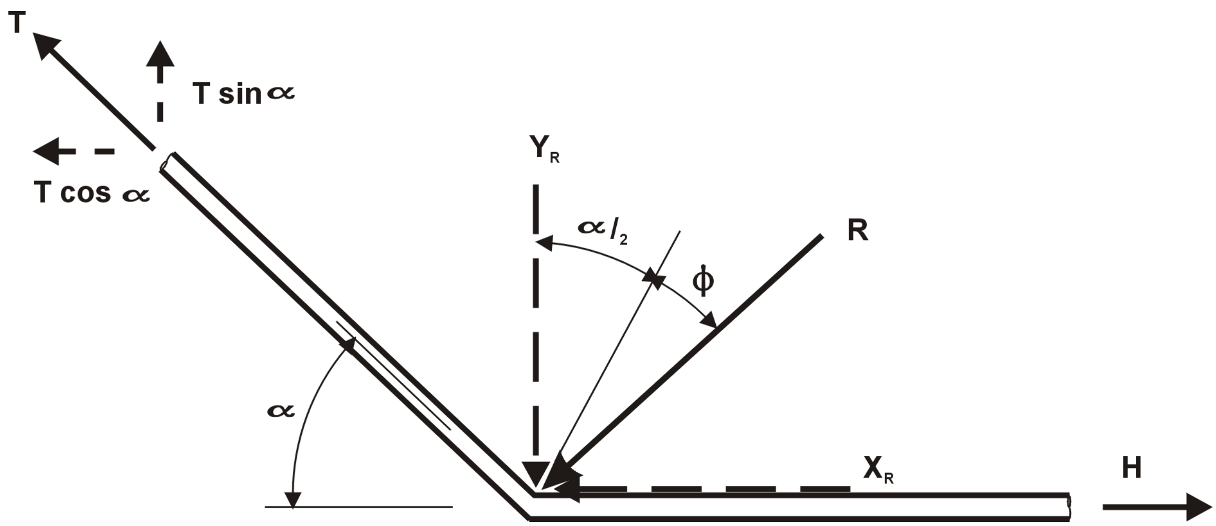


Figure 24.–Free body diagram of half truss showing the truss loads.

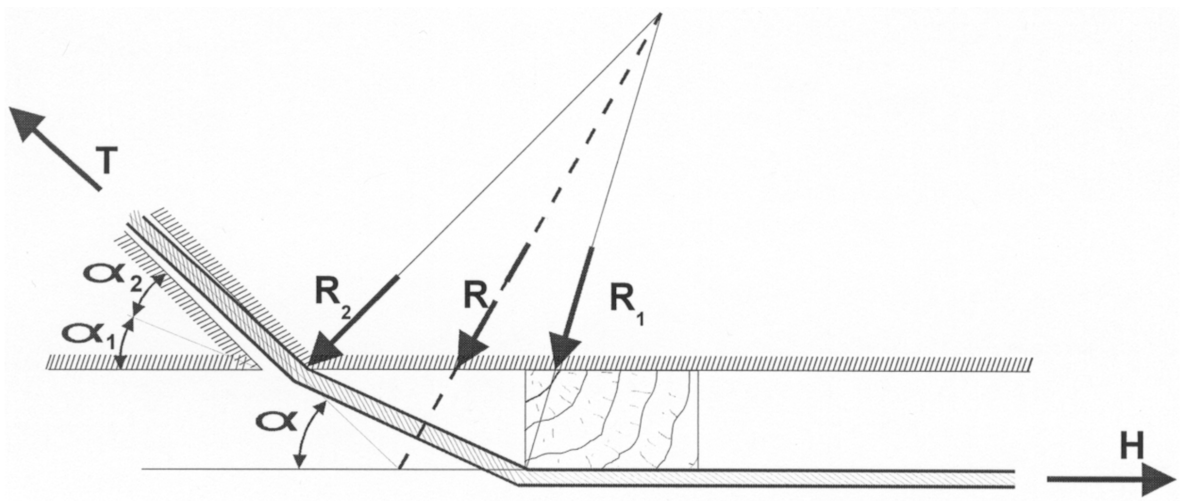


Figure 25.–Free body diagram of half cable truss, showing static loads that develop.

LABORATORY TESTS TO EVALUATE CABLE TRUSS PERFORMANCE

Laboratory investigations have been conducted to evaluate the loading characteristics of cable trusses where special load frames have been constructed to approximate the field conditions. The results from a series of tests conducted at the University of Pittsburgh indicate that only about 80% of the load is transferred from the angle member to the horizontal member as a result of friction across the contact blocks or bearing plates [Mangelsdorf 1979]. In these tests, the angle member was at a 45° angle to the horizontal. When tested to failure, the load in the diagonal cord member was 88.9 kN (20,000 lbf) where the ultimate strength of the 1.27-cm (0.5-in) in diameter cable was 102.3 kN (23,000 lbf). The angle member achieved only 87% of the ultimate load of the cable, with failure resulting from bending of the cable over the contact block, which caused a point of stress concentration and reduced the range of inelastic deformation of the truss. The truss had reached the yield point at the same approximate level as cable yield. At failure, the vertical load calculated from the load measured in the cable diagonal was 62.3 kN (14,000 lbf). The measured vertical stiffness for a half truss was 8.8 kN/cm (2.5 tons/in) and represents a stiffness of 17.5 kN/cm (5 tons/in) for the full truss.

Investigators at the University of West Virginia have also conducted laboratory tests with cable trusses in a specially designed truss frame [Oldsen et al. 1995]. The results of these tests were similar to those in the University of Pittsburgh study. This study did however, provide some further insight into frictional losses and load transfer between sections of the cable and applied loads. In these tests, a 1.52-cm (0.6-in) in diameter cable truss with an ultimate strength of 260 kN (58,600 lbf) was used, although the cables were not taken to failure. Again, the angle between the angle and horizontal members was 45°.

However, there are differences of interpretation of the data regarding the vertical load capabilities of the truss. In the University of West Virginia report, it is stated that the load on the diagonal is 222 kN (50,000 lbf) when the total applied vertical load or plate loads is 400 kN (90,000 lbf). There are two problems with this interpretation. First, the data are being extrapolated beyond the actual test data. Second, the plate loads are assumed to be the vertical loads. Extrapolation beyond the test data can at times be questionable, while using loads applied at the plates as vertical stress involves uncertainties about the frictional conditions within both the jacks used to load the trusses and the test frame, as well as the angle of the applied load between the test frame and the cable. Essentially, the only reliable measurement of vertical load should be that calculated from the diagonal member. By using the diagonal load, the result is that the vertical truss load is only 311 kN (70,000 lbf). Assuming this is near cable failure, the ratio of a vertical load of 311 kN (70,000 lbf) to an ultimate cable load of 260 kN (58,600 lbf) is 120%. The ratio of the vertical load to the ultimate cable strength from the University of Pittsburgh tests was 124.6:102.3 kN (28,000:23,000 lb) or 122%. These calculations were based on symmetrical loading at the plates.

FIELD EVALUATION OF CABLE TRUSS PERFORMANCE

Headgate

Trusses are now being used extensively to provide supplemental support to the headgate entry where the damage to the headgate entry is often the result of high horizontal stresses [Mark et al. 1998; Oldsen et al. 1995]. The ability of cable trusses to handle headgate conditions is illustrated by the following case.

A mine located in western Colorado had roof damage in the headgate ahead of the face as the panel was mined [Dolinar et al. 1996]. This was the result of horizontal stress concentration ahead of the face and geologic features susceptible to stress damage. (See case 1 for a more detailed description of the geology.) A single-piece cable truss with a diameter of 1.52 cm (0.6 in) was installed on 1.2-m (4-ft) centers in the headgate entry (figure 26). To evaluate loading during installation and as the panel was mined, special cable strain gages were installed on the horizontal section of some of the trusses. The installed load on the trusses ranged from 15.1 to 36.9 kN (3,400 to 8,200 lbf). From mining, the maximum load was 74.7 kN (16,800 lbf) for a truss just inby the face, an increase of 55.2 kN (12,400 lbf). This shows that the cable trusses were loading and resisting the roof movement. The cable trusses were able to control the roof conditions that developed in the headgate successfully despite the lateral movement and roof damage (figure 27).

Other investigators have measured 17.8 to 26.7 kN (4,000 to 6,000 lbf) of increase resulting from horizontal stress damage and cutters in headgate situations [Oldsen et al. 1997]. In these cases, the cable trusses also successfully controlled the roof. No cables trusses failed while the ridged trusses had. However, if failure progresses a sufficient depth into the roof, the dead weight load of the rock could exceed truss capacity. This occurred at a mine in western Kentucky where cable trusses were installed on 1.2-m (4-ft) centers [Miller 1996]. The roof failed to a rider seam when the distance to the rider was under 3 m (10 ft) and resulted in truss failure when the weight of the rock exceeded truss capacity.

Tailgate Support

Rigid trusses have been successfully tested as the only secondary support in a tailgate at a test area established in a mine in southwest Pennsylvania [Stankus et al. 1994]. In the test, a section of tailgate entry 112.7 kN (370 ft) long was supported by trusses on 1.2-m (4-ft) centers. Loads on the horizontal members increased by 44.5 kN (10,000 lbf) in the abutment zone. Behind the shields, the roof did stay up for a distance of 7.3 to 9.1 m (24 to 30 ft).

Cable trusses in combination with cable bolts have also been tested as the main secondary support in a section of tailgate at another mine in southwestern Pennsylvania [Molinda et al. 1997]. In this case, the cable trusses were installed on 2.4-m (8-ft) centers and supplemented with one row of 3.7-m- (12- ft) long cable bolts placed along the pillar side of the entry on 1.8-m (6-ft) centers. The tailgate outby the face and along the shields stayed open with only minor damage to the roof being noted. The maximum roof separation measured was just under 2.5 cm (1 in). Behind the shields, the trusses failed almost immediately because of the cave and only about 25% of the entry remained open alongside the pillar for ventilation. This small section was kept open by the cable bolts. Even this small airway appears to have been closed off about three-fourths of the way to the crosscut behind the face.

Design of Cable Truss Systems

Because cable trusses have a low vertical stiffness and are very flexible, they can deform to the shape of the roof. This, in combination with the high strength of the cable, makes the truss an excellent support where especially large lateral deformation occurs. From the laboratory tests, measured vertical truss stiffness of 17.5 kN/cm (5 tons/in) is significantly lower than the stiffness of a cable bolt or a cable bolt system, where up to four cable bolts would be used in place of the cable truss.

The loading of a truss is complex; however, based on laboratory work, the total amount of vertical load or dead weight the cable truss can sustain appears to be about 120% of the ultimate strength of the cable or about 311 kN (70,000 lbf) for a 1.52-cm (0.6-in) in diameter cable with symmetrical loading at the bearing plates. Failure of the cable truss will usually occur between the anchor hole and the bearing plate when cable tension load in the angle section is around 87% of the ultimate strength of the cable (figure 28) [Tadolini et al. 1998]. Furthermore, a cable truss in situ can be subject to asymmetrical loading, resulting in an even lower load capacity. Therefore, to determine the performance of a cable truss in the field, it would be necessary to measure the strains or loads on all three sections of the cable as well as measuring roof sag [Mangelsdorf 1979]. For cable trusses in general, strain measurements are usually determined only on the horizontal member. Thus, a complete picture on the performance of cable trusses in situ has not been obtained.

Generally, trusses are installed between the existing rows of primary roof support, so spacings will be on 1.2-or 1.5-m (4- or 5-ft) centers. However, if the roof failure is deep enough, the dead weight load of material can exceed truss capacity. Based on laboratory tests, for a 1.52- cm-(0.6-in) long cable with an ultimate strength of 260 kN (58,600 lbf) and the truss carrying a load 120% of cable strength, the dead weight load capacity is 313 kN (70,500 lbf). With 5.5-m (18-ft) wide opening, a 1.2-m (4-ft) truss spacing, and a rock density of 2,307 kg/m^3 (144 lbf/ft^3), a failure depth of about 2.1 m (7 ft) would exceed this capacity. In such cases, either tighter truss spacing, higher capacity trusses, or additional supplemental support must be used in conjunction with the truss system.

For the cable trusses, anchorage requirements are the same as for a cable bolt where a minimum of 1.2 to 1.5 m (4 to 5 ft) of anchorage length should be used. The trusses are anchored in angle holes that are usually drilled at a 45° angle over the coal rib. Other angles can be used, but this will affect loading and load distribution in the truss. These angle holes allow the truss anchorage to be outside the potential failure zone. Once the anchorage is undercut so that when the trusses are behind the face, this is no longer the situation and the truss fails because of the loss of the anchor. Similar conditions may also develop in intersections or the shield zone where the angle member is not anchored above a coal rib, but in a potential failure zone.

Bearing plates are used and installed up to around 15 to 30 cm (6 to 12 in) from the anchor hole. These bearing plates allow two points of contact on the roof, lessens the cable bend, and allows for more efficient load transfer along the cable.

Cable trusses have been used successfully as supplemental support in the headgate to control damage from high horizontal stresses that can develop near the longwall face. The strength, low stiffness, and flexibility of the cable trusses are important characteristics that allow the support to survive and maintain control of a damaged and highly deformed roof. Other types of support, especially rigid trusses, have failed under conditions where large lateral movements occur. As the main secondary support in the tailgate, cable trusses have been relatively successful in a few test cases. However, the trusses do have trouble maintaining the tailgate open behind the shields because of loss of anchorage. Also, there are no data to indicate whether there is any loss of anchorage and therefore support as the cables are undermined between the face and the back of the shield. When mining through an intersection, either side of the truss could fail as a result of loss of anchorage because the truss is anchored in a potential failure zone.

Figure 26.–Cable trusses installed in the headgate entry.

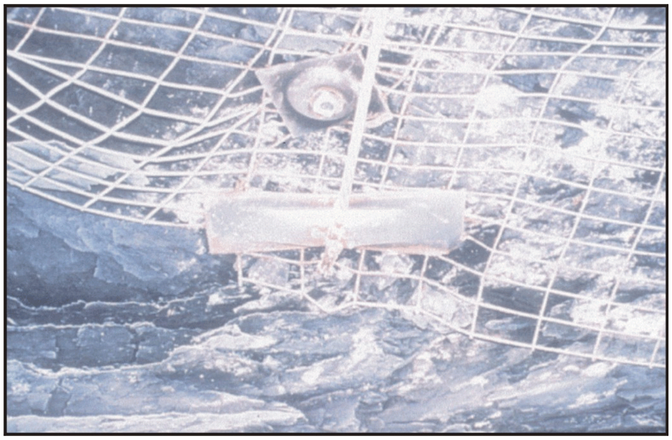


Figure 27.–Roof damage from horizontal stress around cable truss in headgate just outby the face.

Figure 28.–Shear failure of cable truss near bearing plate.

SUMMARY AND CONCLUSIONS

Cable technology as used in the longwall gate roads in U.S. coal mines was developed in the 1990's. This technology includes cable bolts and cable trusses. Cable bolts consist of a headed cable utilizing a partial grout column anchor formed from a resin cartridge and installed with a roof bolting machine.

When evaluating cable bolts, there are several characteristics and components of the cable system that are important to bolt performance. This includes cable strength, elongation, stiffness, and shear resistance, and system anchor capacity. In general, the cable should be the weakest part of the system. Therefore, to exceed the ultimate capacity of the cable, the anchorage length should be a minimum of 1.2 to 1.5 m (4 to 5 ft) long. The stiffness of the cable system, the ability to resist loading, is determined by the free length and elastic properties of the cable. As determined from in situ pull tests, the elastic modulus of the cable was found to be about 131.7 GPa (19.1 million lbf/in^2). This value can be used to calculate cable bolt stiffness. For improved long-term performance, galvanized wire strands or epoxy-coated cable should be used to resist cable corrosion and limit the potential for any strength reduction of the cable. Furthermore, high- capacity bearing plates and heavy-duty mats or channel provide added protection with surface control and an element of structural support for the immediate roof.

Design of cable bolt systems as secondary support in tailgate entries is based to a large extent on suspension, although this is somewhat of an oversimplification of the conditions that can develop. Cable lengths are determined by the depth in the roof of a potential failure horizon over the entry plus an adequate anchorage length. The number or density of cables will then be determined by the dead weight load of rock below that failure horizon. Lateral roof movement may also cause significant loads to develop in the cables where the cables resist the movement and increase the residual shear strength of the rock However, in some cases, the cables may not be able to stop or limit lateral movement and, as a result, can fail. It has been estimated that the cables can handle up to 5 to 10 cm (2 to 4 in) of lateral movement. There are measures that can be taken to reduce the potential for cable failure, including locating the bolts outside the highest lateral deformation zones or using yielding bolt heads.

For supporting tailgates, there are three zones that must be considered when evaluating the design and performance of the cable system. These zones include the outby abutment zone for both vertical and horizontal stress, the shield zone, and the cave zone. In situ tests have been used to further define and confirm cable bolt designs and performance in each of these zones. In these test cases, the number of cables used per row was four with 1.2- to 1.8-m (4-to 6-ft) row spacings. The cable lengths at the site varied from 3.7 to 4.9 m (12 to 16 ft). These cable bolt systems were very successful in supporting longwall tailgate entries with few resulting ground control problems.

From a ground control standpoint, cable bolts have an advantage over standing support where they do not resist main roof-to-floor convergence. This is especially important with a yield pillar system because much of the capacity of the standing support will be taken up by this convergence. Although cable bolts have maintained the tailgate entry behind the shield for long distances, the cave and roof geology are the main factors that determine this distance and not the cable system design. Therefore, cable bolts cannot guarantee that the tailgate can be kept open to the first crosscut behind the face for ventilation.

Cable trusses were greatly improved in conjunction with cable bolt development, and as a result, are now used more extensively than previously, especially as supplemental support in headgate entries. In the headgate, the cable truss has been used to control damage caused by horizontal stress. High strength and flexibility and low stiffness are reasons why trusses can survive in a high-stress and high-deformation environment and still function to maintain a highly deformed roof.

Furthermore, the cable truss anchorage is outside the potential roof failure zone. Therefore, cable trusses have been successful in providing supplemental support in critical headgate entries.

However, based on laboratory tests, the capacity of trusses to carry dead weight loads appears to be only about 120% of the ultimate strength of the cable. More tests, including in situ studies, are required to determine if this capacity could be used for design or must be modified.

To evaluate the performance and capacity of a truss in situ though requires monitoring loads on all three cable legs along with roof sag. Although trusses have been instrumented, to date this has not been done to the level required for a complete evaluation of their performance. As secondary support in the tailgate entry, cable trusses have been tested or used only on a limited basis. Behind the shields, trusses can only keep the tailgate open for a very limited distance. Beyond such distances, there are questions on how well heavily loaded trusses will perform when the anchors are undercut by mining or in intersections where anchors are not supported by a coal rib.

REFERENCES

Barczak TM, Zelanko JC, Molinda GM [1996]. Innovative secondary support systems for gate roads. In: Proceedings of Longwall U.S.A. Pittsburgh, PA: pp. 21-43.

Dolinar DR, Tadolini SC, Blackwell DV [1996]. High horizontal movements in longwall gate roads controlled by cable support systems. In: Ozdemir L, Hanna K, Haramy KY, Peng SS, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 497-509.

Gale WJ [1987]. Design considerations for reinforcement of coal mine roadways in Illawarra coal measures. AusIMM Ground Movement and Control Related to the Design of Coal Mines, Commonwealth Scientific and Industrial Research Organisation (CSIRO), Australia.

Gale WJ, Fabjanczyk MW [1987]. Application of field measurement techniques to the design of roof reinforcement systems in underground coal mines. Commonwealth Scientific and Industrial Research Organisation (CSIRO), Australia.

Goris JM [1990]. Laboratory evaluation of cable bolt systems (in two parts). 1. Evaluation of cable supports using conventional cables. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9308.

Goris JM [1991]. Laboratory evaluation of cable bolt systems (in two parts). 2. Evaluation of cable supports using conventional cables with steel buttons, birdcage cables and epoxy-coated cables. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9342.

Goris JM, Brady TM, Martin LA [1993]. Field evaluation of cable bolt support, Homestake mine, Lead, SD. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9474.

Goris JM, Martin LA, Curtin RP [1995]. Shear behavior of cable supports in horizontal bedded deposits. In: Proceedings of the 97th Annual General Meeting of CIM Rock Mechanics and Strata Control Session. Canadian Institute of Mining, Metallurgy and Petroleum: pp. 92-99.

Goris JM, Martin LA, Curtin RP [1996]. Shear behavior of cable supports in horizontal bedded deposits. In: Ozdemir L, Hanna K, Haramy KY, Peng S, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 497-509.

Goris JM, Nickson SD, Pakainis R [1994]. Cable colts in North America. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, IC 9402.

Kadnuck LM [1994]. Mine ventilation enhancement through the use of cable supports. SME preprint 94-36. Littleton, CO: Society for Mining, Metallurgy, and Exploration, Inc.

Karabin GJ, Hoch MT [1979]. Contemporary roof support systems provide a diversified approach to modern ground control. In: Proceedings of the 10th Annual Institute on Coal Mining Health and Safety Research. Blacksburg, VA: Virginia Polytechnic Institute and State University, pp. 249-267.

Koehler JR, DeMarco MJ, Marshall RJ, Fielder J [1996]. Performance evaluation of a cable bolted yield-abutment gateroad system at the Crandall Canyon No. 1 Mine, Genwall Resources, Inc., Huntington, Utah. In: Ozdemir L, Hanna K, Haramy KY, Peng S, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 477-495.

Mangelsdorf CP [1979]. Evaluation of roof trusses, phase 1. Pittsburgh, PA: University of Pittsburgh, Department of Civil Engineering. U.S. Bureau of Mines grant No. G0166088.

Mangelsdorf CP [1982]. Roof truss systems and there application ground. In: Proceedings of the Second Control in Room and Pillar Mining Conference. Carbondale, IL: Southern Illinois University, pp. 113-118.

Mark C, Mucho TP, Dolinar D [1998]. Horizontal stress and longwall headgate ground control. Min Eng *Jan*:61-68.

Martin L, Girard JM, Curtin RP [1996a]. Laboratory pull tests of resin-grouted cable bolts. In: Ozdemir L, Hanna K, Haramy KY, Peng S, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 455-468.

Martin L, Girard JM, Goris JM [1996b]. Installation and safety practices for cable bolts in underground mines. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, IC 9441.

McDonnell JP, Tadolini SC, DiGrado PE [1995]. Field evaluation of cable bolts for coal mine roof support. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, RI 9533.

Miller D [1996]. Progression of longwall gateroad support as conditions change at Lodestar Energy's Baker Mine. In: Ozdemir L, Hanna K, Haramy KY, Peng S, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 212-220.

Molinda GM, Dolinar DR, Barczak TM, Hustus J [1997]. Performance of various standing and cribless tailgate support configurations in a large eastern coal mine. In: Peng SS, ed. Proceedings of the 16th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 9-15.

Mucho TP, Barczak TM, Dolinar DR., Bower J, Bryja J [1999]. Design methodology for longwall tailgate secondary support. In: Proceedings of Longwall U.S.A. Pittsburgh, PA: pp. 150-170.

Mucho TP, Compton CS, Oyler DC, Zelanko JC, Horvath S [1996]. An evaluation of strata behavior and tailgate support performance of Eagle Nest mine. In: Ozdemir L, Hanna K, Haramy KY, Peng SS, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 523-526.

Oldsen, JG, Stankus J, Gou S, Khair AW [1997]. Continuing development of innovative cable support systems. In: Peng SS, ed. Proceedings of the 16th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 117-129.

Scott JJ [1989]. Roof bolting a sophisticated art. Coal Age, August.

Scott JJ [1994]. An entry support system to eliminate the need for timber cribbing. In: Proceedings of the Fifth Conference on Ground Control for Midwest U.S. Coal Mines, pp. 179-189.

Serbousek MO, Signer SP [1987]. Linear load-transfer mechanics of fully grouted roof bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9135, 17 pp.

Stankus JC, McCaffrey JJ, Peng SS [1994]. Innovative concept in tailgate entry support: elimination of crib blocks through utilization of new high capacity roof truss systems. In: Peng SS, ed. Proceedings of the 13th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 166-174.

Tadolini SC, Koch RL [1993]. Cable Support for improved longwall gateroad stability. In: Peng SS, ed. Proceedings of the 12th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 9-15.

Tadolini SC, Koch RL [1994]. Resin-grouted cables for longwall tailgate support stability. In: Peng SS, ed. Proceedings of the 13th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 175-184.

Tadolini SC, McDonnell JP [1998]. Recent developments in cable support technology. SME preprint 98-26. Littleton, CO: Society for Mining, Metallurgy, and Exploration, Inc.

Tadolini SC, Trackemas JD [1995]. Cable support for longwall gateroad stability. In: Peng SS, ed. Proceedings of the 14th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 9-17.

Vandekraats JD Watson SO [1996]. Direct laboratory tensile testing of select yielding rock bolt systems. In: Ozdemir L, Hanna K, Haramy KY, Peng SS, eds. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 321-332.

Wilson AH [1972]. A hypothesis concerning pillar stability. Min Eng *141*:409-417.

Wire Rope Technical Board [1985]. Wire rope users manual, 2nd ed. Stevensville, MD: American Iron and Steel Institute.

Zelanko JC, Mucho TP, Compton CS, Long LE, Bailey PE [1995]. Parameters affecting resin-anchored cable bolt performance: results of in situ evaluation. In: Peng SS, ed. Proceedings of the 14th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 497-509.

MATERIAL HANDLING CONSIDERATIONS FOR SECONDARY ROOF SUPPORT SYSTEMS

By Thomas M. Barczak[1]

ABSTRACT

Secondary roof support systems play a vital role in preserving the safety of underground mine workers by preventing the unintentional collapse of the mine roof. Hundreds of thousands of standing roof supports are constructed each year in underground coal mines. Historically, wood and concrete cribs and timber posts have been used for secondary roof support. These support constructions require the handling of heavy and bulky materials, causing numerous injuries to the mine workers and resulting in more than 40,000 lost workdays in the past 9 years. Since 1993, various alternative support technologies have been developed. These new support technologies not only provide superior roof support, but many also have significant material handling advantages by using smaller and lighter weight materials, fewer components, mechanically installed support systems, and pumpable support systems. The National Institute for Occupational Safety and Health (NIOSH) has conducted extensive material handling research and has developed recommended lifting thresholds to reduce material handling injuries. An analysis of roof support construction reveals that conventional support materials used in wood and concrete cribbing exceed the recommended lifting thresholds, while the engineered support systems are closer to the recommended weight thresholds. Finally, recommendations are made relative to proper lifting techniques and material handling practices to prevent injury to mine workers when constructing roof supports in underground coal mines.

[1]Research physicist, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh PA.

INTRODUCTION

The primary goal of roof support is to prevent unplanned roof falls from occurring. While falls of ground are the leading cause of *fatalities* in underground coal mining, many more *injuries* from the effort required to install these essential roof support systems than from the fall of ground due to poor roof support design. There is also continued pressure to install supports more quickly to keep pace with the escalating productivity, particularly in modern longwall mining operations. These requirements, coupled with the fact that the workforce is aging, strongly suggest that a higher priority should be given to the material handling considerations and construction practices during the roof support selection process.

During the 1990s, there was an unprecedented increase in the development of standing roof support systems to replace conventional wood and concrete cribbing and timber posts that have historically been used for secondary roof support throughout the mine. These innovative roof support systems were designed to provide superior roof control, but most also provide material handling benefits that allow supports to be constructed with less effort and at faster rates.

This paper highlights the material handling characteristics of these modern roof support systems and describes their impact on the installation of secondary roof support and safety of the mine workers performing this function. A complete assessment of the transportation and construction requirements for each support is made to provide mine operators with a guide for evaluating these alternative support technologies. Regardless of the physical properties of the support, injuries can also be prevented by using proper lifting techniques and avoiding excessive stress that occurs when recommended lifting thresholds are exceeded. These issues are also addressed through some practical examples.

INJURY INFORMATION

According to the Mine Safety and Health Administration (MSHA) accident database, over 55% of the permanent disabling injuries in underground coal mining during 1992-94 were due to material handling [MSHA 1996]. An even greater number of nondisabling injuries may be linked to material handling in mining. It is likely that these accident trends will continue and perhaps grow worse due to an aging workforce. It is also very likely that younger and inexperienced miners that will replace retiring miners will also experience a high incidence rate for these kind of accidents until they become more skilled.

The construction of secondary support systems has historically required repetitive lifting of large volumes of bulky support materials. Coupled with the poor conditions, underground limited space and maneuverability, and the fact that many of the support materials exceed 40 lb, this activity is responsible for numerous injuries to mine workers. Such activity has been classified by the National Occupational Health Survey of Mining as a heavy lifting risk factor, exposing miners performing this activity to a high risk for musculoskeletal repetitive trauma disorders [Winn and Biersner 1996]. Thus, material handling should also be a primary consideration in the application of longwall tailgate support technologies.

A review of the MSHA accident database reveals that 1,483 lost-time accidents were reported from 1990 to 1998 associated with timber handling. Further review of these accidents indicates that 1,204 were directly related to support construction, accounting for 40,147 lost workdays during this 9-year period. The average lost time per incident was 33.34 days. Figure 1 shows the number of material handling accidents attributed to crib construction from 1990 to 1998; figure 2 shows the incidence rate for the same period. Generally, both the number of accidents as well as the incidence rate dropped during this period, except for a moderate increase in 1998. Without more extensive data than are available in the current database, there is no apparent reason for the increase in 1998. It could be that more attention has been drawn to material handling issues recently, and minor incidents that were previously not reported or mislabeled are now being reported. It might also be a 1-year anomaly.

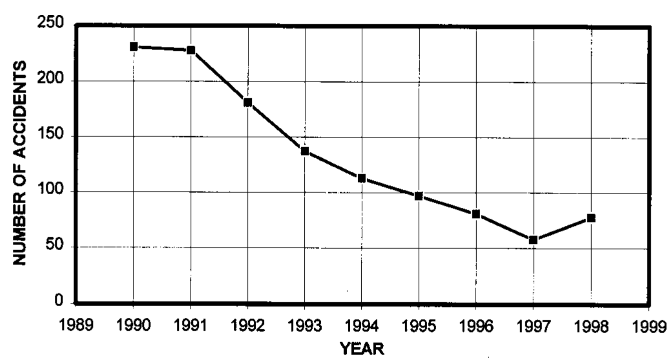


Figure 1.–Number of timber-handling accidents.

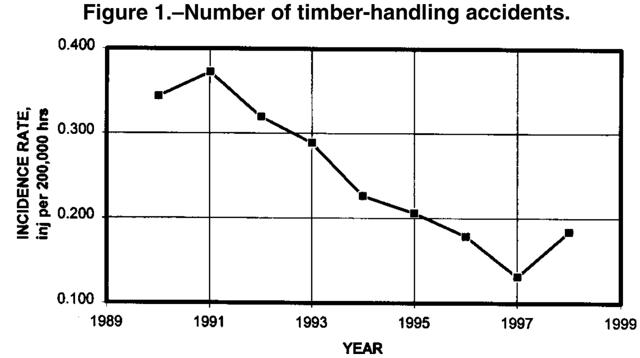


Figure 2.–Incident rate for timber-handling injuries.

The decrease in injuries from roof support construction correlates with the implementation of new support technologies that exhibit material handling advantages over conventional wood cribbing. These alternative support systems are addressed in detail in the remainder of the paper. Before 1992, all longwall tailgates used either conventional wood or concrete cribbing. As shown in figure 3, only 39% of the longwall tailgates were supported with conventional wood or concrete cribbing in 1996. Figure 4*A* compares the incidence rate for timber material handling injuries with the replacement of conventional cribbing with engineered timber supports. As seen, the trend of decreasing material accidents continues during the period when new support technologies were introduced. Figure 4*B* shows that the severity of injury decreased considerably after 1994, correlating with the implementation of the engineered alternative support systems. These findings suggest that these lighter weight support materials are having a positive impact on reducing material injuries associated with support construction in underground coal mines.

Figure 5 shows a breakdown of lost workdays due to timber handling injuries in underground coal mines. As might be expected, 71% of the lost workdays are due to back-related injuries. Figure 6 depicts the number of lost workdays per

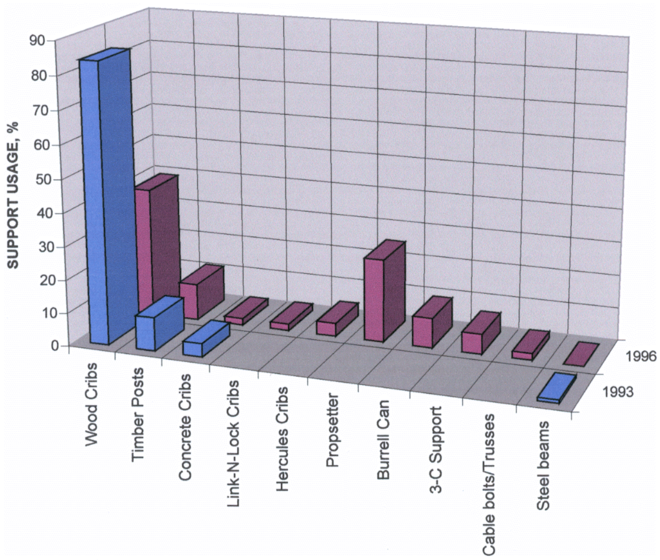


Figure 3.–Comparison of support technologies used in 1993 and 1996.

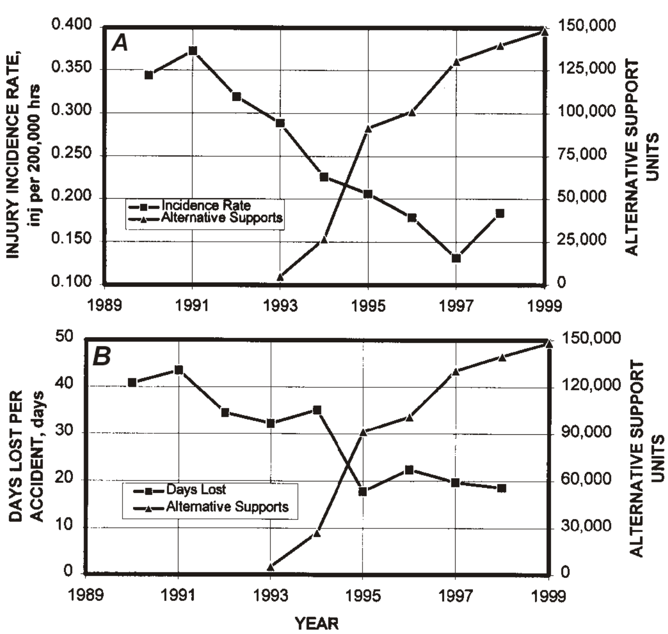


Figure 4.–A, Comparison of timber-handling incident rate with the implementation of alternative engineered timber support systems. B, Comparison of days lost due to timber handling with the implementation of alternative engineered timber support systems.

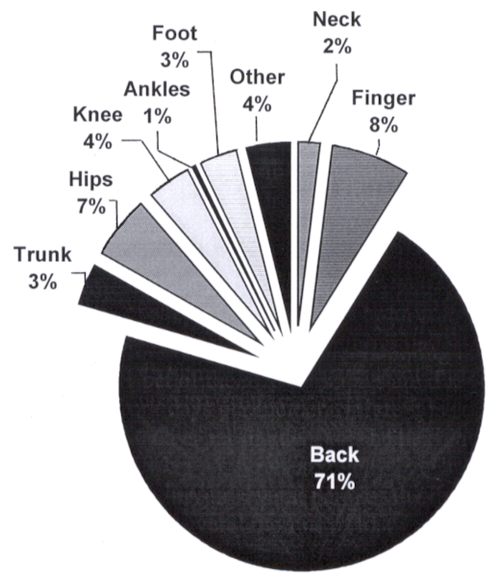


Figure 5.–Assessment of lost workdays relative to body part for timber-handling injuries.

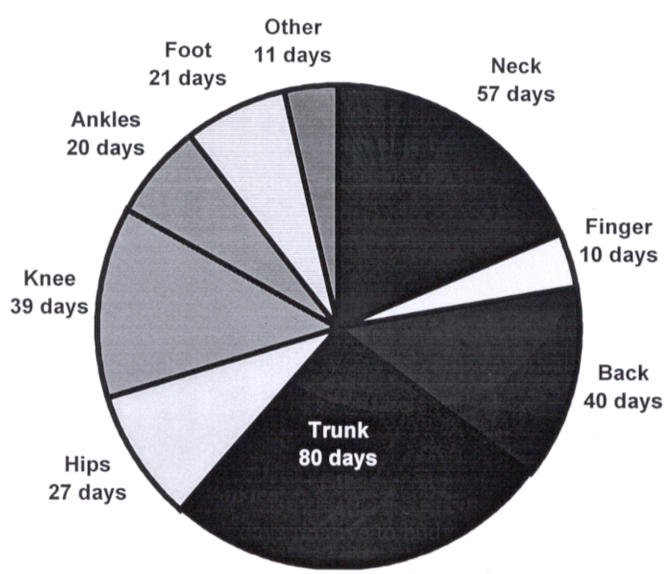


Figure 6.–Bodily assessment of lost workdays per incident for timber handling accidents.

incident. Here it is seen that the most severe injuries are to the trunk of the body (80 days per incident). The next most severe injury is to the neck (57 days per incident), followed by back injuries at 40 days per incident. In conclusion, timber handling activities can result in injury to several parts of the body, and many injuries will be severe enough to cause several days to several months of lost work time.

SUPPORT DESIGN IMPROVEMENTS THAT REDUCE MATERIAL HANDLING REQUIREMENTS

Conventional wood and concrete cribs require piecemeal construction of relatively heavy and bulky materials weighing 35 to 55 lb. Significant material handling advantages are realized in various alternative support technologies by the use of lighter weight and more compact support components; other support systems use fewer components in the support construction. Manual material handling is all but eliminated in supports that are installed with a machine. Innovative supports that are pumped in place in the mine greatly reduce transportation requirements and minimize manual effort in the confined space of the underground mine where material handling efforts are considerably more difficult than on the surface.

SMALLER AND LIGHTER WEIGHT COMPONENTS

Conventional wood cribs are typically constructed in four-point or nine-point configurations, as shown in figure 7. In these configurations, the roof load is carried only through the area where the adjacent timbers contact one another. For example, in a four-point wood crib, this occurs only at the four corners of the support structure. As seen in figure 7, over 50% of the timber *does not* contribute to the capacity of the support. Modern engineered timber supports, such as the Link-N-Lock crib and the Link-N-X crib developed by Strata Products USA and the Tri-Log crib developed by American Commercial, Inc., as shown in figure 8, achieve full timber contact by notching the timbers. By establishing full timber contact, the timber dimensions can be reduced without sacrificing supporting capability. Table 1 compares the timber dimensions and weights for conventional wood cribs and selected engineered timber supports.

As shown in table 1, the timber weight is reduced by about a factor of about 3 for selected designs of the Link-N-Lock, Tri-Log, and Link-N-X cribs. In addition, these designs provide 1.6 to two times the support capacity of a conventional four-point oak crib. Comparing these engineered timber supports to a four-point crib constructed from poplar timbers, which most closely represents that of a mixed hardwood crib, it is determined that despite half the timber weight, the support capacity is increased by a factor of 2.4 to 3.0 for the engineered crib support structures.

CONSISTENT MATERIALS

Studies have shown that one of the risk factors for back injuries is unexpected or unanticipated movements [Marras et al. 1987]. When materials are of different and unknown weights, there is an increased chance of back injury. Thus, a miner constructing conventional wood crib supports from timbers consisting of mixed wood species where one piece of wood may weigh 30 lb and the next piece may weigh 50 lb is more prone to injury than if the timbers are all of the same weight. Engineered timber supports such as the Link-N-Lock, Link-N-X, Hercules, and Tri-Log cribs all use the same species of timber for the support construction. Thus, these timbers are much closer in weight than the mixed wood species used in conventional wood crib supports. This reduces the injury potential that occurs when a miner is expecting a light piece of wood, but instead gets a much heavier piece of wood during the support construction process.

FEWER COMPONENTS

The Hercules crib is an example of a support that is designed to provide superior support capability with fewer pieces required for the support construction. It is constructed from preformed mats that are stacked on top of one another (figure 9). The Hercules crib can be constructed in a variety of configurations to provide a wide range of support designs and

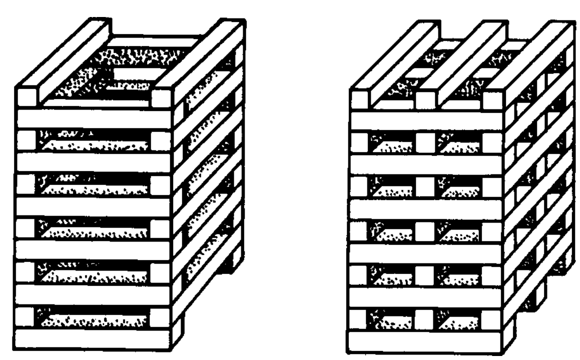


Figure 7.–Construction of four-point and nine-point conventional wood crib supports.

A *B* *C*

Figure 8.–*A*, Link-N-Lock crib; *B*, Tri-Log crib, and *C*, Link-N-X crib.

Table 1.—Comparison of conventional wood cribbing to engineered crib support systems

Parameter	Four-point oak crib	Link-N-Lock (24-in)	Tri-Log (30-in)	Link-N-X (30-in)	Four-point poplar crib
Timber length, in	36	24	30	30	36
Timber width, in	6	3.5	4	6	6
Timber rise, in	6	6	6	4	6
Timber weight, lb	40.5	13.1	19.1	19.1	31.6
Support weight per foot of height, lb/ft	208	102	88	90	126
Capacity at 2 in of convergence, tons	52	94	105	85	35

performance profiles. The comparison in table 2 shows the HM-9(308) provides 38% more capacity than a four-point oak crib. The HM-9(308) mat weighs slightly more than a 6- by 6- by 36-in oak timber, but since each mat provides a full layer in the Hercules support, the material weight per foot is 18% less than a four-point crib.

There is also a variety of prop-type supports that are constructed as units rather than the piecemeal construction required for crib-type supports (figure 10). The unit weights of these supports are typically greater than the unit weights of the piecemeal crib components, but since they are installed as unit and generally stood in place as opposed to being lifted, the cumulative effort to install the support is considerably less. In addition, the construction of the prop-type supports does not require lifting of material above the shoulder and thus reduces the risk of injury by eliminating this awkward lifting condition. The primary advantages of the prop supports from a material handling perspective are twofold: (1) supply cars can transport more support units and (2) supports can be installed in less time than crib-type supports since there are considerably fewer pieces to handle. Table 3 documents transport volumes and installation rates of various prop-type supports in comparison to conventional wood cribbing.

MECHANICALLY INSTALLED SUPPORT SYSTEMS

Other support systems are installed with machines that all but eliminate the material handling effort by the miners. Table 4 describes relevant material handling parameters for three mechanically installed support systems. The Burrell Can developed by Burrell Mining Products pioneered the development of mechanically installed supports. The Burrell Can is installed with a hydraulically powered gripper claw that can be attached to a loading machine or a scoop (figure 11*A*). Strata Products has recently developed the Star Prop that can be installed with the aid of the Prop Handler or the Microtrax. Figure 11*B* shows the installation of Propsetter supports using these mechanical aids.

Table 2.—Comparison of a Hercules crib to conventional four-point oak crib

Parameter	Nine-point oak crib (6x6x36 in timbers)	Hercules crib HM-9(308)
Timber weight, lb	40	44
Support weight per foot of height, lb/ft	162	132
Capacity at 2 in of convergence, tons	52	72

Table 3.—Comparison of prop-type supports to conventional four-point wood crib

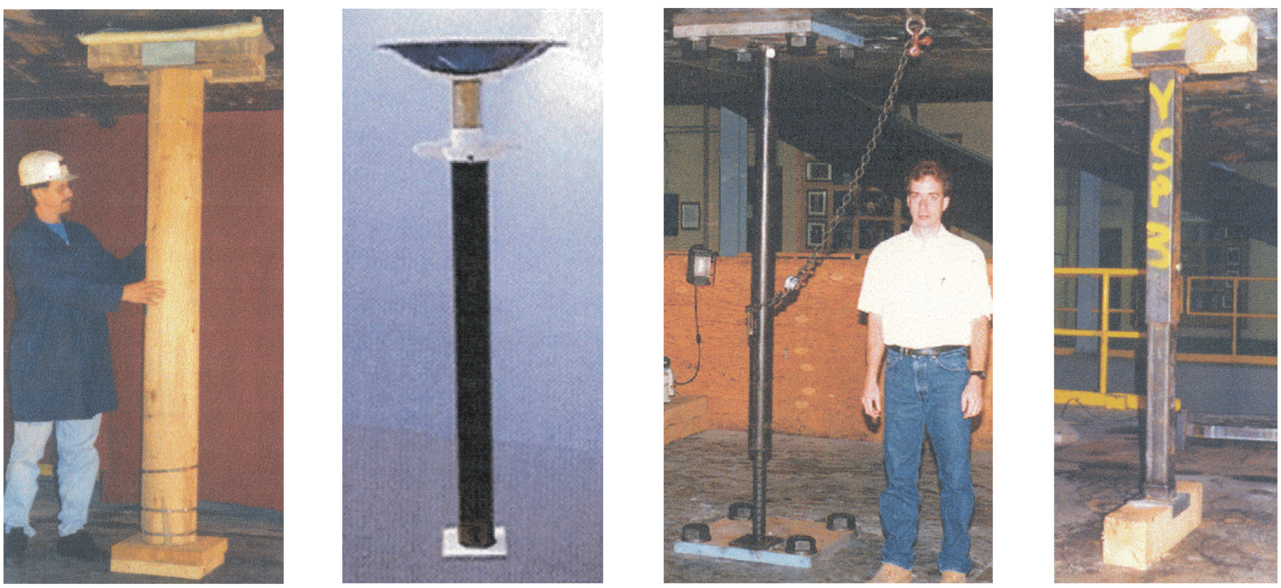
Parameter	Four-point crib	Propsetter	ACS	Yippi prop	Stretch prop
Transport volume, ft^3	24	4.6	4.3	2.8	3.7
Supports per supply car	16	84	91	140	107
Installation rate, supports per shift	15	48	53	80	60

Figure 9.–Hercules crib.

Table 4.—Comparison of some mechanically installed support systems

Parameter	Burrell Can (24-in diam)	Propsetter (10-in diam)	Star Prop (standard design)
Installation rate, supports per shift	[1]40	50	50
Transport numbers, supports per car	12	65	43
Capacity at 2 in of convergence, tons	85	68	85

[1]The installation rate of the Burrell Can support can vary considerably, depending on the availability of equipment for delivery of the supports and timbers for topping off the supports. The 40 supports per shift is a representative installation rate for Burrell Can supports that includes delivery time as part of the installation process in a well-planned operation; however, this rate can range from 30 to 50 supports per shift.

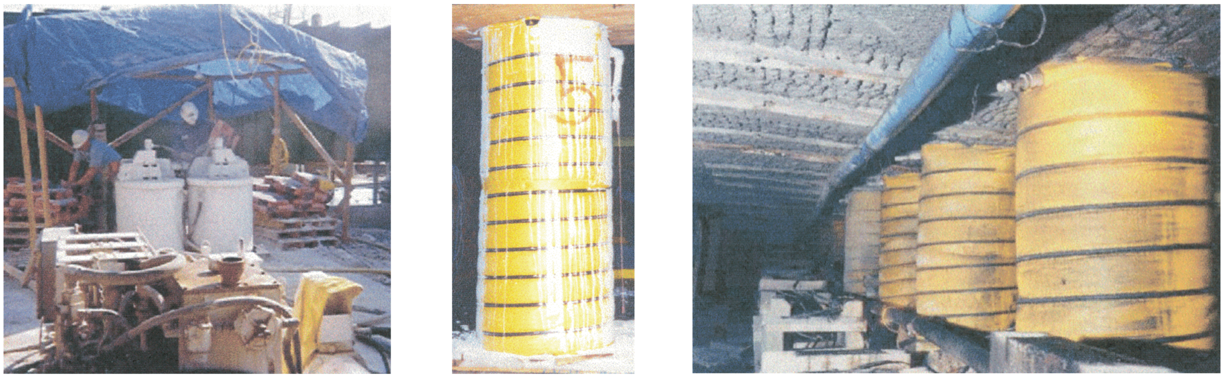


A *B* *C* *D*

Figure 10.–*A*, Propsetter; *B*, ACS; *C*, Yippi support; *D*, Stretch prop.

A *B*

Figure 11.–*A*, Installation of the Burrell Can; *B*, installation of the Propsetter support.



A B C

Figure 12.–*A*, Surface pumping station for Pumpable Crib; *B*, Pumpable Crib; and *C*, underground installation of Pumpable Crib in longwall tailgate.

PUMPABLE ROOF SUPPORT SYSTEMS

Another technology that offers material handling advantages is the use of pumpable support systems where a cementitious grout is pumped in place into some form in the mine entry. This process greatly reduces the transportation effort and can reduce the material handling effort with support installation. Both Heintzmann Corp. (figure 12) and Fosroc Corp. offer pumpable support systems. The Stretch Prop (figure 10*D*) developed by Ferrocraft, Inc., is a prop-type support that uses a pumpable cementitious grout to extend and fill the support during the installation process. Table 5 compares the Heintzmann Corp.'s pumpable crib to a conventional four-point oak crib.

Table 5.—Comparison of pumpable crib and conventional Four-point oak crib

Parameter	four-point oak crib	Pumpable crib
Supports per supply car	16	100
Construction work, ft-lb	5,838	900
Installation rate, supports per shift . . .	15	[1]50
Capacity at 2 in of convergence, tons	52	240

[1]The installation of the pumpable crib in this example is based on a surface pumping operation and a underground crew using a total of 7 people. The installation rates may vary depending on the crew size and the pumping requirements, but the 50 supports per shift is considered to be representative of a typical operation.

RECOMMENDED LIFTING THRESHOLDS TO REDUCE MATERIAL HANDLING INJURIES

Low-back pain and associated injuries from lifting are the most costly occupational health problems facing our Nation [NIOSH 1997]. As a result, the National Institute for Occupational Safety and Health (NIOSH) has conducted extensive ergonomic research to evaluate lifting mechanics and their impact on the human body. Through this research, a lifting equation has been developed that determines a recommended weight limit (RWL) for lifting in various postures [Waters et al. 1993]. The RWL is defined for a specific set of task conditions as the weight of the load that nearly all healthy workers could perform over a substantial period of time (e.g., up to 8 hr) without an increased risk of developing lifting-related lower back pain.

$$RWL = LC \times HM \times VM \times DM \times AM \times FM \times CM,$$

where RWL = recommended weight limit;

LC = load constant = 51 lb; and HM, VM, DM, AM, FM, and CM are various multipliers.

The lifting equation begins with a load constant (LC) of 51 lb, which is the maximum recommended lifting weight under ideal conditions. Interpreted as the most optimal conditions, no more than 51 lb should be lifted regardless of the task characteristics. Putting this into perspective relative to support construction, a 6- by 6- by 45-in oak timber that has been dried for about 30 days weighs 52 lb, whereas a green 6- by 6- by 36-in oak timber weighs as much as 52 lb. The 51-lb upper limit is then decreased through six multiplicative coefficients that further define the lifting task. The variables that impact the recommended weight limit are defined as follows:

1. *Horizontal Position (HM):* The horizontal position refers to the horizontal distance of the lifted object (where the person holds the object) from the centerline of the person's feet or, more specifically, the ankles (figures 13-15). The recommended weight limit is reduced as the object's distance from the body increases.

2. *Vertical Position (VM):* The vertical position refers to the distance the object is from the floor level before lifting

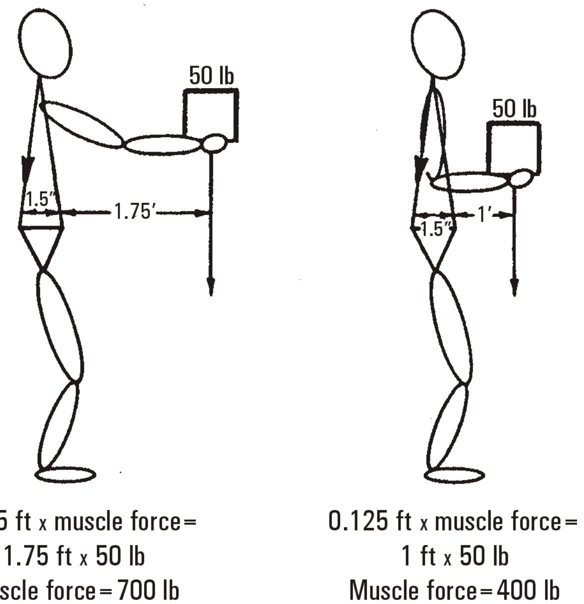


Figure 13.–Biomechanics of lifting showing effect of horizontal distance on muscle force.

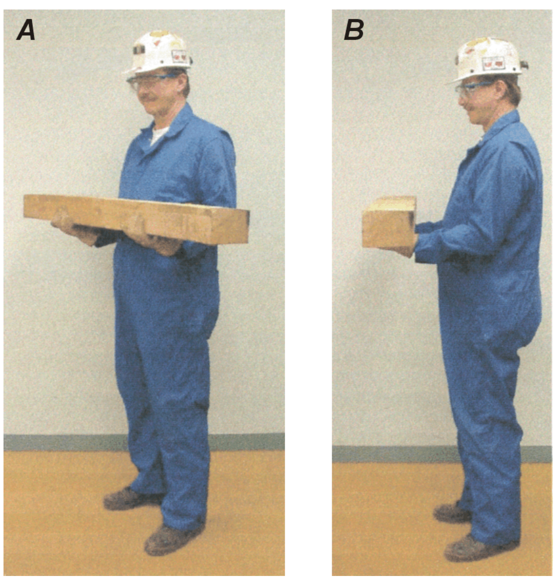


Figure 14.–Material should be kept close to the body (*A*) as the material is held away from the body (*B*) the risk of injury increases.

is executed. The optimum distance is 30 in for an average worker who is 5 ft 6 in tall [Waters et al. 1994]. The vertical position multiplier decreases linearly with an increase or decrease in height from the optimal 30-in position.

3. *Lifting Distance (DM):* The vertical lifting distance is how high the object is lifted. The recommended weight limit is reduced as the lifting height increases. The multiplier ranges from 1.0 for a height of 10 in to 0.85 for a maximum height of 70 in.

4. *Twisting Factor (AM):* The twisting factor is referred to as the asymmetric component, which is defined as the rotation

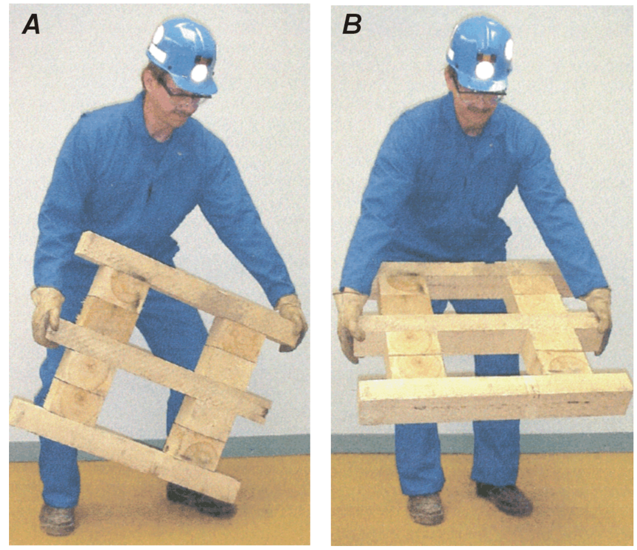


Figure 15.–Picking up a 40-lb Hercules mat. Holding the mat so that it is positioned close to the body (*A*) reduces the lifting effort compared to lifting the mat in a horizontal position (*B*) where the center of gravity is farther from the body.

Figure 16.–Avoid twisting.

(twisting) of the body required to place the object at a destination (figure 16). It is expressed in angular degrees relative to the feet position at the origination of the lift and the twisting of the body at the lift destination. The recommended weight limit can be decreased significantly for asymmetric lifting compared to symmetric lifting.

5. *Frequency Factor (FM):* The lifting frequency refers to the average number of lifts in typical 15-min period. In addition to the number of lifts, the lifting frequency multiplier is influenced by the duration of the lift and the vertical position of the hands at the origin of the lift. The duration of the lift is classified into three categories: (1) short, (2) moderate, and (3) long.

These categories are based on the work/rest pattern of the job. Short duration defines lifting tasks that have a duration of 1 hr or less followed by a recovery (nonlifting) time equal to 1.2 times the work time. Moderate duration includes lifting periods of 1 to 2 hr followed by a recovery time of 0.3 times the work time. Long duration is defined as lifting periods between 2 and 8 hr, with standard industrial rests allowances (morning, lunch, and afternoon breaks). As one would expect, the lifting limits are reduced as the frequency of lifting and/or the lifting period increase.

6. *Handling Factor (CM):* The handling factor technically referred to as the "coupling component" attempts to evaluate the worker's gripping method used on the object. It also considers the vertical location of the hands throughout the entire range of the lift. The coupling component is also divided into three categories: (1) poor, (2) fair, and (3) good. The coupling multiplier ranges from 0.9 to 1.0.

Some representative support construction examples are shown in table 6. Analyzing these data, it can be shown that the lifting threshold is most sensitive to how close the support material is held to the body during the lifting process (horizontal factor). The recommended lifting weight is reduced from 42 to 14 lb when the horizontal distance increases from 0 to 12 in with an initial horizontal origin 6 in from the body (feet) centerline. From a biomechanics analysis as shown in figure 13, it is apparent that muscle force greatly increases as the object is moved farther from the spine. The lifting threshold is also reduced when the body must turn to position the support material during the lifting process. For example, with zero horizontal lifting the recommended lifting weight decreases from 42 to 36 lb when the angular rotation changes from 0° to 45°. Twisting causes increased stresses in the intervertebral disks in the spinal column [Gallagher et al. 1990]. Excessive twisting may cause some of the disk fibers to break, which severely weakens the disk. It is the opinion of some researchers that back injuries caused by twisting are much less likely to heal than injuries due to simple bending [Gracovetsky and Farfan 1986].

The reduced material handling requirements for the engineered timber supports are clearly seen when these factors are considered. Reviewing a previous example, a typical wood crib is constructed from oak timbers weighing 40 lb while the engineered Link-N-Lock (24 in) and Tri-Log cribs (30 in) are constructed from timbers that weigh 13.1 and 19.1 lb, respectively. Therefore, while the standard crib exceeds the recommended lifting thresholds in all but ideal conditions, the Link-N-Lock and Tri-Lock crib meet the typical and extreme conditions. Thus, significantly less musculoskeletal injuries would be expected from the construction of these engineered timber supports compared to conventional wood cribbing.

Table 6.—Recommended lifting weights for selected task parameters relevant to roof support construction

Lifting height, in	Horizontal hand position from centerline of feet, in		Lifting frequency, lifts/min	Asymmetric angle, °	Recommended liftingweight, lb
	Origin	Destination			
60	6	6	1	0	42
60	6	12	1	0	21
60	6	18	1	0	14
60	12	12	1	0	21
60	12	18	1	0	14
60	18	18	1	0	14
60	6	6	1	30	38
60	6	6	1	45	36
60	12	12	1	30	19
60	12	12	1	45	18
60	18	18	1	30	13
60	18	18	1	45	12

INJURY PREVENTION THROUGH PRACTICAL LIFTING TECHNIQUES AND MATERIAL HANDLING PRACTICES

Miners constructing roof supports in an underground mine environment are at risk of musculoskeletal injuries. As previously described, these injuries can be minor or severe enough to cause permanent disability. Although NIOSH is conducting research to promote the development and application of lightweight materials and mechanical aids to reduce the effort required for support construction, injuries can also be prevented by following proper lifting techniques and support construction practices. Following are several practical recommendations to this effect.

1. Hold the support material as close to the body as possible.

As discussed previously, the recommended lifting weights decrease significantly when the lifted object is moved away from the body. Thus, material handling practices that facilitate keeping the material close to the body, as shown in figure 14*A*, should be seriously pursued. Handling material in the manner shown in figure 14*B* should be avoided, particularly when the material weight exceeds 20 lb. Another example is shown in figure 15 with the Hercules crib mat. Ideally, two people should lift the mat. If this is not possible, then lifting the mat in a vertical orientation is better than in the horizontal position. Another good practice is to position oneself as close as possible to the destination of the support component during the lifting process. For example, when constructing a crib, it is better to walk to the side of the crib where the timber is being installed than to reach across the support structure to lift the crib block in place. Although this may take more time, it significantly reduces the stress on the back and, overall, will reduce fatigue so that longer construction times can be realized.

2. Avoid lifting above the shoulders.

When possible use a ladder or some other device to increase the standing height as opposed to lifting above the shoulder. Lifting effort increases significantly once the object is lifted above the waist, because the arms are now required to accomplish the lifting without any further benefit from the leg muscles. Lifting above the shoulder requires more energy and can create an additional risk factor due to awkward lifting where the arms might need to be extended or the body twisted. Lifting materials above the shoulders may also jeopardize the miner's balance, creating an unstable posture that increases the risk of falling and causing further injury.

3. Avoid lifting from the floor.

As much as possible, have support material stacked and delivered to the work site on pallets to minimize material lifting from the floor level. Often in underground mines, the packaging is unnecessarily destroyed during the transportation process. The lifting equation research suggests that a 30-in starting height is the most efficient for an average height person. As a rule, any material handling task should avoid lifts and placements below the knuckle (measured from a relaxed standing posture) and above the shoulder.

4. Lift in one smooth operation.

A large number of back injuries are attributed to sudden or unexpected movements. The back stress incurred by the worker in this situation is often two to three times as great as when the load is expected [Marras et al. 1987].

5. *Avoid excessive twisting during the lifting process.*

When stacking support materials such as in building a crib, there is a natural tendency to keep the feet still and twist the body to minimize the support construction time (figure 16). However, this practice causes excessive strain on back muscles and vertebrae and should be avoided, particularly when lifting materials in excess of 35-40 lb. Repositioning the body to remain directly in front of the lifting destination will significantly reduce the likelihood of severe back injury that can result from twisting motions.

6. Use mechanical assists whenever possible.

Several examples of support systems that are designed for installation with mechanical assists were previously described. The use of these systems and the development of others should be encouraged. Figure 17 depicts one apparatus used for setting timber posts.

7. Rest when needed.

The basic premise of this recommendation is that muscle fatigue can lead to or increase the likelihood of musculoskeletal injury. As the metabolic demands associated with support construction increase, more frequent rest periods are required. The *Work Practices Guide for Manual Lifting* [NIOSH 1981] recommends that for occasional lifting, the metabolic energy expenditure rate should not exceed 9 kcal/min for physically fit males and 6.5 kcal/min for physically fit females. Energy expenditure in crib building has been measured in one study at 8.48 kcal/min [Gallagher 1987]. Thus, crib building approaches the recommended energy threshold for a physically fit male.

The American Industrial Hygiene Association has developed algorithms to determine the recommended rest interval for various work-related tasks. The rest break will allow the heart rate and breathing rate to return to normal, as well as allowing the metabolic end products of the muscle exertion to dissipate and reoxygenate the muscle. The work rest cycles are based on the energy expenditure required to perform the task. Using the 8.48 kcal/min for conventional wood crib, the recommended rest interval equates to 75% of the work time [Gallagher 1987]. This means that if a crib crew spent 30 min constructing a crib, the recommended rest time is 22 min before building the next crib to prevent excessive muscle fatigue.

8. In low-seam heights, lift from a stooped position versus a kneeling position.

Studies have shown that miners have a significantly lower lifting capacity, 10 lb over on average, in the kneeling posture than in the stooped posture [Gallagher et al. 1990]. In terms of biomechanics, the large muscles of the lower back contract much more vigorously in the kneeling posture than in the stooped posture. This implies a greater compressive loading of the spine when kneeling, thus a greater chance of back injury. However, this recommendation needs to be qualified in the sense that if the load can be placed between the legs when squatting, then the load is closer to the spine, causing less stress on the back.

Figure 17.–Apparatus used for setting timber posts. (Photo courtesy of Strata Products USA).

9. Get help when the required lifting load exceeds the recommended weight limit.

It is important to remember that the recommended weight limit is significantly less than a person's maximum lifting capacity. The intent of the recommended weight limit is to define thresholds that will prevent injury. By asking for help, the probability of back injury can be significantly reduced when the load is shared among two or more people.

10. Promote your own physical fitness.

Support constructing can be a very demanding job. Studies have shown that many back injuries may be prevented by strengthening the lower back and abdominal muscles. Stretching before lifting can also be very important in preventing back strains.

SUPPORT MATERIAL HANDLING PRODUCT GUIDE

Material handling requirements can be divided into two functions: (1) delivery of material into the mine and (2) construction of the roof supports. A summary assessment of various support technologies for these material handling functions is provided in tables 7 and 8.

Support materials are typically loaded onto either supply cars or shield carriers and transported into the mine by a rail haulage system. A typical supply car will be 7 ft wide and 20-24 ft long. The car volume then depends on the bed height above the track rail, which is determined by the seam height. The shield cars are smaller (14-16 ft long), but are 8-12 in lower, which allows more clearance to the mine roof. Thus, a higher stack of material can be transported on them. At the mine entry, the supports are typically unloaded by a diesel- or battery-powered forklift, bucket scoop, or shield hauler. How the supports are packaged is critical to the overall material handling requirements. Generally, the goal is to bundle and unload the materials in full support increments. This will minimize the construction effort by placing the correct amount of material in the vicinity where the support is to be constructed and avoid the extra and generally manual effort in carrying surplus materials to a new location. Table 8 documents the relevant parameters associated with transportation of support material into the mine.

Table 7 shows that prop-type supports (Propsetter, Star Props, Lock-N-Load Props, Alternative Crib Support (ACS), and Yippi Prop) and the pumpable supports in collapsible containers (Pumpable Crib and Tekcrib) provide significant transportation advantages in terms of reduced volumes compared to crib-type supports that are constructed in piecemeal fashion or large-diameter, precast, concrete support structures.

The next issue pertains to the construction of the supports. The primary factor to consider is the labor involved in the support construction and the rate of installation of the support structures. From an injury perspective, the size, weight, handling method, and the number of pieces handled per unit time are relevant factors are previously addressed. The energy required to construct the support would be a useful method to define the risk assessment for musculoskeletal injuries. However, an energy analysis that considers the physiological demand on the human body is quite complex and is beyond the scope of this paper. Instead of this complex analysis, a computation of work (weight of piece times the lifting height) can be done. The efficiency of the support construction can be judged in terms of the installation rate and the construction effort (work). Table 8 compares the relevant parameters for construction of the various support technologies.

Table 8 shows that the construction effect increases dramatically for supports that require piecemeal construction using large numbers of heavy pieces such as conventional concrete cribbing. The work required for constructing a steel-fiber-reinforced donut concrete crib is 40% more than the work required for a conventional four-point wood crib. As a result, the installation rate for these supports are among the lowest of all support technologies. The benefits of the engineered crib supports (Link-N-Lock, Link-N-X, and Tri-Log cribs) are clearly seen in comparing the construction work to that of conventional wood cribbing. The least effort is required for the construction of the Propsetter, ACS, and the Burrell Can supports. Average installation rates for the Propsetter and the ACS are three times greater than rates achieved with conventional wood cribbing. The installation rate with the Burrell Can support depends on the logistics of unloading and installation activities that require machinery to accomplish. Installation rates vary from 30 to more than 50 supports per shift. The Pumpable Crib that is poured in place currently requires manual labor to lift the 55-lb grout bags from a pallet positioned on a forklift near the pump. Heintzmann Corp. plans to develop a batch system for the grout pumping activity, which will all but eliminate the material handling efforts for this support, with the exception of installing the forms to hold the bag during pumping and dismantling them afterward. With a three-person pumping crew underground and four people handling material at the pump station aboveground, 50 Pumpable Cribs per shift have consistently been installed in a test section at a mine site in western Pennsylvania. Even if seven people were used on a crib construction crew, the number of conventional four-point wood cribs constructed per shift would probably be in the range of 35-40.

Table 7.—Transport parameters (normalized to 8-ft mining height)

Support system	Support design	Pieces per support	Transport volume, ft^3/support	No. of supports per supply car
Conventional wood cribs	four-point cribs[1]	32	24.0	16
	nine-point cribs[1]	48	36	11
Conventional concrete cribs	Stopping block cribs (16 by 16 in)	32	16.2	24
	SFR donut cribs	32	29.9	15
	SFR 24 -by 24-in solid crib	72	32.0	12
	SFR four-point crib (24×24 in)	48	21.3	18
Hercules cribs	HM-4	24	21	19
	HM-9 and HM-9 (308)	24	40	10
	HM-16	16	49.8	8
Link-N-Lock cribs	24-in Link-N-Lock	64	18.7	21
	27-in Link-N-Lock	64	21.0	19
	30-in Link-N-Lock	64	23.3	17
	36-in Link-N-Lock	64	28.0	14
	42-in Link-N-Lock	64	32.7	12
	48-in Link-N-Lock	64	37.3	11
	60-in Link-N-Lock	64	46.7	8
Link-N-X cribs	24 in (standard design)	32	9.3	42
	27 in (standard design)	32	10.5	37
	24 in (high-capacity design)	48	16.0	25
	30 in (high-capacity design)	48	20.0	20
	36 in (high-capacity design)	48	24.0	16
Lock-N-Load props	Standard design	4	3.2	124
Propsetter	8.5-in diameter	3	4.6	84
	10.0-in diameter	3	6.0	65
Star Props	100 ton (12-in diameter)	3	9.2	43
	60 ton (10-in diameter)	3	6.7	58
Tri-Log cribs	30 in (standard design)	48	19.8	20
	36 in (standard design)	48	23.7	17
	30 in (high-capacity)	48	29.7	13
	36 in (high-capacity)	48	35.6	11
	48 in (high-capacity)	48	47.5	8
Burrell Can	18-in diameter	4	18.0	22
	24-in diameter	4	31.6	12
	30-in diameter	4	49.1	8
	36-in diameter	4	70.4	6
Confined core crib (3-C)	36-in diameter	4	70.4	6
ACS or 55-ton prop	Flat plate (8 in)	1	2.8	139
	Timbers as header	4	4.3	91
	Pizza head plate	2	4.1	95
Pumpable Crib[2]	30-in diameter	1	3.9	100
Tekcrib	42-in diameter	1	7.7	51
Tekprop	18-in diameter	1	12.8	31
Stretch Prop	6-ft collapsed length	3	3.7	107
Yippi Prop	Standard design	1	2.8	140

[1]Based on 6- by 6- by 36-in oak timbers.
[2]Material assessment considers only that used underground, i.e., the bags for forming the support. It does not include the hardware required to support the bags for filling since this hardware is used over and over again. Since the supports were pumped from the surface, the grout material is not included in this assessment.

Table 8.—Support construction parameters (normalized to 8-ft construction height)

Support system	Support design	Weight per piece, lb	Total support weight, lb	Construction work, ft-lb	Installation rate,[2] supports per shift
Conventional wood cribs	four-point cribs[1]	40	1,280	5,760	15
	nine-point cribs[1]	40	1,920	8,640	12
Conventional concrete cribs	Stopping block cribs (16 by 16 in)	52	1,664	8,055	19
	SFR donut cribs	56	1,792	8,064	19
	SFR 24- by 24-in solid crib	53	3,816	17,172	9
	SFR four-point crib (24×24 in)	53	2,544	11,448	13
Hercules cribs	HM-4	47	752	3,008	32
	HM-9 (308)	44	1,056	4,752	26
	HM-16	41	1,968	8,856	16
Link-N-Lock cribs	24-in Link-N-Lock	13	813	3,658	24
	27-in Link-N-Lock	15	941	4,234	24
	30-in Link-N-Lock	17	1,069	4,810	19
	36-in Link-N-Lock	21	1,318	5,933	16
	42-in Link-N-Lock	25	1,574	7,085	16
	48-in Link-N-Lock	29	1,824	8,208	15
	60-in Link-N-Lock	37	2,330	10,483	14
Link-N-X cribs	24 in (standard design	14	343	1,201	64
	27 in (standard design)	16	518	2,333	45
	24 in (high-capacity design)	13	638	2,554	32
	30 in (high-capacity design)	18	854	3,417	30
	36 in (high-capacity design)	22	1,152	4,282	21
Lock-N-Load props	Standard design	44	88	513	96
Propsetter	8.5-in diameter	132	168	820	48
	10.0-in diameter	184	216	1,049	40
Star Props	100 ton (12-in diameter)	247	301	1,668	40
	60 ton (10-in diameter)	172	210	1,220	40
Tri-Log cribs	30 in (standard design)	19	916	3,667	26
	36 in (standard design)	24	1,133	4,531	21
	30 in (high-capacity)	18	846	3,806	17
	36 in (high-capacity)	22	1,593	7,167	14
	48 in (high-capacity)	31	2,241	10,083	13
Burrell Can	18-in diameter	N/A	162	1,134	40
	24-in diameter	N/A	270	1,890	40
	30-in diameter	N/A	405	2,835	40
	36-in diameter	N/A	567	3,969	40
Confined core crib (3-C)	36-in diameter	N/A	567	3,969	40
ACS or 55-ton prop	Flat plate (8 in)	151	151	680	64
	Timbers as header	151	232	1,247	53
	Pizza head plate	140	178	896	53
Pumpable Crib	30-in diameter	N/A	200	900	50
Tekcrib	42-in diameter	N/A	160	720	60
Tekprop	18-in diameter	N/A	80	360	59
Stretch Prop	6-ft collapsed length	50	77	231	69
Yippi Prop	Standard design	92	92	414	80

[1]Based on 6- by 6- by 36-in oak timbers.

[2]The installation rates may vary considerably due to the labor and equipment used in support construction and delivery of support material to the working area. The numbers shown are representative installation rates. The installation rates are not normalized to man-hours or effort. The support construction crew is generally two or three people, although the PumpableCrib installation currently uses as many as seven people (all of whom work for the support manufacturer).

CONCLUSIONS

Material handling is an important aspect of secondary roof support construction and more attention should be paid to it in the support design and selection process. With more than 40,000 lost workdays attributed to timber-handling injuries in the past 9 years, the construction of conventional wood cribs and timber supports is the primary cause of injury to these underground mine workers. Included in this paper is a detailed summary of the various roof support systems and a description of relevant material handling parameters to facilitate consideration of material handling factors in the selection of a standing roof support system.

In recent years, several alternative support technologies have been developed, which in addition to providing superior roof control, offer material handling advantages. Surveillance data show that the increase in use of these alternative support technologies is consistent with the decreasing trend of material handling injuries due to roof support construction in recent years. The severity of these injuries has decreased by nearly 50% during 1995-98, which is when the use of alternative support technologies attained proportions where they exceeded the number of conventional cribs. Thus, these new support technologies are having a positive impact on reducing material handling injuries to coal miners.

Using the NIOSH lifting equation, which defines lifting thresholds for various conditions and lifting scenarios, it is seen that the weight of conventional crib timbers exceeds the recommended lifting threshold. Conversely, the weights of the engineered timber products is within the recommended lifting thresholds and provides further confirmation that these lightweight materials are reducing material handling injuries. Systems that are installed with mechanical aid, such as the Burrell Can Support (Burrell Mining Products) and the Star Prop Propsetter (Strata Products USA), or pumpable support systems that are installed in place in the mine, such as the Pumpable Crib (Heintzmann Corp.), can substantially reduce the effort required to install supports and thus dramatically reduce the risk of injury to the mine worker. Therefore, depending on the other parameters being held fairly constant, these alternative supports should reduce the risk of musculoskeletal injuries due roof support construction in underground mines.

When material handling is required, following some basic lifting practices can make a difference in preventing injury. Most importantly, the material should be held as close to the body as possible to reduce the stress on the back, and twisting of the body during the lifting process should be minimized and avoided, if possible. Extra care must be exercised in the restricted environment of an underground mine to avoid injury. Each miner is different, but everyone has a comfort zone in being able to lift materials of a certain weight for a given amount of time. The probability of injury increases when muscle fatigue occurs, so proper rest periods to avoid over exertion can mean a lot in preventing injuries.

ACKNOWLEDGMENTS

The author acknowledges the efforts of Deno M. Pappas, civil engineer, NIOSH Pittsburgh Research Laboratory (PRL), who collected pertinent injury data pertaining to the construction of roof support systems in underground mines. These data provide valuable insight into the type and magnitude of injuries involved in support construction and were useful in comparing injury trends for alternative support technologies that have material handling advantages over conventional wood and concrete cribs. The author also acknowledges Kim M. Cornelius, industrial engineer; Lisa J. Steiner, safety engineer; and Sean Gallagher, research physiologist, also with NIOSH PRL, for their contributions in the application of the NIOSH Revised Lifting Equation and for their research accomplishments pertaining to the biomechanics of lifting heavy materials. Much of their work is being adopted for the roof support construction application, which is addressed in this paper.

REFERENCES

Gallagher S [1987]. Back strength and lifting capacity of underground miners. In: Proceedings of the Human Engineering and Human Resources Management in Mining, Bureau of Mines Technology Transfer Seminars, Pittsburgh, PA, July 7-8; St. Louis, MO. July 15-16; and San Francisco, CA, July 21-22. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, IC 9145, pp. 21-32.

Gallagher S, Bobick TG, Unger RL [1990]. Reducing back injuries in low coal mines: redesign of materials-handling tasks. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, RI 9614.

Gracovetsky S, Farfan HF [1986]. The optimum spine. Spine *11*(6):543-573.

Marras WS, Rangarajulu SL, Lavender SA [1987]. Trunk loading and expectation. Ergonomics *30*(3):549-560.

NIOSH [1981]. Work practices guide for manual lifting. Cincinnati, OH: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 81-122.

NIOSH [1997]. Musculoskeletal disorders and workplace factors. A critical review of epidemiologic evidence for work-related musculoskeletal disorders of the neck, upper extremity, and low back. Bernard BS, ed. U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 97-141.

Waters TR, Putz-Anderson V, Garg A, Fine LJ [1993]. Revised NIOSH equation for the design and evaluation of manual lifting tasks. Ergonomics *36*(7):749-776.

Waters TR, Putz-Anderson V, Garg A [1994]. Applications manual for the revised NIOSH lifting equation. U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 94-110.

Winn FJ, Biersner T [1996]. Exposure probabilities to ergonomic hazards among miners. Int J Ind Ergonomics *18*:417-422.

NIOSH SAFETY PERFORMANCE TESTING PROTOCOLS FOR STANDING ROOF SUPPORTS AND LONGWALL SHIELDS

By Thomas M. Barczak[1]

ABSTRACT

The safety of mine workers depends on the proper installation of roof supports to prevent the ground from collapsing into the working areas of an underground mine. As new support systems are developed, they need to be properly evaluated to make sure that they are capable of providing adequate roof support before they are first used in a mine. In addition to making certain that the supports meet basic safety criteria, the limitations of the support need to be fully defined in order to avoid improper application of a particular support design. The National Institute for Occupational Safety and Health (NIOSH) operates a world-class facility called the Safety Structures Testing Laboratory. This laboratory contains a unique load frame, the Mine Roof Simulator, which is capable of simulating the ground behavior in underground mines for conducting full-scale evaluations of roof support systems. Safety performance testing protocols using the unique Mine Roof Simulator have been developed for both standing roof support systems and longwall shield supports. The purpose of this paper is to describe these test procedures. The protocol for standing roof supports incorporates seven test series: (1) uniform loading baseline tests, (2) height evaluations, (3) asymmetric loading, (4) biaxial loading, (5) load rate studies, (6) active loading determination, and (7) static loading evaluations. For longwall shields, a four-series test program that accurately simulates in-service conditions on a longwall face is proposed. This test program consists of (1) transfer of horizontal load to the caving shield-lemniscate assembly (zero-friction test), (2) point loading of shield joints due to lateral movement or rotation of the canopy, (3) evaluation of leg socket and leg cylinder integrity, and (4) face-to-waste racking of the shield. In addition, an evaluation of the shield's hydraulic components will be conducted prior to the performance testing. These protocols will provide state-of-the-art safety performance evaluations of emerging support technologies, as well as a means to assess the safety of both new and aging longwall shields. Hence, this effort will enhance the safety of mine workers by ensuring that critical support elements are properly designed and that aging supports are retired before their support capability is jeopardized.

[1]Research physicist, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

Ground control is one of the most fundamental aspects of underground mining. Roof support systems are needed to stabilize exposed mine openings and prevent collapse of the mine roof. Without these critical support structures, the safety of the miners would continuously be in jeopardy. Therefore, it is essential that roof support systems be properly designed so that they can provide adequate ground control in all circumstances.

The National Institute for Occupational Safety and Health (NIOSH) is available to conduct safety performance testing of emerging roof support technologies as they are developed to assist manufacturers in meeting basic safety standards before the supports are ever used in an underground coal mine. These tests are also designed to ensure that the support is a viable roof support system. Hence, in addition to evaluating basic safety criteria, the safety performance tests are designed to determine the limitations of the support system by evaluating the support performance to failure under various loading conditions, so that performance characteristics can be matched to ground behavior in a particular mine in which the support system is installed. These tests are conducted at the Safety Structures Testing Laboratory in the unique Mine Roof Simulator load frame and simulate actual in-service conditions in a mine. The tests are conducted through cooperative agreements established with the various support manufacturers.

In the past 7 years, over 1,000 tests have been conducted on various secondary roof support systems. As a result of this effort,18 new support systems have been successfully adopted by the mining industry, making a significant impact on longwall tailgate support as alternatives to conventional wood and concrete cribbing. These include the following supports developed by Strata Products USA: Hercules crib, Link-N-Lock crib, Link-N-X crib, Propsetter support, Power Wedge, Rock Prop, and Star Prop. Heintzmann Corp. developed the Alternative Crib Support (ACS), the 55-Ton Prop, Quick Timber, and the Pumpable Crib. Burrell Mining Products conducted tests on The Can support. Fosroc Corp. developed the Tekcrib and Tekprop supports. American Commercial, Inc., developed the Tri-Log crib. Ferrocraft, Inc., developed the Stretch Prop and other yieldable timber posts systems. Safety performance tests were conducted on the YIPPI Prop (Western Support Systems) and the Coal Post (Dywidag Systems International, Inc.).

In addition to the development of innovative alternatives to conventional wood and concrete cribbing, safety performance testing protocols have been developed for longwall shield supports. The unique loading capabilities provided by the Mine Roof Simulator are especially suited to testing shield supports. The caving mechanics of strata in longwall mining, and in-service loading conditions can be simulated much more realistically than is possible in a static load frame. Cyclic testing procedures have also been developed to evaluate the remaining life of aging longwall shields.

The purpose of this paper is to describe the support testing protocols developed for the unique loading capabilities of the Mine Roof Simulator load frame, protocols that will improve the safety of mine workers by helping design support systems properly and by evaluating aging supports so they are not used beyond their useful life span.

NIOSH SAFETY STRUCTURES TESTING LABORATORY

The Mine Roof Simulator is a servo-controlled hydraulic press custom built by MTS Systems Corp. to U.S. Bureau of Mines (USBM) specifications. It was designed specifically to test longwall shields, and is the only active load frame in the United States that can accommodate full-size shields.

A functional diagram of the load frame is shown in figure 1. The load frame has several distinctive characteristics. The size of the platens are 20 ft x 20 ft. The upper platen can be moved up or down and hydraulically clamped into a fixed position on the directional columns to establish a height for tests. With a maximum vertical opening between the upper and lower platen of 16 ft, the load frame can accommodate the largest shields currently in use. Load application is provided by controlled movement of the lower platen. The load frame is a biaxial frame, capable of applying both vertical and horizontal loads. Load actuators are equipped with special hydrostatic slip bearings to permit simultaneous load and travel. This allows vertical and horizontal loads to be applied simultaneously. The capability to provide controlled loading simultaneously in two orthogonal directions is unique at this scale.

Vertical load is applied by a set of four actuators, one on each corner of the lower platen. Loads of up to 3 million pounds can be applied in the vertical direction by upward movement of the lower platen. Each actuator is capable of applying the full 3 million pounds of force, so that the specimen can be placed anywhere on the platen surface and the full 3 million capacity can be provided. The vertical (upward) range of motion of the lower platen is 24 in.

Horizontal loading is applied by four actuators, with two actuators located on both the left and right side of the load frame just below floor level. These actuators act in pairs to provide horizontal displacement of the lower platen in either a positive or negative (x) direction. The horizontal range of motion of the lower platen is 16 in.

There is no programmable control of the lower platen in the lateral horizontal axis (y-direction). The load frame has a reactive capacity of 1.6 million pounds in this direction, but loads can not be applied laterally. The range of motion of the lower platen in this direction is ± 0.5 in.

The lower platen is controlled within six degrees of freedom through the unstressed reference frame. This frame provides feedback on platen displacements and rotations to the closed-loop control system. Pitch, yaw, and roll of the lower platen are controlled to keep the lower and upper platens parallel during load application.

A shock absorber actuator is positioned on the left and right sides of the lower platen. These shock absorbers will control displacement of the lower platen to less than 0.1 in in the event of a sudden failure of the support specimen. The shock absorber action absorbs energy stored in the load frame so that it is not unintentionally released to the test specimen.

Two hydraulic pumps provide up to 3,000 psi of pressure to the vertical and horizontal actuators during load application. The rate of movement of the lower platen is limited by the 140-gal/min capacity of the hydraulic pumps. The maximum platen velocity is 5 in/min, assuming simultaneous vertical and horizontal displacement.

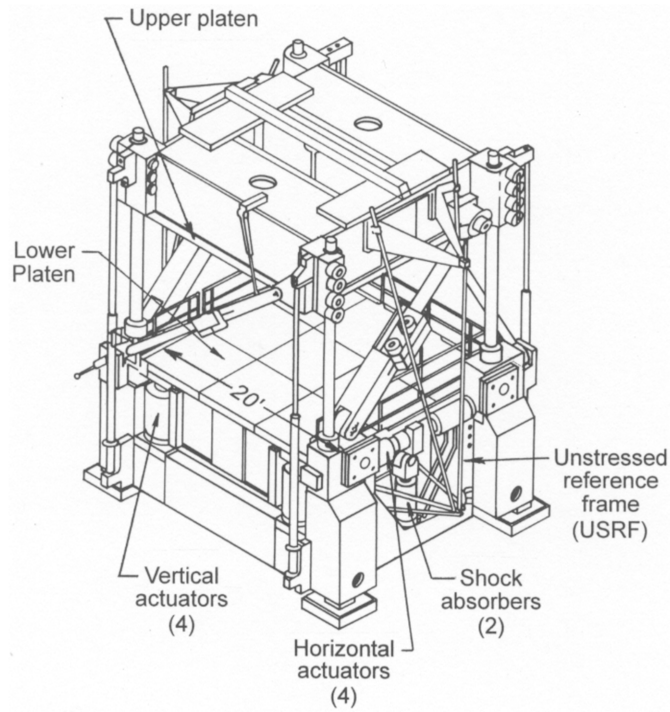


Figure 1.–Functional diagram of Mine Roof Simulator.

STANDING ROOF SUPPORT TESTING PROTOCOL

Standing roof supports are structures that are placed in a mine entry between the roof and the floor. Their performance can be described relative to three primary design factors: (1) strength, (2) stiffness, and (3) stability.

Strength – The strength of a roof support generally refers to its ultimate load capacity. Hence, all supports are tested to failure to determine the strength of the support.

Stiffness – Stiffness is a measure of how quickly a support develops its load-carrying capacity and determined by measuring support load capacity as a function of applied convergence.

Stability – Stability is a measure of how long a support can sustain its load-carrying capacity. The stability of a support structure is affected by several parameters. These include (1) aspect ratio of the support, (2) boundary conditions established with the load frame at the roof and floor contact, (3) direction of load application, (4) quality and properties of the specimen, and (5) rate of loading.

STANDARDIZED TESTS FOR STANDING SUPPORTS

TEST SERIES I – UNIFORM LOADING BASELINE TESTS

Objective – Establish baseline performance of a support under ideal loading conditions.

Test Requirement – Simulate roof-to-floor convergence by applying uniform loading to the support element. The response of the support structure is measured relative to its stiffness, strength, and stability.

Test Procedure – A representative support is placed in the Mine Roof Simulator with full roof and floor contact to establish uniform loading on the support. A controlled vertical displacement at a rate of 0.5 in/min is applied to the support system by the load frame to simulate convergence of the mine roof and floor. The applied load is measured as a function of vertical displacement to determine the stiffness of the support. Convergence continues until the support (1) becomes unstable, (2) sheds load to the point where the support provided is inadequate, or (3) until the full 24-in stroke of the load frame is reached. Ultimate strength and complete performance profiles are determined by plotting support load versus applied displacement.

TEST SERIES II - IMPACT OF SUPPORT SIZE ON STABILITY AND CAPACITY

Objective – Determine the impact of the size of the support on its capacity and define proper support sizes that will ensure stability through a useful convergence.

Test Requirements – Vary support sizes and provide uniform loading through controlled roof-to-floor convergence. The capacity of the support as a function of the support area will be determined from this suite of tests. The stability of some support systems is largely governed by the aspect ratio or the height-to-width ratio of the support. When this is a design parameter, the support will be evaluated at several heights representing various aspect ratios to determine the limits of the support is stability. Standard heights are 4, 6, 8, 10, and 12 ft. Typically, the support is widened to maintain stability at higher operating heights. The goal of the test is to determine an acceptable aspect ratio range over which the support will maintain stability at all recommended operating heights. For example, tests on conventional wood crib supports have determined that the aspect ratio should be maintained between 2.5 and 5.0, with 4.3 considered an optimum for uniform load conditions.

Test Procedure – The test procedure is basically the same as in the first test series. A representative support is placed in the Mine Roof Simulator with full roof and floor contact to establish uniform load on the support. A controlled vertical displacement is applied to the support system to simulate convergence. Convergence continues until the support (1) becomes unstable, (2) sheds load to the point where the support provided is inadequate, or (3) until the full 24-in stroke of the load frame is reached. The ultimate strength and capability of the support needed to sustain load resistance while yielding will be determined by analysis of the load-displacement profile.

TEST SERIES III - ASYMMETRIC LOADING

Objective – Determine the impact of asymmetric loading on the stability and overall support capability of the support.

Test Requirements – Simulate asymmetric loading conditions that occur with uneven roof and floor contact or because of wedging the support in place. Figure 2 illustrates four asymmetric loading configurations that can be applied to standing roof supports. Figure 3 shows some examples of actual supports being subjected to these asymmetric loading conditions.

Another condition that creates asymmetric loading is floor heave. Floor heave is simulated by creating a foundation that rotates as support load is developed. This is accomplished by

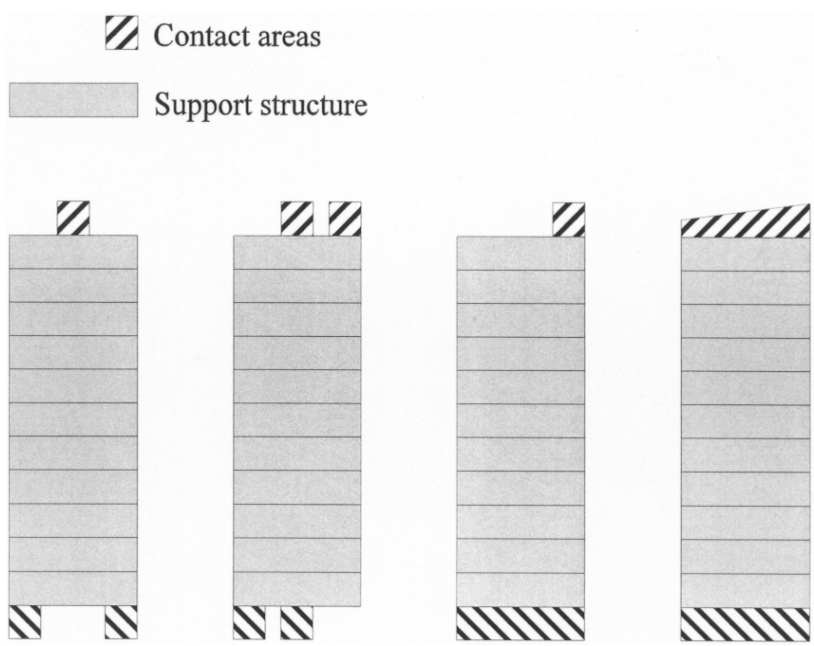


Figure 2.–Asymmetric loading configurations.

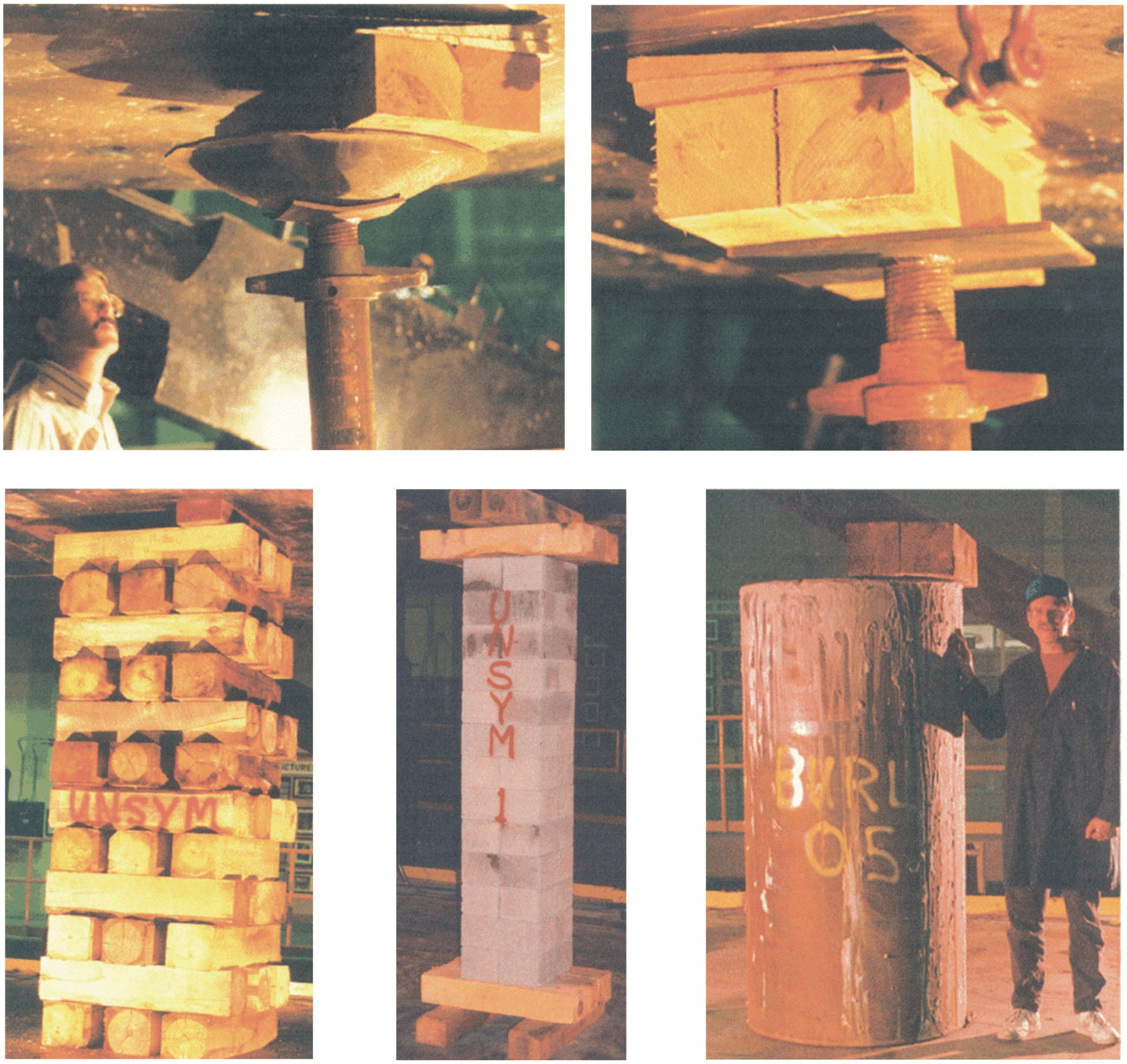


Figure 3.–Examples of asymmetric loading conditions.

placing the roof support structure on a rigid steel plate that is supported on one side by a soft (crushable) support and a stiff (rigid) support on the opposite side. Figure 4 illustrates this arrangement and an example of a test conducted in the Mine Roof Simulator.

Test Procedure – A support is placed in the load frame. A specific roof and floor contact is established in accordance with the diagrams shown above by strategically placing contact blocks at the roof and floor interface. A controlled vertical displacement is applied to the support. Convergence continues until the support (1) becomes unstable, (2) sheds load to the point where inadequate support is provided, or (3) until the full 24-in stroke of the load frame is reached. The ultimate strength and capability of the support to sustain load resistance while yielding will be determined by analysis of the load-displacement profile. Upon completion of this test, another support is installed and another contact configuration is established to evaluate a different asymmetric loading condition. The test procedure is then repeated for this and any other asymmetric loading configuration.

TEST SERIES IV - BIAXIAL LOADING

Objective – Determine the impact of horizontal loading on support capability.

Test Requirements – Simulate both vertical (roof-to-floor) convergence as well as lateral movements associated with bending or buckling of laminated roof or floor structures as a result of horizontal stress. This is accomplished by moving the floor of the load frame simultaneously in both vertical and horizontal directions creating a load vector in which the base of the support is moved laterally with respect to the top of the roof support at the same time the support is being squeezed by roof-to-floor convergence (see figure 5).

Figure 4 - Floor heave simulation and examples of support testing.

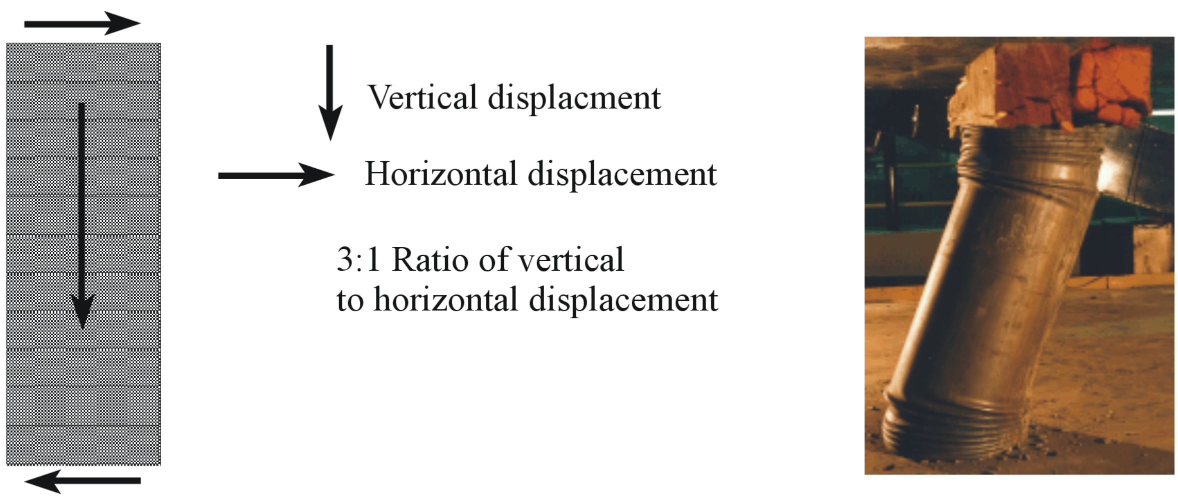


Figure 5.–Biaxial load conditions and example of support being subjected to biaxial loading.

Test Procedure – A support is placed in the load frame. Typically, full roof and floor contact is utilized for this test series. The Mine Roof Simulator is commanded to apply a ratio of vertical to horizontal displacement. The standard ratio (vertical to horizontal displacement) is 3:1, although the ratio can be varied if desired. The applied biaxial convergence continues until the support (1) becomes unstable, (2) sheds load to the point where inadequate support is provided, or (3) until the full 24-in vertical stroke of the load frame is reached. The ultimate strength and capability of the support to sustain load resistance while yielding will be determined by analysis of the load-displacement profile. If the support stability is sensitive to changes in the aspect ratio as determined in test series II, then the support height will also be varied.

TEST SERIES V - LOAD RATE STUDIES

Objective – Some supports have a tendency to provide greater load resistance as the loading rate is increased. The objective of this test series is to determine the impact of loading rate on the support's behavior.

Test Requirements – Vary loading rate by controlling the applied roof-to-floor convergence. The Mine Roof Simulator can control the rate of roof-to-floor from 0.1 to 5.0 in/min.

Test Procedure – Baseline test data were established in test series I at the standard loading rate of 0.5 in/min. To establish a load rate profile, supports are tested to failure at least two additional rates. Typically, rates of 0.1 and 5.0 in/min are utilized. Support load as a function of convergence is then compared for the different loading rates, and the impact of loading rate on the stability of the support and the nature of the failure are documented. Figure 6 illustrates a concrete crib exploding during a high rate of loading.

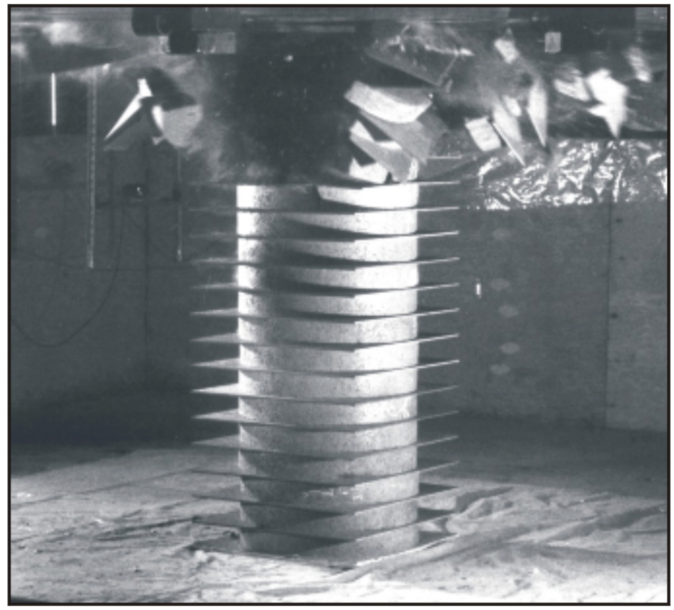


Figure 6.–Violent failure of concrete crib at 5 in/min applied convergence.

TEST SERIES VI -ACTIVE LOADING DETERMINATION

Objective – Some roof supports are capable of providing an active roof load during installation of the support. The objective of this test series is to determine the active loading capability of those supports.

Test Requirements – Measure the active roof loading generated by a support during its installation.

Test Procedure – A load cell is placed on top of the support to obtain a more accurate measure of applied roof loading, particularly when the measured active roof loads are expected to be less than 20 kips. The load frame platens remain stationary during the test. An effort is also made to determine whether active loading remains constant or is shed over time once the support is installed. Hence, a plot of active roof loading as a function of time is made, and a decay rate is determined. The amount of time can depend on type of support, but the initial period is 30 min.

TEST SERIES VII - STATIC LOADING EVALUATIONS

Objective – Static loads are used to assess creep and relaxation in support material construction. The objective of this test series is to determine the creep and relaxation properties of a support.

Test Requirements – Creep is the continuation of deformation after a static load has been applied. To measure creep, a constant force must be applied to the support. Relaxation is the opposite of creep. Relaxation is the reduction in stress or load after an applied displacement. Hence, the test requirement to measure creep is maintaining constant displacement.

Test Procedure – For the creep study, a support is placed in the load frame. The load frame is operated in force-control, and a load is applied and held constant for an extended period. The change in displacement is then measured to determine the rate of creep.

For the relaxation study, the load frame is operated in displacement control, and the support is loaded through a designated convergence, that is held constant by the load frame. The change in support load is then measured as a function of time to determine the relaxation properties of the support.

NIOSH SAFETY PERFORMANCE TESTING PROTOCOL FOR LONGWALL SHIELDS

NIOSH also conducted shield performance tests in the Safety Structures Testing facility (figure 7). The Mine Roof Simulator can simulate in-service loading conditions on shields more accurately than static load frames. It is the only active load frame in the United States with sufficient size and load capacity to accommodate shield testing and allows realistic and cost-effective shield evaluations by combining both vertical and horizontal (racking) loads into a single load cycle.

The ultimate goal of the NIOSH shield testing program is to ensure the safety of mine workers by ensuring that new shields are adequately designed and that aging or damaged shields retain adequate structural integrity for continued use. This section describes the shield testing protocols, developed through extensive studies of shield mechanics and performance tests.

SIMULATION OF IN-MINE SERVICE CONDITIONS

There are two basic aspects to shield loading. The initial load condition is determined by actively setting the shield against the mine roof and floor. Subsequent loading is produced by the movement of the surrounding strata during the caving process and the associated internal forces developed within the support structure.

Figure 7.–NIOSH Mine Roof Simulator.

As the shield is set against the mine roof and floor, there is a tendency for the canopy to be displaced horizontally relative to the base (figure 8). This is due to the resultant horizontal component of the leg forces, which causes either slippage of the canopy along the roof interface or displacement (compaction) of fractured strata or debris immediately above or below the shield. The Mine Roof Simulator accurately simulates this behavior by allowing the floor of the load frame to move horizontally and transfer horizontal load from the horizontal component of the leg forces to the caving shield-lemniscate assembly. When a shield is tested against a rigid frame, the canopy and base are restrained from moving horizontally. This restraint eliminates load development in the caving shield-lemniscate assembly and therefore does not properly simulate in-mine service conditions.

Once the shield is set against the mine roof and floor, load development within the shield is controlled by—

(1) Contact configuration established with the mine roof and floor,

(2) Vertical displacement of the canopy relative to the base induced by deflection of the main roof beam and the weight of the fractured immediate roof strata being supported by the shield (figure 9),

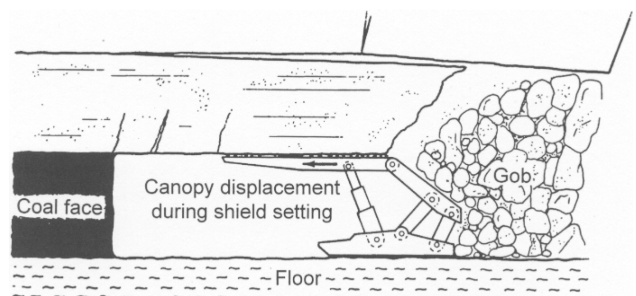


Figure 8.–Horizontal movement of canopy toward longwall face as the shield is set against the mine roof.

(3) Face-to-waste movement of the immediate roof as the strata break into disjointed blocks because of face abutment loading and loss of confinement (figure 10), and

(4) Waste-to-face loading induced by gob material acting on the caving shield and/or the internal forces developed within the shield resulting from leg forces and component reactions (figure 11) and lateral loading due to skewing of the canopy caused by setting against adjacent shields, inclination of the face, or rotation of the canopy due to uneven roof and/or floor conditions (figure 12).

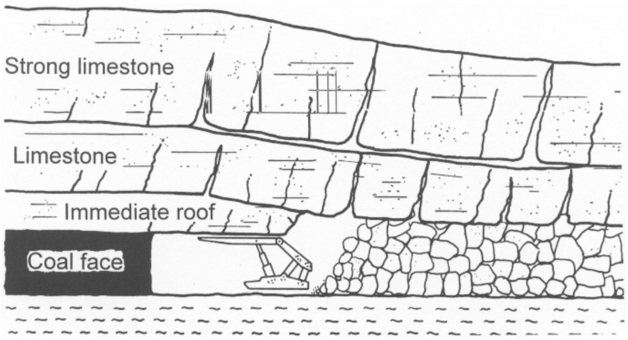


Figure 9.–Vertical shield loading induced by deflection of main roof and weight of damaged immediate roof.

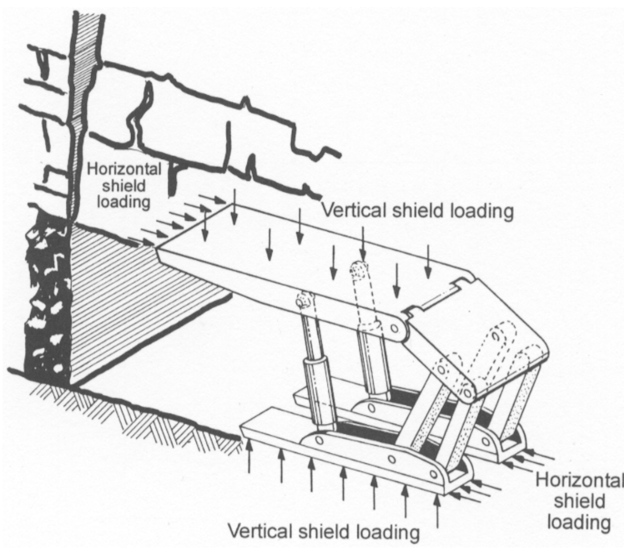


Figure 10.–Horizontal shield loading induced by face-to-waste movement of immediate roof.

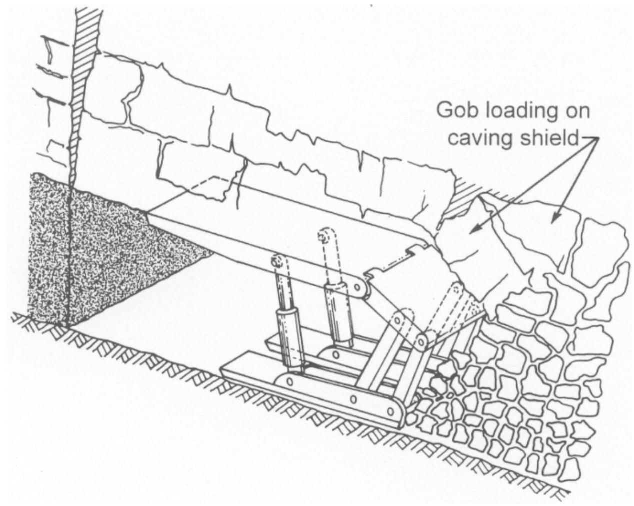


Figure 11.–Horizontal loading toward coal face induced by gob loading on caving shield.

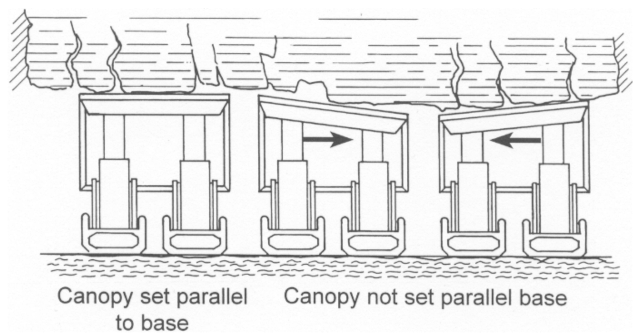


Figure 12.–Lateral loading induced by uneven roof contact.

NIOSH STANDARDIZED SHIELD TEST PROCEDURES

Standard tests consist of an evaluation of hydraulic components under static loading conditions followed by a series of cyclic tests to evaluate the structural integrity of the shield. For each test, the shield is positioned in the load frame in an orientation consistent with the objectives of the test. Prior to cyclic loading, the shield is actively set against the load frame platens by pressurization of the leg cylinders to some nominal load, typically 50 to 75 bar. Cyclic loading is provided by controlled displacement of the Mine Roof Simulator load frame floor against a stationary roof. Each load cycle consists of ramping the load, a hold, an unloading ramp, and a hold. A combination of vertical and horizontal displacements are applied, often simultaneously, to produce the required load conditions. The loading rate is dependent on the shield stiffness and capabilities of the load frame. Two load cycles per minute are a typical loading rate. The loading profile in each test series is designed to maximize total shield loading. Hence, load profiles are typically chosen that provide load equal to the yield load rating for the shield. Since the Mine Roof Simulator is an active load frame, there is no need to exceed the rated capacity of the shield to account for friction effects within the support structure. A minimum of 5,000 cycles for each test series is recommended, but this number may be varied depending on the customer's needs.

HYDRAULIC COMPONENT EVALUATIONS

A series of tests are conducted to determine the performance and condition of the leg cylinders and shield hydraulics.

Yield setting – The shield is loaded until each yield valve on all leg cylinders opens. The recorded pressure and maximum shield rating for the designated operating height are determined.

Leakage Test –The leg cylinders are pressurized to some nominal load and held for 30 min. During the hold, the pressures are monitored. Leakages are evaluated to determine the source of the leakage.

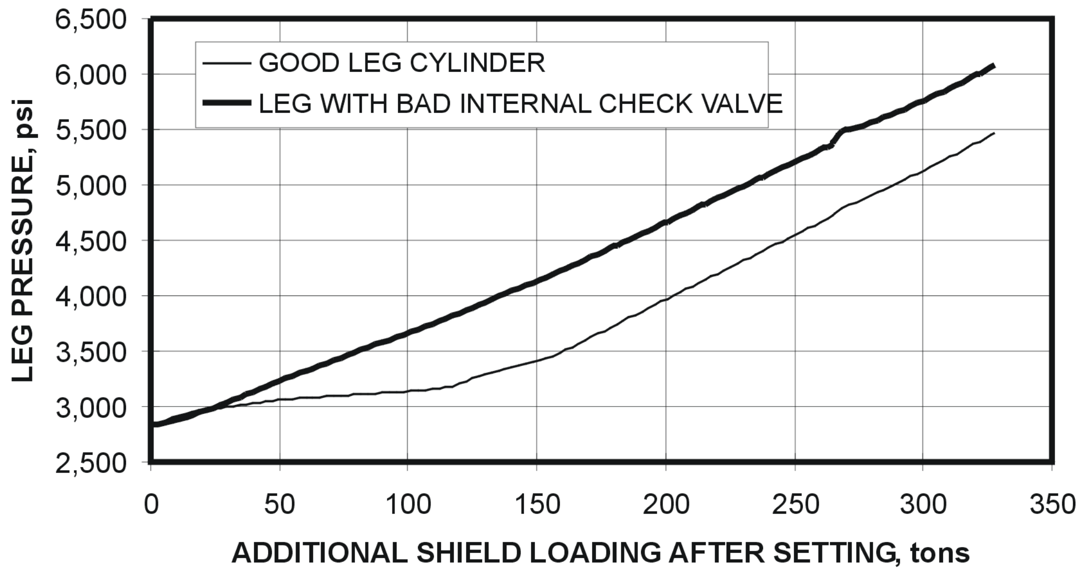


Figure 13.–Test to determine defective staging valve.

Staging Valve Test – The bottom stage on all leg cylinders is fully extended, and the shield is actively set against the load frame platens. Additional load is then applied to the shield while hydraulic pressure is monitored. If the staging valve is working properly, the pressure in the bottom stage should not increase until the force in the upper stage equals the setting force developed in the bottom stage (figure 13).

TEST SERIES I – TRANSFER OF HORIZONTAL LOAD TO THE CAVING SHIELD-LEMNISCATE ASSEMBLY

Objective – Minimize external horizontal load acting on the shield to ensure that horizontal components of the leg forces are transferred to the lemniscate links, thereby maximizing load development in the caving shield-lemniscate assembly.

Test Requirements – The canopy must be free to displace horizontally with respect to the base to allow the caving shield-lemniscate assembly to participate to a degree consistent with underground shield behavior. This is accomplished by commanding the floor of the load frame to move horizontally with respect to the roof in a direction and magnitude consistent with the resultant leg force. Main roof loading and deflection of the immediate roof beam are simulated by controlled vertical displacements. The applied displacement and associated shield response are shown in figures 14 and 15.

Canopy and Base Contacts – A four-point contact on the corners of the canopy is used to maximize bending produced by the increase in leg pressure. A three-point canopy contact where one of the rear contacts is removed can also be used to further intensify stress development in the canopy. Base contacts are located at the ends of each base section to maximize bending in the base.

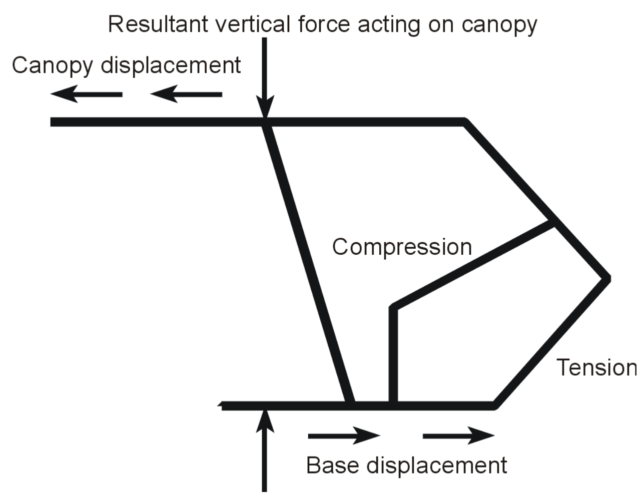


Figure 14.–Applied loading for test series I.

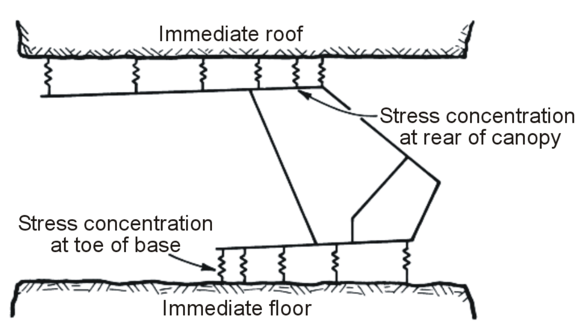


Figure 15.–Shield response to test series I loading.

Test Procedure

1. The shield is set against the load frame roof and floor at 50- to 75-bar leg pressure using an external hydraulic power supply.

2. The floor of the load frame is moved horizontally in a direction that eliminates the horizontal load applied by the load frame during the setting operation, which causes this horizontal load to be transferred to the shield components. The elimination of the external horizontal restraint moves the resultant force acting on the base forward, which intensifies toe loading, a critical load condition for two-leg shields.

3. Cyclic loading is initiated by a controlled vertical and horizontal movement of the lower platen of the load frame, inducing a combined vertical and waste-to-face displacement of the canopy relative to the base. The horizontal platen movement is calibrated to minimize horizontal load restraint provided by the load frame throughout the loading cycle. For two-leg shields, this requires the canopy to be displaced in a faceward direction at a rate that is proportional to the increasing horizontal component of the leg force developed from vertical closure. The result of these actions is that the caving shield-lemniscate assembly is fully loaded to provide internal equilibrium within the shield.

TEST SERIES II – POINT LOADING OF SHIELD JOINTS DUE TO LATERAL MOVEMENT OR ROTATION OF CANOPY

Objective – To maximize loading in the various shield joints and component clevises by causing point load conditions due to tilting of the lemniscate links caused by lateral movement of the canopy relative to the base.

Test Requirements – Joint wear is the most common problem causing premature shield retirement. The requirement for this test is to induce a resultant load vector that skews the canopy laterally with respect to the base (see figure 16). The shield joints have a single degree of rotation, much like a person's knee functions. Stress on the connecting pins is intensified when the canopy is skewed laterally, causing partial contact of the connecting pins within the clevis (figure 17).

Canopy and Base Contacts – A three-point canopy contact with one contact omitted from the rear canopy corner is used to further maximize twisting of the canopy and connecting joints (figure 18A). The outside base, away from the direction of the tilt is supported at the ends, while the inside base in the direction of the tilt has full contact (figure 18B).

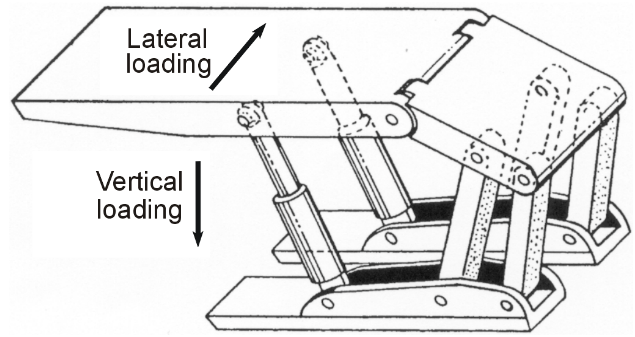


Figure 16.–Lateral loading caused tilting of leg cylinders and lemniscate links during test series II.

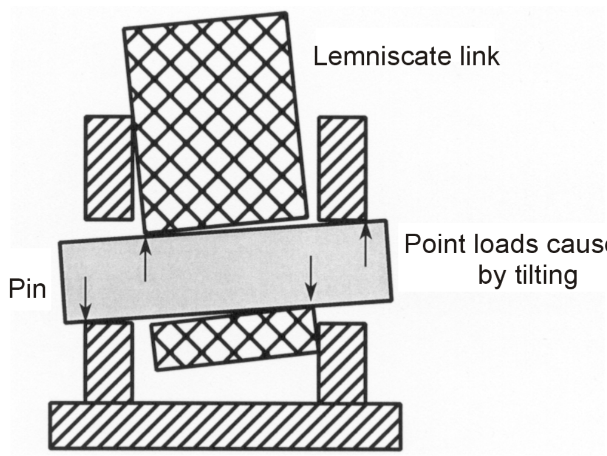


Figure 17.–Illustration of point loading in lemniscate link joints due to tilting of lemniscate link in test series II.

Test Procedure

1. The shield is positioned in the load frame so that the direction of applied horizontal displacement (loading) is across the canopy, as shown in figure 16.

2. The shield is set against the load frame roof and floor at 50 to 75 bar of leg pressure.

3. The canopy is displaced laterally with respect to the base, causing the leg cylinders and lemniscate links to tilt toward the direction of applied lateral loading.

4. Cyclic loading is initiated by the active load frame applying vertical displacement while maintaining the lateral displacement of the canopy with respect to the base. If necessary, the internal load can be increased during the vertical load application.

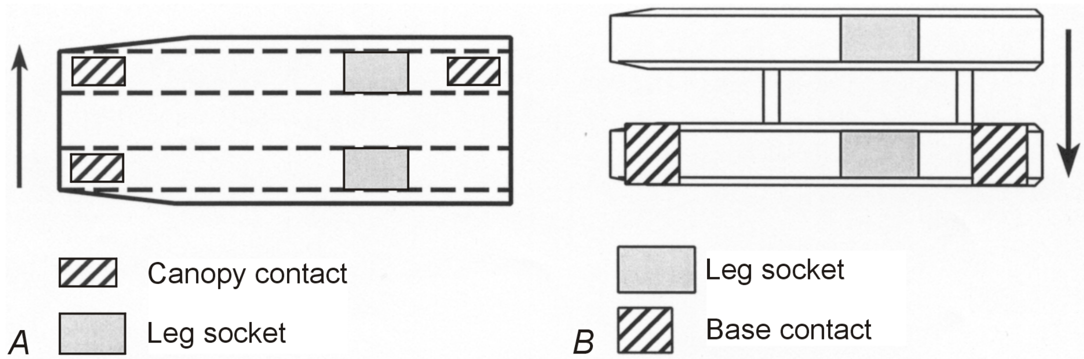


Figure 18.–Setup for test series II. A, Canopy contact; B, base contact.

TEST SERIES III – EVALUATION OF LEG SOCKET AND LEG CYLINDER INTEGRITY

Objective – Maximize stress development in the leg socket welds and expose the leg cylinder seal and piston to maximum side loading at full-stage extension.

Test Requirements – Failure of the leg sockets is a common shield problem. The leg socket is a casting that is welded along the top four sides to the side rib plates of the base and to horizontal stiffening plates in the base construction. The canopy construction is similar, except that unlike the base, the canopy is a single unit with additional stiffening plates built into the structure. The test requirement is to induce maximum loading into the welds.

Canopy and Base Contacts – For the base structure, plates are cut so that their width is approximately 50 mm less than the inside dimension of the side rib plates (figure 19A). This contact arrangement requires that the full load be supported by the bottom plate without being carried directly through the side rib plates. Therefore, the plates are designed to maximize stress development in the rib socket welds. The base contact plates are spaced 1 m apart during loading to simulate steps in the floor caused by shearer cuts. A centerline canopy contact is established as shown in figure 19B. The contact is positioned between the leg sockets to induce transverse bending of the canopy structure and focus loading on the leg socket welds.

Test Procedure

1. The shield is configured so that the upper stage of the leg cylinders are at full extension (figure 20). The transverse bending of the canopy with full extension of the upper leg cylinder staging will cause maximum side loading of the piston and seals.

2. The shield will be set against the load frame with 50 to 75 bar of leg pressure.

3. Cyclic loads are applied by controlled vertical displacement of the load frame platens. Horizontal positioning of the canopy and base will be restrained by the load frame during load application. The shield is cycled between the setting load and yield load.

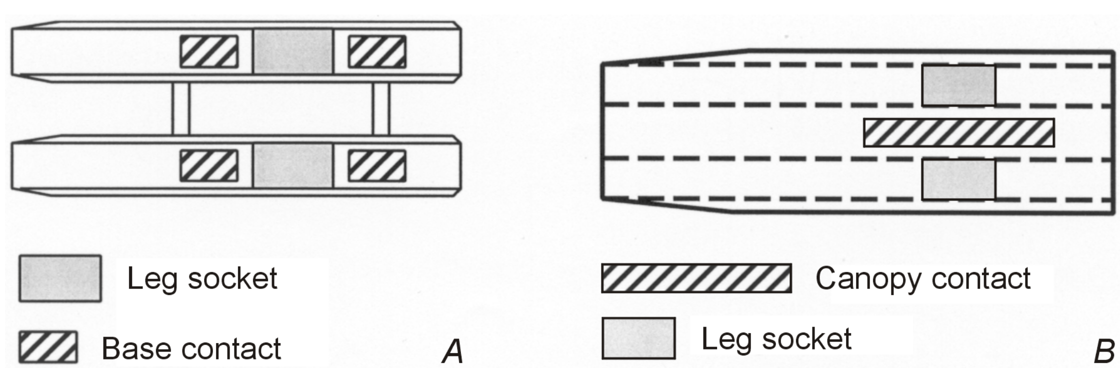


Figure 19.–Setup for test series III. A, Base contact B, canopy contact.

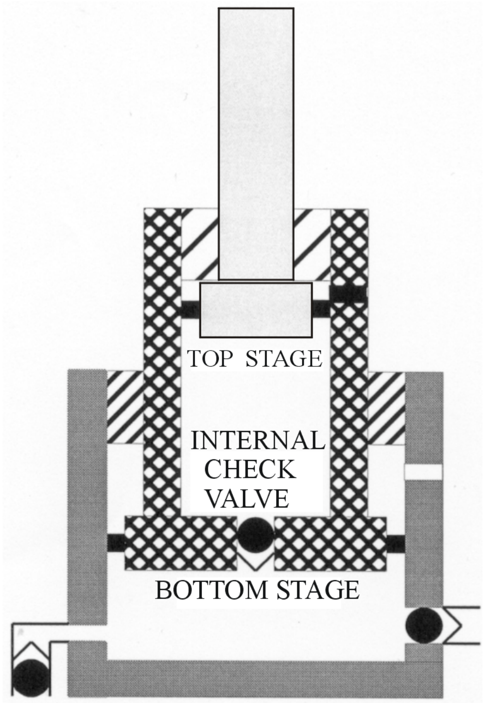


Figure 20.–Upper stage of leg cylinders is fully extended during test series III.

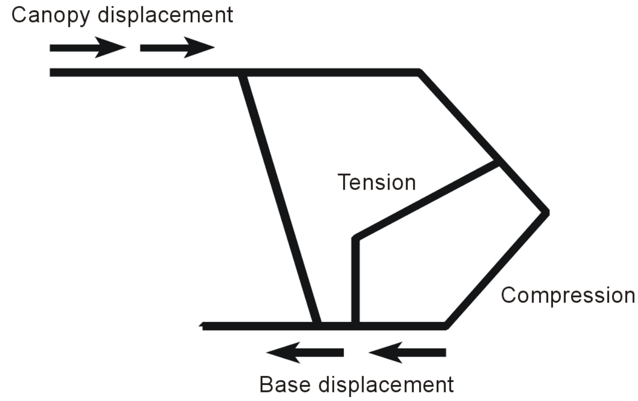


Figure 21.–Canopy is displaced toward the gob in test series IV to cause face-to-waste racking of shield.

TEST SERIES IV – FACE-TO-WASTE RACKING OF SHIELD (OPTIONAL)

Objective – Simulate the effects of roof strata pushing toward the gob.

Test Requirements – Induce a load vector that produces face-to-waste racking of the canopy with respect to the base. The caving shield-lemniscate assembly is designed to alleviate bending moments on the hydraulic leg cylinders by absorbing all horizontal loads acting on the shield. This condition is unlikely to occur in two-leg shields except at high operating heights where the leg and lemniscate link orientation is closer to vertical. Hence, this test requirement is considered optional depending on shield design and kinematics of the shield.

Canopy and Base Contacts – Full canopy and base contact is utilized to facilitate frictional contact along the canopy and base.

Test Procedure

1. The shield is positioned in the load frame so that the direction of applied horizontal displacement of the canopy is toward the gob (figure 21).

2. The shield is set against the load frame roof and floor at approximately 50 to 75 bar of leg pressure.

3. Cyclic loading is initiated by the active load frame applying a combined face-to-waste and vertical displacement of the canopy relative to the base. Vertical displacement is applied to the degree necessary to sustain horizontal loading. Horizontal displ acement is applied until the legs reach yield load. Since the horizontal displacement is in the opposite direction to that in test series I, the lemniscate link force is also opposite, with the front link acting in tension and the rear link acting in compression. This change in state of stress produces maximum wear and fatigue loading.

SHIELD INSTRUMENTATION AND DATA ACQUISITION

To monitor load development through each of the shield components, strain gages are installed at selected locations on the various shield components, as described below. The primary purpose of the strain gages is to monitor load transfer through each of the shield components. The gages are not necessarily installed in areas where stress concentration is greatest. Signal conditioning and data acquisition are provided by a data acquisition system. Typically, each sensor is sampled once a second to provide a reasonably complete load profile. Sampling rates up to 10 kHz are available if needed to assess failure developments. Both historical and real-time observations of the data are possible through the data acquisition system. The shield is inspected after every 1,000 load cycles for structural damage. A dye penetrant can be used to assess crack developments. The utilization of magnetic flux, x-ray, or ultrasonic technologies is beyond the scope of the standard test program and will require additional funds.

Canopy – Two to four gages are installed on the main vertical ribs of the canopy structure to assess bending strains. One gage is typically installed near the leg connection where the bending moment is the largest. The other gage is typically installed forward of the leg connection. Strain gages are generally installed on both the left and right side of the canopy.

Caving shield – A strain gage is installed near the canopy clevises and/or the lemniscate link clevis on the main load-transferring members of the caving shield.

Lemniscate links – One or two strain gages are installed on each of the lemnscate links to measure load development in the lemniscate links. Both axial and bending-induced strains are measured. Gages on both the top and bottom surface are used in link designs that promote bending of the link structure.

Base – A series of two or three strain gages are installed on the inside and outside ribs of each base fabrication to measure bending in the base sections. Gages are also applied to the bridge connecting the two base sections in split-base designs.

Leg Sockets – Gages are installed on the plate sections that support the leg socket in both the canopy and base.

Hydraulic – Pressure transducer is installed in each hydraulic cylinder to measure pressure development during loading.

Cycle count – The number of cycles is counted automatically by tracking leg pressure development.

DATA PRESENTATION

The applied loading and strain developments are monitored during each load cycle. A full profile of strain development during the loading cycle will be recorded at 100-cycle increments. Ten strain profiles collected during 1,000 loading cycles will be plotted on a single graph for comparison. Maximum and minimum strains will be recorded for each loading cycle. The maximum and minimum strain values will be plotted in groups of 5,000 loading cycles to examine trends over a sustained loading period. NIOSH reserves the right to modify this standard data presentation plan when extenuating circumstances dictate that other data presentations would be adequate or more appropriate.

SHIELD PERFORMANCE TESTING REPORT

A performance test report is provided as shown below. The report describes how the tests were conducted and the results of the tests. A failure assessment is made documenting time of the failure and the component(s) involved.

**SHIELD SAFETY PERFORMANCE TESTING
FINAL REPORT**

1. Scope of Work

2. Testing Objectives and Simulation Methods
    - Objectives
    - Simulation of the In-Mine Service Conditions
    - Standardized Shield Tests and Exceptions

3. Shield Instrumentation and Data Acquisition

4. Test Results
    - Hydraulic Component Evaluation
    - Cyclic Tests
        - Test Series I
        - Test Series II
        - Test Series III
        - Test Series IV

4. Failure Assessment and Problem Report

5. Conclusions and Recommendations

CONCLUSIONS

The safety of mine workers depends heavily on roof support systems that prevent the unintentional collapse of ground in both working and access areas of the mine. Support manufacturers continually strive to develop new support technologies that provide more effective roof support at less cost and with less effort to install. It is imperative that these prototype support technologies be thoroughly evaluated to make sure that they meet required design criteria for use in various underground mine conditions.

The availability of the NIOSH Safety Structures Testing Laboratory with the unique Mine Roof Simulator load frame provides support manufacturers and mine operators with the most precise simulation of underground conditions to ensure the safety of their products prior to installing prototype systems. The safety performance testing protocols developed by NIOSH for application in this world-class facility are based on years of research into support design and testing requirements. As such, they are believed to provide the best possible evaluation of support technology.

In recent years, numerous roof support technologies have been successfully developed and evaluated at the NIOSH Safety Structures Testing Laboratory utilizing the protocols described in this paper. In the past 7 years, over 1,000 tests have been conducted on various secondary roof support systems. As a result of this effort,18 new support systems have been successfully introduced to the mining industry, making a significant impact on longwall tailgate support as alternatives to conventional wood and concrete cribbing.

The NIOSH Safety Structures Testing Laboratory provides an opportunity for coal operators andsupport manufacturers to have shields tested domestically. The Mine Roof Simulator is unique in its capabilities to apply active vertical and horizontal loads simultaneously, providing realistic simulations of underground load conditions. Since the Mine Roof Simulator more accurately simulates in-mine service conditions than static frames, testing at the Safety Structures Testing Laboratory reduces the risk of premature failures due to poor design. In addition to performance-testing new shields, the remaining life of aging shields can be determined with more confidence and can provide an engineering basis for shield retirement and new shield procurement.

In summary, the benefits of shield performance testing using the unique capabilities of the Safety Structures Testing Laboratory include—

- Unbiased assessment of support performance.
- A location in the United States that improves access to mine operators.
- Freedom of control over the test program.
- NIOSH knowledge of shield mechanics and design issues.
- Capability for active loading as opposed to static-frame testing.
- Controlled loading that simulates in-mine service conditions accurately.
- Participation in research programs to improve shield design and operation.

Further information concerning utilization of the Safety Structures Testing Laboratory can be obtained by contacting Tom Barczak at (412) 386-6557, by fax at (412)386-6891, or by email at thb0@CDC.gov.

EXAMINING LONGWALL SHIELD FAILURES FROM AN ENGINEERING DESIGN AND OPERATIONAL PERSPECTIVE

By Thomas M. Barczak[1]

ABSTRACT

Longwall operators are again pushing the envelope in terms of life expectancy for longwall shields. State-of-the-art shields are now expected to last more than 60,000 loading cycles, twice the life expectancy compared with those of a decade ago. A review of trends in shield design shows that shields continue to increase in both size and capacity. Some state-of-the-art shields now weigh over 30 tons and provide up to 1,200 tons of support capacity. Although life expectancy has increased and modern shields are structurally more reliable, premature failures do still occur. This paper provides an engineering and operational assessment of shield design and provides key points to observe in what causes premature shield failures. Design practices to improve structural margins of safety that will prevent premature failures from occurring are also examined. A survey of recent shield failures is provided, as well as trends in shield design and how they might impact the performance and longevity of a shield. Hydraulic failures are more common than structural failures. Although hydraulic failures occur on all aging longwall shields, they often go undetected for long periods, resulting in degraded support capacity that can lead to serious ground control problems. The fundamentals of shield hydraulics are described in order to evaluate hydraulic failures that plague all shields at some point in their service life, and practical methods to detect hydraulic failures are examined. The paper concludes with recommendations for inspecting damaged shields and safety precautions regarding their continued use.

[1]Research physicist, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

The decline in the number of longwalls from a high of 118 in 1985 to 62 in 1999 has forced a reduction in the number of shield manufacturers through mergers from 8 to 2. The two major suppliers of shields to the U.S. market are Joy and DBT America (formerly Mine Technik America or MTA). Their share of the market is split fairly evenly: Joy has 32 faces and DBT America has 30 faces [Fiscor 1999]. The author believes that the merger of the shield manufacturers has been beneficial to shield design in two ways. First, the merger has allowed the best of the U.K. shield designs (Joy) and the best of the German designs (DBT America) to be brought forward. By combining the design strengths from each company, the result has been an improvement in both structural design and control system technology. Second, the fierce competition during the decade of the 1980s may have forced manufacturers to "cut corners" relative to design in order to compete for sales, resulting in deficiencies in design that led to premature structural failures and less than adequate control technology. Thus, while competition is generally healthy, resulting in lower prices, there needed to be a better balance of quality and price than existed in the 1980s. The hard reality is that shield manufacturers have to make a profit in order to produce a quality product and to stand behind this product with warranties; this capability was jeopardized in the past and has been improved with the recent mergers.

A survey of the longwall industry was conducted to evaluate problems with current shield technology and future needs. The survey indicates that shields are lasting longer than ever before, but that *premature failures still occasionally occur* and some mistakes from the past continue to be made in recent shield designs. Structural failures tend to be the most catastrophic, often requiring modifications to the original shield design. These failures can cause considerable downtime and loss of production, and expose the mine workers who must change out the damaged components to increased risk of injury. Hydraulic failures are common to all aging supports. These failures often go undetected, resulting in degraded support performance that can lead to serious ground control problems. Methods to detect these hydraulic failures are discussed later in this paper.

Due to the maturity of shield technology and the always present economic pressures of the mining industry, longwall operators are now keeping shields in service longer than ever before. In addition, due to the increases in longwall productivity, the number of operating cycles per year continues to increase. The result is that the operators are again pushing the envelope in terms of extending the life expectancy of the supports. *Today's shields are expected to last 60,000-70,000 cycles*; 10 years ago the life expectancy was about 35,000 cycles. Thus, while fatigue failures were not a design issue in the early generation of shield supports, they have become critical to the survival of most longwall operators in an ever increasingly competitive market. Much is being learned about the behavior of aging shields, which are kept in service long enough to fail from fatigue as opposed to failure from poor design or replacement before the end of their useful life due to technological improvements.

Deciding when to retire an aging longwall face and the specifications for a new shield can be a critical decision for any longwall operator. Most mines do not have structural engineers that can actively participate in these decisions. An overview of the fundamental engineering aspects of shield design is provided in the paper, as well as key points that should be considered in the design process to avoid premature structural failures. A primary goal of this paper is to provide mine operators with a better sense of design issues and engineering mechanics that are relevant to shield design. *The intent is not necessarily to have the operators learn enough to know all the answers, but to provide them with enough insight so they know what questions to ask* and what to look for relative to failures that may occur. These insights into shield design should also be beneficial to the Mine Safety and Health Administration in evaluating the safety of longwall shields.

The paper concludes with some practical recommendations regarding what to look for and actions to take when failures do occur. These key points will help mine operators examine failures and provide responsible actions to ensure the safety of the mine workers when shield performance is degraded.

RECENT TRENDS IN SHIELD DESIGN

The basic shield structure has remained unchanged for the past 20 years (figure 1), although the structures have grown dramatically in size and capacity. There have also been technological improvements in electrohydraulic control systems that have dramatically impacted the operation of the support. Consequences of these changes in shield design are discussed below.

MATERIAL SPECIFICATIONS

The German shield companies (currently conglomerated under DBT) have in recent years promoted the use of high-strength steels (>100,000-psi yield) to minimize component cross-sectional dimensions. This trend continues in the present with both DBT and Joy Technologies. High-strength steel applications were first used in canopy structures to minimize the cross-sectional thickness of the canopy in low- to moderate-seam applications. Today, high-strength steel in all shield designs are necessary to provide the required strength for various shield components in high-capacity designs. Although the added strength has helped to extend shield life, the high-strength fabrications are more susceptible to brittle failure and catastrophic fatigue failures. Another consequence of some high-strength steel applications is that special welding practices are required (heat control, etc.), which makes underground repairs more difficult.

CAPACITY

Support capacity has continued to increase throughout the history of longwall mining. This trend for the past 15 years is shown in figure 2 [Fiscor 1999]. Average support capacities in the United States have increased by nearly 50% since 1985 to an average support capacity at yield of 768 tons for the 62 operating longwalls in the United States in 1999. Twelve installations (19%) of the current longwalls employ shields with capacities greater than 900 tons, and 19 installations (50%) have capacities between 700 and 900 tons. The current distribution of shield capacities is shown in figure 3. The highest capacity shield used in the United States is 1,170 tons [Fiscor 1999]. As shown in figure 2, maximum shield capacities have evolved from 800-ton shields, which were common from 1985 to 1990, to 1,200-ton shields in 1999.

The increase in capacity has significantly impacted shield design and ground control capability. In order to provide the increased support capacity, larger diameter leg cylinders had to be utilized. One consequence of the larger diameter leg cylinders has been a proportional increase in shield stiffness. This increase means that the load developed in the support structure will occur at less displacement or face convergence. Although the increased stiffness may provide superior control of the immediate roof, main roof weighting that causes irresistible (in terms of the shield capacity) convergence on the longwall face will result in greater shield loading with the stiffer shield design. As a result, it is not uncommon for the high-capacity shields to be fully loaded to yield capacity just as often as the lower capacity designs that they replaced. An example is illustrated in figure 4, where a 500-ton, 800-ton, and 1,000-ton shield are all fully loaded when 0.5 in of convergence occurs.

Another consequence of the larger diameter leg cylinders is that the leg socket must be designed to accept greater loads. Historically, the leg socket has been a source for premature shield failures, and the increased loading makes the design even more demanding. In addition, the larger diameter leg cylinders require a wider and longer socket, which places further demands on the design of the socket and load transfer to the base and canopy structures. In general, the larger diameter cylinders have resulted in wider support components, which are more susceptible to torsion loading than before.

Several models have been developed over the years to determine support capacity requirements. While these concepts attempt to capture the support and strata interaction principles, state-of-the-art shield capacities cannot be justified based on these concepts. A "bigger is better" attitude has prevailed, which has been promoted by the manufacturers largely because of the demands of mine operators to improve the life expectancy of the shields. Thus, it is the life expectancy issue, along with ground control requirements, that have controlled recent developments in longwall shield design and capacity determinations.

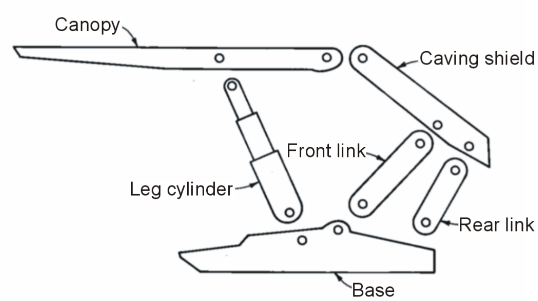


Figure 1.—Diagram of shield components .

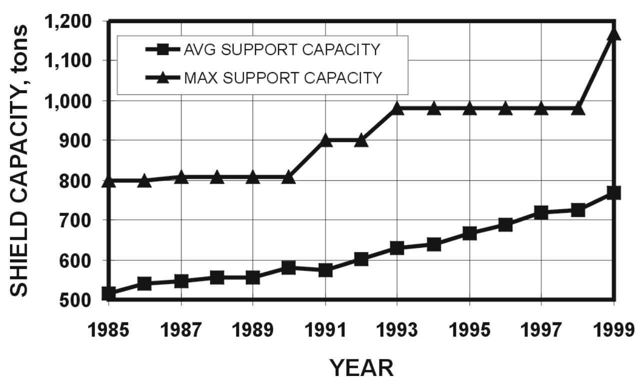


Figure 2.—Trend showing increase in support capacity.

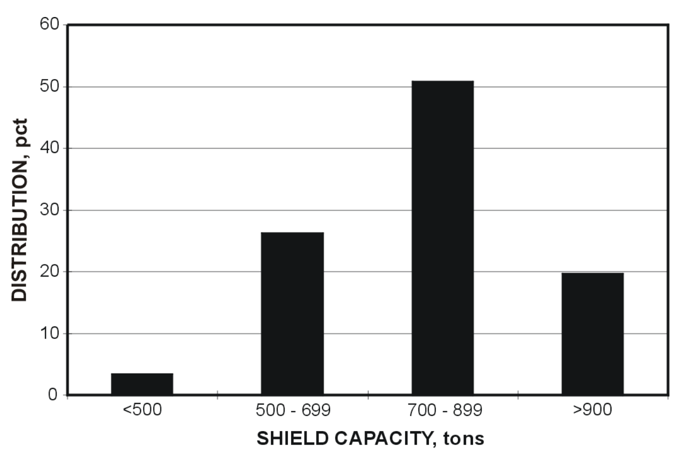


Figure 3.—Distribution of capacity for shields operating in 1999.

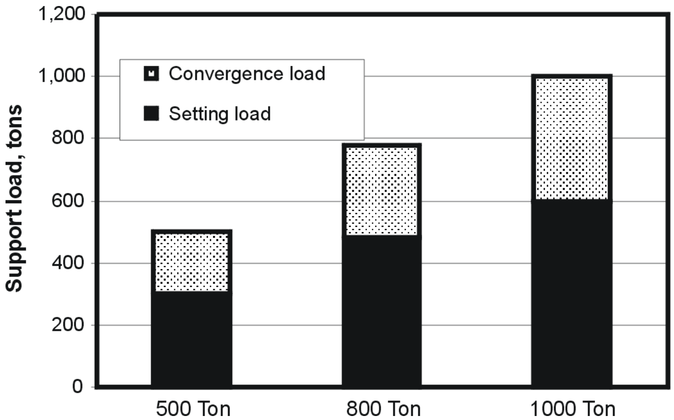


Figure 4.—Setting load and load developed due to 0.5 in of roof movement for shields of various capacities.

SHIELD TYPE

There has been a steady increase in the use of two-leg shields in favor of four-leg shields during the past decade, and two-leg shields are becoming the favored support worldwide. In 1997, 63 of the 65 longwall faces in the United States were two-leg shield systems, compared to only 53% in 1985. Larger size hydraulic cylinders have been developed in recent years to accommodate the increased demands for higher shield capacities and have allowed these capacities to be realized with two-leg designs that were not possible 10 years ago.

There are several consequences of the two-leg design. From a support strata interaction perspective, the two-leg shield provides an active horizontal force toward the coal face due to inclination of the leg cylinders. This active horizontal force improves overall strata stability by arresting slippage along fracture planes or by preventing the expansion of highly jointed or friable immediate roof geologies, which may be further damaged by the front abutment loading. In terms of the shield loading, this increase in active horizontal loading also translates into proportionally higher lemniscate link loading. Historically, lemniscate link pins and wear in bores have been a primary cause of premature shield retirement and/or rebuild. Thus, pin diameters and bore areas need to be increased in higher capacity shields to prevent this problem from becoming more prevalent.

Another issue related to the two-leg concept is higher contact pressure on the canopy and base. High toe loading, caused by the moment created by the line of action of the resultant vertical forces acting on the canopy and base, can be a problem in high-capacity two-leg shields and should be considered in the support design. Base toe pressures of $\geq$600 psi can be expected on high-capacity two-leg shields. Base toe lifting devices are now standard on most two-leg shields to assist in the advancement of the shields, particularly in soft floor conditions. There has also been a trend toward solid base designs to reduce floor-bearing pressures in two-leg shields.

SIZE

In addition to wider shields required by larger diameter leg cylinders, there is a trend toward wider shields to minimize hydraulic cylinder maintenance and reduce the total cost by employing fewer leg cylinders on a longwall face. Again the issue of torsion (twisting of canopy and base) is important in designing wider shields. It is not as easy as simply extending the width of the canopy and base. These components also need to be strengthened for torsional loading. Additionally, there is the issue of weight. The 2-m-wide designs may represent an upper limit with current shield construction materials. If high-capacity designs are to prevail, then lighter weight materials such as composites are likely to be needed to develop widths much beyond 2 m. The application of composite materials to longwall shields is an area that has not yet been explored and will require a whole new set of engineering requirements to implement into shield design. Although these materials offer significant strength-to-weight ratio advantages, there are problems regarding abrasion, pin-bearing areas where components rotate relative to one another, torsion and shear stress control, and costs.

In addition to increases in width, shields have increased in length to accommodate one-web-back operations, larger face conveyors, and deeper shearer webs. Longer canopies and bases create much larger bending moments that require stiffer and stronger components than in previous generation supports. The increase in length is largely responsible for the need for greater shield capacity as the area of roof loading carried by the shield increases or the greater convergence is seen by the cantilevering of the immediate roof beam as the resultant shield force moves further from the coal face.

SETTING FORCES

Setting forces have increased in proportion to the increase in yield capacity because the size (diameter) of the leg cylinders has increased to accommodate the higher yield capacities, while the hydraulic setting pressures have remained constant in the 4,000- to 4,200-psi range. This design practice, coupled with the increased stiffness of the higher capacity supports, means that the higher capacity shields are fully loaded as often as their lower capacity predecessors (figure 4).

HYDRAULIC COMPONENTS AND CONTROL SYSTEMS

Both Joy and DBT continue to make improvements in the electrohydraulic control systems, making them more reliable, more user-friendly, and easier to diagnose when problems occur. A description of these systems can be found in Barczak et al. [1998]. Solenoid-operated valving systems are now becoming standard. Spool valves have been shown to be superior to ball-and-seat designs, which are prone to contamination problems. In addition, these systems allow the solenoid to be activated upon demand, unlike previous systems that required the hydraulic feed to be interrupted by a control solenoid. This leads to both quicker and smoother control of support functions.

HYDRAULIC EMULSIONS

For many years, longwall shields have utilized a water/mineral oil emulsion as a hydraulic medium for the leg cylinders. The standard system has been a 5% oil/water emulsion. There is a trend toward the use of synthetic fluids. Most western mines have now switched to "low-treatment" systems with synthetic oils in concentrations of only 1% to 2%. This has largely been due to environmental issues imposed by the Utah Department of Natural Resources. Only one eastern mine is currently using the low-treatment emulsion system. Fazos, Inc., (Australia) has experimented with an all-water system.

Although the synthetic oils are environmentally preferred, they cost significantly more. The synthetic concentrate is about three to five times the cost of mineral oils. Thus, despite the lower concentration used in the low-treatment systems where less than 2% oil is utilized, the overall cost is typically about 50% greater than high-treatment systems using 4% to 5% mineral oil concentrations. The major disadvantages of the synthetic low-treatment system is that there is little room for error. A small drop in the oil concentration can lead to lubrication and acidity problems. Therefore, maintenance of the oil/emulsion is much more critical than in the high-treatment systems, where the oil content can be reduced from 5% to 4% with little, if any, detrimental effects. Bacteria growth can also be accelerated in very low concentrations of oil emulsions, which can cause more severe corrosion problems than if there were no oil at all.

SUMMARY OF SHIELD FAILURES EXPERIENCED BY MINE OPERATORS

There have been numerous shield failures throughout the history of the shield support. A summary of these is provided below. Although failures have declined, particularly premature failures that occur early in the shield life, there are still isolated cases of premature failures. Shields have a finite life, and fatigue failures will eventually occur on all shields if left in service long enough.

HISTORICAL OVERVIEW OF SHIELD FAILURES

Base Failures

Base failures seem to be the most prevalent and usually occur from fatigue after the support has been in operation for several panels of extraction. A common failure mechanism is when the leg socket casting breaks away from the base structure. Formation of this failure is difficult to detect while the support is in service, as the leg socket is housed deep inside the base structure and this area usually is full of debris. Once the leg socket breaks loose, the support quickly becomes inoperable. The bottom plates of the base have insufficient strength to withstand the leg forces, and the leg cylinder will literally rip the base apart by tearing off the bottom plate.

Failure of the base structure (plates) can also occur without the leg sockets failing. The probable failure mechanism is bending of the base. This is more likely to occur in mine sites that have very strong immediate floor strata. In these hard floor conditions, steps in the floor may be left by the shearer, as it is difficult to maintain a constant height of extraction from cut to cut. The base structure is then simply supported in two locations and is flexed as loading is applied. Repeated flexure causes the base to deform (plastically) or promotes fracture from fatigue, which eventually results in failure of the base structure. In softer floor conditions, the strata deform to provide a fuller contact to the base. This alleviates much of the bending and reduces the risk of failure. Standing the support on the toe of the base can also result in damage of the base structure. This configuration causes maximum stresses in the toe region, and the base will deform or fail usually where the cross section is a minimum in the section of the base forward of the leg connection.

Internally, the base structure is constructed with stiffeners that hold the top and bottom plates apart to form a beam arrangement, which gives the base its bending strength. Cases have been reported where these stiffeners have not been properly welded in place or where the dimensional tolerances were not within specifications. In these cases, the stiffeners broke loose and the base structures literally collapsed. This problem seems to be largely a matter of quality control, but it is critical to support safety. Since the stiffeners are hidden inside the base structure, it is virtually impossible (excluding x-ray inspection) to see these deficiencies prior to the failure.

Canopy Failures

Canopy structures are constructed of stiffened (top and bottom) plates similar to base structures, and thus they are susceptible to bending-induced failures as well. Structurally, canopies are less stiff than bases, which makes them more susceptible to failure from bending than base structures. However, while permanent deformation of the canopy is a fairly common occurrence, destructions of canopies seem to be less frequent than observed destructions of bases. This suggests that canopies are less often subjected to critical bending. Three reasons why canopies might avoid critical loading are: (1) immediate roof strata are usually partially fractured, and full contact with the canopy is more easily obtained, which minimizes bending moment; (2) tip loading on the canopy is usually smaller than toe loading on the base since the resultant force is more likely to be located near the toe of the base than the near the tip of the canopy; and (3) the canopy surface area is larger than the base area, which allows the canopy to distribute load more efficiently.

Another common deformation of the canopy is "wrinkling" of the top plate between the internal stiffeners. This is due probably to concentrated loading at locations between the stiffeners, but might also be an indication of failure of the weldments that hold the stiffeners in place. If the stiffeners are not secure, the plate may buckle from excessive stress that results in bending of the plate between the more rigid stiffeners.

Caving Shield And Links

Link members have become considerably more robust in shield designs over the past 10 years, and failures have been substantially reduced. Since the caving shield-link assembly has very little vertical load capacity (stiffness), links are not highly stressed for most load conditions. Almost all link failures can be attributed to conditions or operating practices that promote standing the support on the toe of the base or conditions that cause large horizontal displacement of the canopy relative to the base.

Failure of the structure is most likely to occur in the region near the (pin) hole located on each end of the member. The failure mechanism is most likely crack formation somewhere on the circumference of the hole from localized high stress development. The pinholes elongate from continued wear and contact with the higher strength link pins. This results in point loading of the pins and high stress development at the contact areas. These failures are difficult to detect since this area is obscured from view by the caving shield clevises or base structures. Although link failures are rare, they can be catastrophic, as the links provide horizontal stability to the support structure.

Likewise, caving shield failures are fairly rare, but are more likely to occur than link failures. While links are designed primarily for axial loading only, shield mechanics indicate the primary loading mechanism for the caving shield is bending and torsion. Maximum stresses and failure are most likely to occur in the clevis areas, where pins connect the link members to the caving shield. Some general yielding by bending deformation of the caving shield structure may also occur.

Leg Cylinders

Assuming that the face area is sufficiently stable to prevent bumps (violent outbursts of energy), it is unlikely that leg cylinders will experience structural failure since they are designed to control loading by hydraulically yielding at specified pressures. The most common failure associated with leg cylinders is seal leakage. Internal leakage may occur through other sources, which will be discussed later in this paper.

Another potential failure mechanism for hydraulic leg cylinders is malfunction of the yield valves, which allows leg pressure to increase beyond design levels. Usually, the excessive pressure will cause seal leakage, so that it is unlikely that sufficient pressure will build up to rupture the cylinder casing. The more common problem is leakage through the yield valve, which causes unplanned pressure losses during normal loading. This leakage can be caused by dirt or contamination not allowing the valve to seat properly or worn seats, defective springs that maintain the valve in the closed position, or the fitting itself may leak due to bad O-rings or seals.

FAILURE ASSESSMENT OF SHIELDS CURRENTLY IN SERVICE

Based on an informal survey at longwall mines, the age distribution of the shields operating in 1999 is shown in figure 5. The range of operating life extends from 0 to 14 years of service. An average shield operates about 4,000 cycles per year; this translates into a face production of approximately 2.5 million tons per year. Thus, an average 10-year-old shield would have 40,000 loading cycles, whereas a shield employed on a high-production longwall (4 million tons per year) would have about 65,000 loading cycles. Approximately one-third of the shields have been in operation from 4 to 6 years (16,000-24,000 cycles), and slightly more that than one-third (37%) have been operating for more than 6 years (24,000 loading cycles).

Figure 6 shows a near-linear increase in structural failures with age. Failure is defined as structural damage to any support component. It does not necessarily mean catastrophic failure that renders the support inoperable. The linear relationship suggests that *more than just fatigue failures are occurring*, since

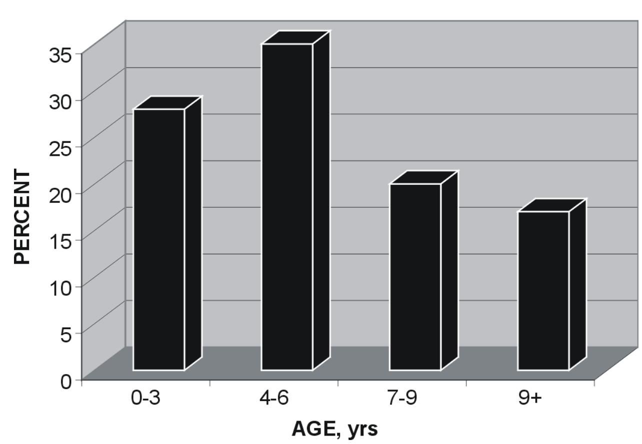


Figure 5.—Age distribution of shields operating in 1999.

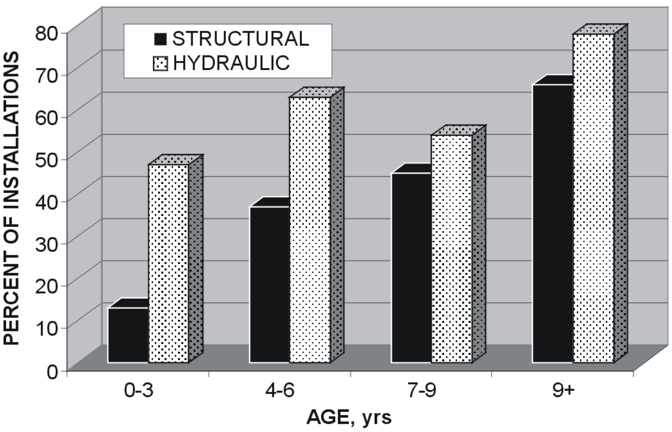


Figure 6.—Distribution of structural and hydraulic failures on shields as a function of age.

the frequency of fatigue failures should increase at a much greater rate as the shield ages. There were a wide range of structural failures reported. Most of the major failures involved either the leg sockets or pin joint problems (i.e., lemniscate link pins and clevises). These two failures are most likely to put a shield out of service and/or require structural modification to keep the support in service. Other structural failures occurred at areas of high stress concentration, including the side shield on the canopy structure, the canopy capsule tilt cylinder bracket, the base bridge, the link cutout areas in the caving shield, various mounting brackets including the pilot-operated/yield valve manifold, and dishing of canopy skin covering plate. There were a few isolated cases of leg cylinder casing problems, but no catastrophic failures.

There were *more hydraulic problems reported than structural problems* (figure 6), particularly in shields that have been in operation less than 6 years. There were nearly four times the number of hydraulic failures in shields operating 3 years or less and nearly twice as many for shields in the 4- to 6-year range than there were structural failures for the same age group. Obviously, the weak link in shield design is the hydraulic system. Numerous hydraulic failures or problems were reported. The high frequency of failures in the first 3 years of operation is distorted by failure of a component called a hydrafuse. The hydrafuse is a safety device incorporated by Joy as part of the leg cylinder design on its recent generation of shield supports in response to a fatality that occurred recently in Australia. An Australian mine worker was killed when the hydraulic fluid in the retract annulus of the leg cylinders was inadvertently pressurized while setting the support. Normally, the retract port is open through the return line back to the hydraulic reservoir whenever the leg cylinder is being pressurized. This is necessary since the pressure would be highly intensified (by a factor 30 or more) if the port was blocked, as in the case of the Australian fatality. The resultant pressure intensification caused the cylinder end cap to rupture, striking and causing a fatal injury to the mine worker.

The hydrafuse is incorporated into the retract circuit and acts as a yield valve to prevent the inadvertent buildup of pressure in the retract annulus of the leg cylinder. The design utilized by Joy features a brass clip that shears off at a predetermined load (hydraulic pressure). Once the clip is broken, the "fuse" must be replaced. With a failure pressure of nearly twice the pump operating pressure, it was expected that the fuse would provide the desired safety on the rare occasions when there was abnormal pressure and that replacement of the hydrafuse after the failure would not be an inconvenience. Surprisingly, hydrafuse failures began occurring on several shields at several mines. The failures seemed to occur whenever the support had set idle for some time and whenever the leg pressure was at or near yield pressure and almost always on the top stage retract circuit. The National Institute for Occupational Safety and Health (NIOSH) conducted a series of tests on a longwall shield under of variety of loading conditions in the Mine Roof Simulator to determine if unexpected pressures were occurring in the retract circuit. None were found. Similar measurements of retract pressure on active shields in underground mines that reported hydrafuse failures were taken by Joy; again, no abnormal pressure spikes were found. However, it is believed that a water-hammer effect is being created by the valve operation. Joy has since installed a restricter valve in the circuit to dampen out any potential pressure spike. Early indications from the field installations indicate that this may be working to prevent the undesirable failure of the hydrafuses. Unfortunately, the restricter also increases the time required to lower the top stage. Fortunately, in two-stage leg cylinder designs, the bottom stage is lowered first, and lowering of the top stage is only required when the bottom stage is fully collapsed. Thus, the increased time to lower the top stage will not be a problem during most production mining.

Other cylinder problems caused pressure losses that limited support capacity. One example was due to poor fabrication where the internal bore of the top stage was off center, resulting in a weakened casing that was unable to sustain the pressure intensification that occurred in the top stage. Other problems were reported with defective yield valves that would not reset, resulting in the inability of the shield to adequately hold load after yielding. Similar problems were reported with the staging valve, which also caused loss of pressure in the leg cylinders at a few mines. One mine reported clearance problems with the leg cylinder and the base rib plates where the cylinder leaned into the rib plates, causing internal damage to the cylinder and seal leakage. Failures also occurred with the advance ram cylinders, including failure of shuttle valve springs and structural failure of a hollow tube relay bar design.

Most hydraulic problems are related to internal leakage due to seal wear and/or corrosion of the cylinders. These problems typically begin when shields have been in service 4 to 6 yrs. As figure 6 shows, 60% of the shields had some sort of hydraulic problems during this timeframe. Many problems go undetected for extended periods of time, resulting in degraded support capacity that can contribute to ground control problems in heavy loading conditions. Methods to detect the onset of internal hydraulic leakages are discussed later in this paper. As previously described, there is a trend toward the use of low-treatment synthetic fluids in western U.S. mines. One western mine reported severe leg cylinder corrosion due to problems with a low-treatment synthetic emulsion on shields that have been in operation less than 3 years.

Problems with the electrohydraulic control systems were also reported. These included sticking solenoid valves (problem with soaping in emulsion formulation) and chattering valves due to fluid dynamics at high flows. Most mines reported that significant improvements have been made in the latest generation of electrohydraulic controls. In particular, the DBT (MTA) PM-4 system seems to be a significant improvement over the previous PM-3 design.

ENGINEERING ASSESSMENT OF SHIELD DESIGN AND FACTORS THAT CAUSE STRUCTURAL FAILURES

Structural failure can be divided into the following four basic types: (1) general yielding or excessive plastic deformation; (2) buckling or general instability, either elastic or plastic; (3) subcritical crack growth (fatigue, stress-corrosion, or corrosion fatigue), leading to weakening of the component or unstable crack growth; and (4) unstable crack extension, either ductile or brittle, leading to either partial or complete failure of a member. Structural shield failures are generally of types 1 and 3 and occasionally type 4. A better understanding of the cause of these failures can be obtained by reviewing the principles of engineering mechanics for structural design.

BASIC ENGINEERING PRINCIPLES IN SHIELD DESIGN

Classical structural engineering design is based primarily on a strength of materials approach. The goal of this approach is to define the load requirements for the structure and then to proportion the sizes and shapes of the various components to prevent tension failure or to prohibit instabilities such as buckling in a compression failure. Failure is prevented by employing safety factors to ensure that the allowable stress in components remain within the elastic range of the strength of the steel. These basic engineering principles are used in current shield design, except *much higher allowable stresses are permitted in shields* compared to the safety factors employed in the conventional design of structures, such as buildings and bridges. Failures by general yielding, such as that shown in figure 7, indicate that shields have been designed without sufficient margins of safety to prevent yielding. This is in part due to the low life expectancy of early-generation shield supports. As the demands for greater life expectancy grew, the manufacturers were forced to employ more conservative design approaches. However, even with state-of-the-art shields, it is safe to say that the margins of safety may still remain below that of conventional structural design where life expectancies are much longer and failure of any sort is unacceptable.

Since the longwall face and shield supports are advanced with each shearer cut, the shield is subjected to repetitive loading and the potential for fatigue-related failures. Fatigue is the process of cumulative damage that is caused by repetitive fluctuating loads. Ductile as well as brittle materials are susceptible to fatigue failures. Since there is no large amount of plastic deformation prior to the fatigue failure, even in ductile materials, *fatigue failures appear with little or no warning*. The failure mechanism is considered to be quite complex, but in general a fatigue crack is initiated at some microscopic or macroscopic stress riser. The crack itself then acts as a stress riser to promote localized yielding in the vicinity of the crack tip. In general, the more severe the stress concentration and the greater the load fluctuation, the shorter the time to initiate a fatigue crack. After a certain number of load fluctuations, the accumulated damage causes propagation of a crack or cracks, leading to failure of the structure.

Conventional structural design for things such as machines and bridges employ relatively conservative design practices relative to fatigue loading to ensure the desired life and safety of the structures. Typically, the fatigue (endurance) limit of conventional steels is about 50% of the tensile (yield) strength. Generally speaking, this means that the nominal load (stress) developed in the shield components must be kept below 50% of the yield strength to prevent fatigue failures from occurring, thereby providing an indefinite life expectancy relative to fatigue failure. Such a design approach for shield supports would require much larger component sizes, which would significantly increase the weight of the shield and, as such, are considered impractical for shield design. As the stress is increased beyond this endurance limit, the number of cycles to failure is reduced, starting at about 1 million load cycles. For a life expectancy of 100,000 cycles, which is more representative for longwall shields, the allowable stress to prevent fatigue failure is much closer to the yield strength of the steel, suggesting that a factor of safety of 2 is not needed in shield design, at least in relation to fatigue issues. Of course, these generalities are subject to the specific properties of the steel and the support construction. The allowable stress may be considerably less in some circumstances. For example, the allowable stress to prevent fatigue failure for a plate welded on the flange of an I-beam (similar to a plate welded on the bottom of the base pontoon) is only 15 ksi [American Institute of Steel Construction 1980].

In addition to fatigue, stress corrosion causes failures to occur under *statically* applied loads with stress developments well below the yield strength of the material and independent of the number of load cycles. This can account for structural failures that occur at low-production mines with aging shields where the number of cycles is low, but the age is high. In other words, a shield does not necessarily have to have many cycles to fail. The failure occurs due to crack initiation caused by the corrosion and subsequent propagation of this crack or material defects similar to those observed for fatigue loading. The author believes that *stress corrosion is a significant and the most overlooked factor in current shield design.* The mine environment is often wet and acidic, which accelerates corrosion of shield components, as shown in figure 8. The base units in particular are subjected to extensive corrosion since the wet debris continually forms around and often covers much of the base structure. An area of particular concern is the leg socket, which collects rock and coal debris throughout much of the shield's operating life. When combined with fatigue loading (corrosion-fatigue), stress corrosion causes a further reduction in the useful life of the longwall shield. Corrosion-fatigue damage occurs *more rapidly* than would be expected from the individual effects or from the algebraic sum of the individual effects of fatigue, corrosion, or stress-corrosion cracking.

The science of fracture mechanics was developed during 1946-66 to analyze unexplained failures in several large-scale complex structures where brittle fractures were observed at stress levels no larger than were expected when the structure was designed [Barsom and Rolfe 1987]. The underlying premise in fracture mechanics is that real structures contain numerous discontinuities of some kind. These discontinuities can be flaws in weldments, poor fabrication practices where torch cutting leaves surface scars and abrasions, or cracks in base or weld materials due to factors like corrosion. These discontinuities act as stress risers, and unstable, rapid fracturing occurs when the stress intensity factor at the crack tip reaches a critical value. Such discontinuities certainly exist in longwall shields and are undoubtedly a *primary source of structural failures.*

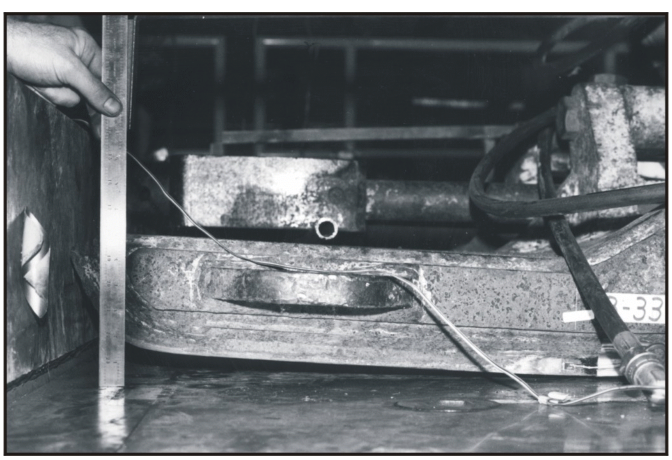


Figure 7.—General yielding of toe of base unit in shield design from early 1980s.

The goal in fracture mechanics design is to keep the stress intensity level below the critical value that promotes crack propagation (figure 9), much like the goal of strength of materials design is to keep the design stress within the material's elastic range. To ensure that a structure does not fail by fracture, the number of cycles to grow a small (often microscopic) crack to a critical crack length must be greater than the life of the structure. Thus, the key idea to consider in a fracture mechanics design is the ability of the steel to absorb strain energy. If the material has a high absorption capability, such as mild steel, crack growth will be limited to plastic flow of the material, and general yielding will typically prevent brittle fractures from occurring. Conversely, the probability of fatigue-related failures increases with low-energy absorption materials. Unfortunately, from a fracture mechanics perspective, state-of-the-art shields are now constructed from high-strength steels (100,000- to 120,000-psi yield strength), which are much *more susceptible to brittle fracture* than the early generation of shield supports, which were constructed from

Figure 8.—Corrosion of a lemniscate link clevis.

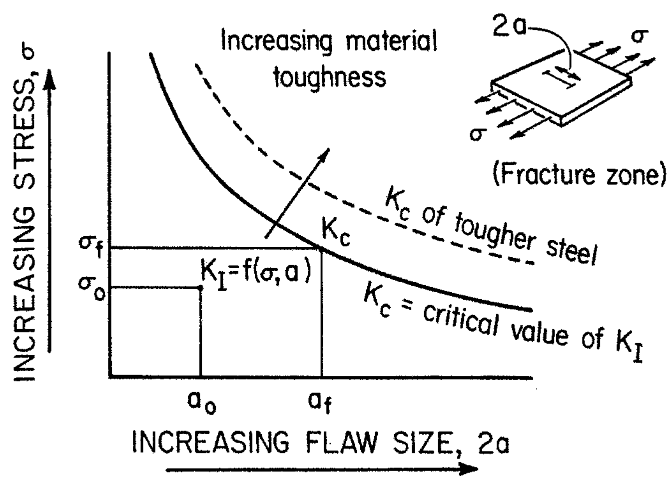


Figure 9.—Fracture mechanics principles regarding material toughness, stress, and flaw size.

mild steel (35,000- to 45,000-psi yield strength). The motivation for the high-strength steel, of course, is to minimize the size of the components and keep the weight of the shield down, which is an ever increasing challenge for the design engineers as the capacity of the shields continue to rise. In essence, the term high-strength steel is a misnomer for two reasons. First, the fracture toughness is reduced, which means that the allowable stress, as a percent of the yield strength, must also be reduced to prevent fracture from occurring. In other words, the full advantage of increased strength of the steel may not be realized. Second, unlike lower strength steels where the weld material is of equivalent or higher strength of the base metal, it is not uncommon for weaker weld materials to be used in the high-strength steel constructions.

DESIGN PRACTICES TO IMPROVE STRUCTURAL MARGINS OF SAFETY AND EXTEND SHIELD LIFE

Since most structural shield failures can be attributed to some form of fracture, the basic elements of fracture control can significantly improve shield life. These are: (1) use a lower design stress, (2) minimize stress concentrations, (3) reduce flaw size or control crack growth, (4) minimize corrosion, and (5) use materials of improved toughness.

Lower Design Stress

Some margin of safety should be employed in the design stress relative to the yield strength of the steel. Civil engineers typically use a factor of 1.66, which means that the allowable stress is about 60% of the yield stress for nonfatigue loading and further reduced by 50% or more when fatigue loading applies. While these levels of safety are not practical in shield design due primarily to cost and weight limitations, it is important to recognize that a small reduction in (tensile) stress developments will significantly reduce crack growth since the two are related by an exponential function. Past practices of designing to or near yield strength should be avoided in modern shield design where the life expectancy exceeds 50,000 loading cycles.

Link pins and clevises are a prime example of historically poor design practices in shield supports, which continue even today. Deformation and/or excessive wear in the pin clevises is undoubtedly the primary cause of premature shield retirement and/or structural rebuild. There are clear indications that these areas are subjected to stress beyond the yield strength of the steel. This poor design is caused partly by manufacturers not giving sufficient credence to the conditions that cause high loading in the caving shield-lemniscate assembly, namely, loss of frictional contact at the roof and floor interface and standing the support on the toe of the base. With the possible exception of shields designed for low-seam heights, there is adequate space available to increase the bearing area of these clevises and pin diameters to reduce the stress and substantially improve the life expectancy of these components. The joint design problem also needs to recognize the importance of pin tolerance. First, the pin contacts only a portion of the clevis. Typically, arcs of 45° to 60° are used in the design analysis. Obviously, the 45° arc assumption will lead to more conservative designs. Conservative assumptions should be made to allow for reduced areas as wear occurs (figure 10). Excessive pin tolerances can lead to point loading by allowing the pin to rack within the joint clevis, as shown in figure 11. Corrosion effects are also often ignored or not sufficiently accounted for in the design of pins, despite the fact that corrosion is a leading cause of premature pin failures or abnormal wear. Corrosion causes pits to occur in both the pins and clevises (figure 8), which can reduce the bearing area by 25% to 50% and cause a proportional increase in stress.

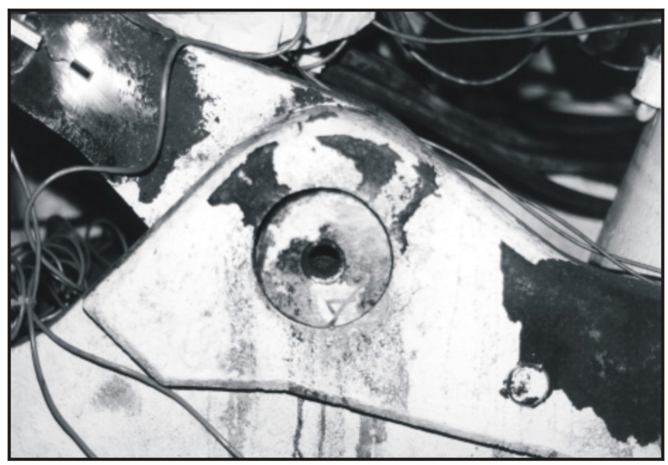


Figure 10.—Reduction in lemniscate pin contact area for worn joint.

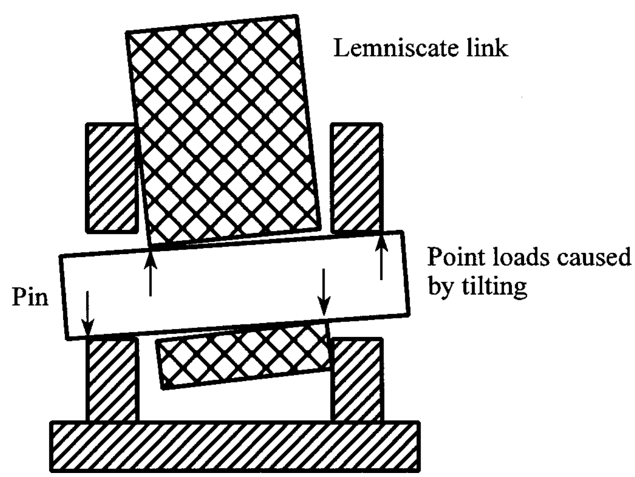


Figure 11.—Racking of pin joint.

Several failures have occurred where the leg socket casting weldments break because of fatigue or stress-corrosion and thereby cause failure of the bottom base plate as the loading is transferred fully to the bottom plate instead of being distributed to the side rib plates. Most of the time, the leg cylinder and casting punch through the bottom plate and into the mine floor, rendering the support inoperable. A common modification to alleviate base leg socket failures of this nature is to add another plate to the underside of the base pontoon, as shown in figure 12. Typically, this plate is about 1 in thick and usually covers most of the length of the base pontoon. Reinforcement is also typically added to the top area of the side base plate. While this plate stiffens the side plate, its primary purpose is to restore the location of the centroidal axis, which was changed by the addition of the bottom plate, to its location in the original base design. Because of past failures of this nature, support manufacturers are now beginning to incorporate a thicker bottom plate in the initial shield design.

Some mines have successfully reduced operating stress levels by *derating the shield support* before underground installation. The derating is accomplished by installing yield valves with a lower operating pressure than specified in the design. Since the leg capacity controls the maximum load developments within the support, lower leg loads translate into reduced component loading. For example, if a 1,000-ton shield support is derated by 10% to a 900-ton capacity, the margin of safety relative to the tolerable crack size that will prevent fatigue fracture may increase by 20% to 30%.

Minimizing Stress Concentrations

There are numerous sources of stress concentrations in longwall shield supports. The most common is a change in geometry. These stress concentrations should be identified and their magnitudes quantified during the design and performance testing phase. One way to do this is to use photoelastic plastics. The photoelastic plastic can be applied to almost any area of the shield structure. Colored fringes will appear on the plastic when observed through a polarized lens that correlate to the stress profiles. *Stress intensity factors of 2 to 3 are not uncommon* for sharp changes in geometry, such as holes or sharp bends in structural members.

One example of a sharp change in geometry is a lemniscate link design with offset pinholes, as shown in figure 13. This link design is typically employed on a shield with a low profile designed for operation in low-coal seams. The bend is necessary to provide clearance with the caving shield in the collapsed or low operating height. Lemniscate links are primarily axially loaded members, but the offset pinhole geometry induces additional stresses due to bending and thereby significantly reduces the margin of safety for this component. Figure 13 depicts failure of a bottom lemniscate link on a 620-ton Westfalia shield that occurred during performance testing at NIOSH's Safety Structures Testing Laboratory. Although the shield had 45,000 load cycles from underground service before testing, a new link was installed on the shield that was performance tested in the laboratory. This was a new link design that was fabricated for the mine by an outside vendor (not the support manufacturer). Failure occurred after only 14,000 loading cycles. This failure illustrates two problems. First, the design (allowable stress) was too high. Test results revealed that the nominal stress in the link exceeded 90% of the material yield strength. In addition, it appeared that the link side plates had been torch cut, adding an additional stress riser to the already sharp change in geometry at the bend in the link in the area where the failure occurred. These two factors resulted in an unacceptable time to failure. Figure 14 illustrates another problem where the fabrication process left large flaws that led to premature fatigue failure of the base rib. This problem may have been alleviated if the surface had been smoothed to remove most of the surface flaws.

Any hole in a structural plate is another area where stress is concentrated. The structural components of a longwall shield (canopy, caving shield, lemniscate links, and base) are connected by a pin and clevis arrangement. These areas are also sources of stress concentration and fatigue failures on aging longwall shields (figure 15). Another example of a stress concentration caused by a sharp change in geometry is shown in figure 16. Holes are sometimes cut into the canopy or caving shield structure to accommodate the placement of hydraulic hoses. In the example shown in figure 16, failure occurred at the stress riser caused by the sharp corner in a cutout on the caving shield made to accommodate hosing for the side shield. The best solution to this problem is to avoid the hole altogether, and if the hole is necessary, to ensure that the corner radius is as large as possible. Another example of failure due to stress risers created by sharp geometries is shown in figure 17, where a crack developed in the caving shield near the canopy hinge. A clean-out hole is often placed in the side of the base structure to facilitate removal of debris from the leg socket area. This too can be a source for concentration of stress. Since this is a critically loaded area of the base, care should be taken in the design to minimize the stress concentration by a incorporating a favorable geometry and orientation with respect to the stress field in this member.

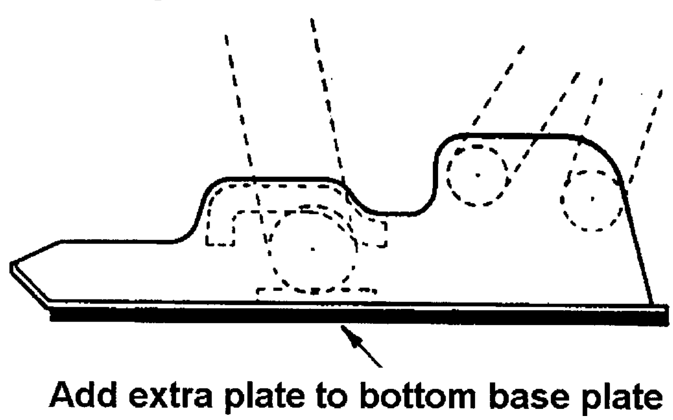


Figure 12.—Modification made to strengthen base in order to prevent leg socket casting failures.

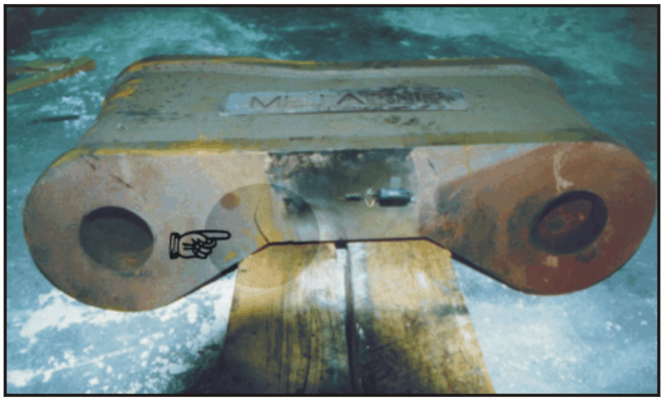


Figure 13.—Failure of bottom lemniscate link.

Figure 14.—Failure of base rib side plate where fabrication process left large flaws.

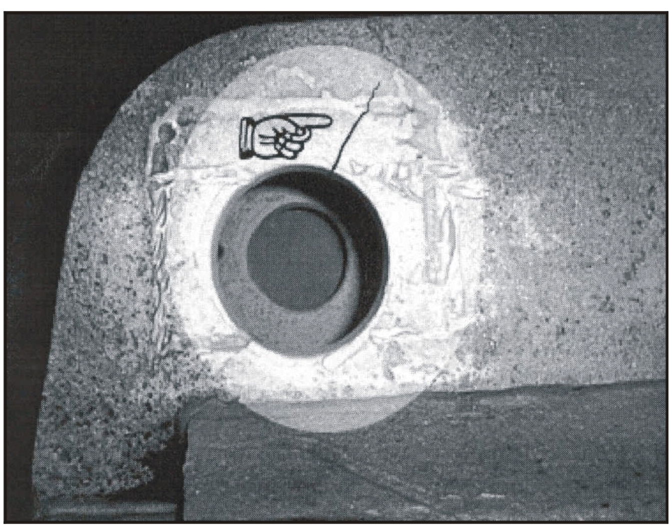


Figure 15.—Failure in base lemniscate link clevis.

Figure 16.—Failure due to stress concentrations in cutout sections of caving shield.

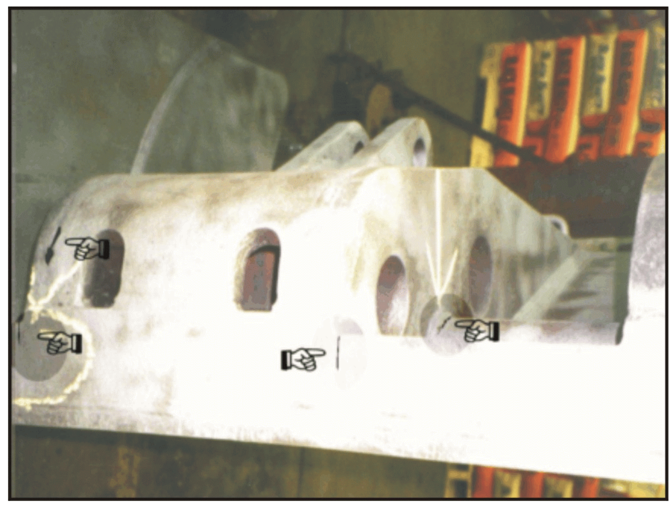


Figure 17.—Failures of caving shield due to stress concentration in sharp corners where lemniscate links connect.

Figure 18 shows failure of a canopy leg socket casting due to a stress riser created by a sharp change in geometry. The failure in this case occurred in the casting itself. This failure occurred on several shield supports in the late 1980s, all of which used this same basic socket design. Again, this failure probably could have been prevented by a smoother geometry. Leg sockets are often a source of fatigue failures in aging longwall shields. The leg socket is a critical area since the full load developed within the hydraulic leg cylinder must be transferred into the canopy and base structures to be distributed to the mine roof and floor. An examination of the structural mechanics associated with these base socket failures has led to some other design changes that are intended to reduce the stress concentration in the leg socket casting/base structure connection. The rectangular geometry of the socket casting and its placement between the side rib plates of the base structure position it at right angles to the principal tensile stress caused by the bending of the base structure. This orientation creates a stress intensity factor, which helps to promote fatigue-induced fracturing of the weldment. Some shields are now being designed with an elliptical or zipper-shaped casting (figure 19) so that the front and/or rear edge is not perpendicular to the principal tensile stress, thereby resulting in a reduction of the stress intensity factor.

Reducing Flaw Size or Controlling Crack Growth

Weldments are an essential part of the shield fabrication process. However, since weldments are a primary source of structural flaws, the quality of the welds is critical. As the principles of fracture mechanics illustrate, the initial flaw size created by the welding process is critical to the crack propagation and the margin of safety achieved in this structure. Once a crack develops, it is desirable to keep the crack contained within the weld and not have it progress to the base material adjacent to the weld. This action may depend on the nature of the heat-affected zone in the immediate vicinity of the weld. In general, the heat-affected zone results in anisotropic material properties and residual stresses that tend to reduce the toughness of the steel in this area and increase the likelihood of fracture into the base metal. For the high-strength steels used in modern shield supports, proper heating and cooling of the steel during the weldment process are crucial to preventing crack initiation and growth in the heat-affected zone. This is why it is very difficult to conduct repairs to damaged shields underground, since it is virtually impossible to be able to preheat and properly cool the steel when welding at the longwall face. Another approach that can be used to keep crack growth contained to the weldments is to create breaks in the weldments. This practice is sometimes used in leg socket castings. The break in the weld at the corners of the casting acts like as a crack arrester to stop the growth of the crack. This same technique is used in the airplane industry by drilling holes in the metal at the end of an observed crack before it reaches a critical crack length.

Perhaps the best approach to avoid problems associated with weldments is to eliminate them when possible. A good example of this pertains to the leg socket design. A typical construction for the leg cylinder base socket is shown in figure 20. A casting 3-4 in thick and 18-24 in long with a spherical seat to accommodate the bottom of the hydraulic cylinder is placed on top of the bottom cover plate on the base pontoon and is welded in place along the four sides of the top of the casting to the side rib plates and cross plates that connect the two side base plates together. This design is highly dependent on the welds to transfer load into the side rib plates and maintain the structural integrity of the socket connection. An alternative design that is now being used in some canopy leg sockets to alleviate the weld fatigue problem is to cut rectangular holes into the side base rib plates and extend the width of the casting so that it bridges across the cylinder opening, but is supported by the side rib plates (figure 20). In this configuration, leg cylinder loading is transferred directly to the side rib plates of the base structure entirely through base metal contact and is not dependent on the weldments to achieve this load transfer.

Corrosion Control

Some steps can be taken to circumvent the problems caused by corrosion. The most convenient approach is to try to protect the shield from the environment. For shield supports, this applies mostly to painting with an industrial paint that is resistant to the wet mine environment. However, for some components such as the pins and clevises where there is considerable wear due to the kinematics of the shield during load application, painting or even plating with more resistant material is not an option. Sherardizing pins has proven beneficial, but the effectiveness is limited due to the wear in the joint. Another option worth considering is some form of lubrication for these joints. A simple grease fitting would be an improvement, although somewhat impractical to maintain on an active longwall face. Thus, a sealed joint of some form would be preferable.

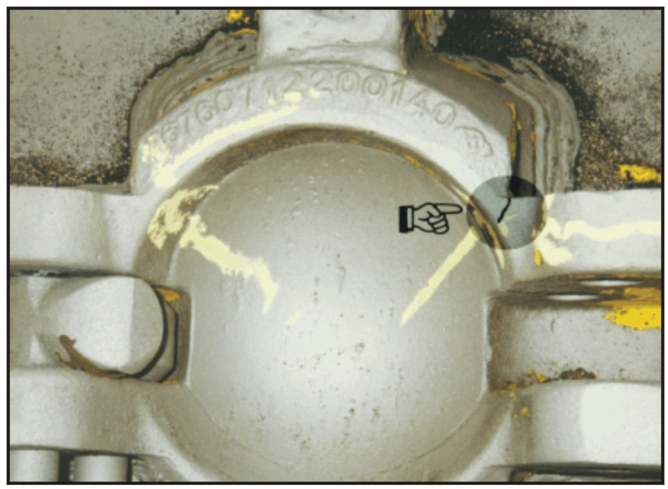


Figure 18.—Failure due to stress riser created by sharp corner in canopy leg socket casting.

Figure 19.—"Zipper-shaped" leg cylinder casting to avoid stress concentration caused by orientation of casting to principal stress field.

Tests have shown that compressive stresses on the surface of materials that are exposed to the environment will not necessarily minimize corrosion-fatigue crack initiation, but they can reduce the possibility of crack growth and thereby prevent failure from occurring. This could be done by induction hardening the joint pins. This, however, might make the pins more brittle.

Material Toughness

The strength of materials approach essentially ignores the toughness of the material, which as described is the most significant material property for fracture control. There is a tradeoff in the use of high-strength steels if they provide superior strength but reduced toughness with greater chance for failure. Since the author is not familiar with the German and U.K. steels used in the shield construction, no specific recommendations are made here, but the issue should be investigated with the shield manufacturer when new shields are purchased.

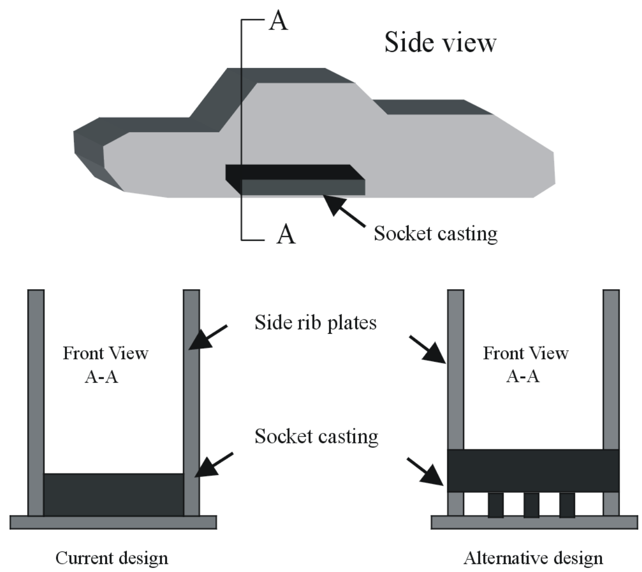


Figure 20.–Conventional and alternative leg cylinder casting design.

KEY POINTS IN EVALUATING HYDRAULIC FAILURES

As described earlier, hydraulic failures are common on aging longwall shields. Although hydraulic failures are generally not as catastrophic as structural failures, they cause a reduction in supporting capability that can lead to serious ground control problems.

FUNDAMENTALS OF SHIELD HYDRAULICS

Hydraulic Cylinder Operation

Since the capacity of a shield is controlled by the hydraulic leg cylinders, a basic understanding of their operation is essential to understanding shield design and causes of hydraulic failures [Barczak and Gearhart 1998]. The basic operation of a hydraulic cylinder can be described as follows (see figure 21). A hydraulic power supply pumps fluid into the cylinder cavity. The fluid acts against a piston, causing the piston and attached steel rod to displace outward. This displacement will continue with very little hydraulic pressure until the support is set against the mine roof and floor. The hydraulic pressure then increases rapidly until the full pump pressure is reached. After the pump supply is turned off or isolated from the support, this pressurized hydraulic fluid is trapped inside the cylinder by a pilot-operated check valve. The pressure of the hydraulic fluid inside the cylinder is then intensified in proportion to any increase in roof loading. A yield valve limits the maximum pressure inside the cylinder to prevent excessive loading that would damage the cylinder. In longwall shields the cylinders are double acting, which means that they are hydraulically powered to both extend and retract (figure 21). Powered lowering of the support is achieved by pumping fluid inside the retract annulus (figure 21) while at the same time applying pilot pressure to open the check valve to allow fluid from inside the main cylinder cavity that supports the weight of the mine roof to escape back to the hydraulic power supply tank.

The previous example describes the basic operation of a single-stage cylinder, where extension and retraction is provided by one stage. Longwall shields utilize two-stage hydraulic cylinders, as shown in figure 22. Incorporating more stages into a leg cylinder design generally allows for a lower collapsed height, combined with an equal or greater maximum working height to provide operation in a wider range of mining heights.

The basic operating principles of these multistage designs are the same as the single-stage hydraulic cylinder previously described, but there are some noteworthy differences relative to how the individual stages perform:

1. The bottom stage extends and retracts first, followed by the top stage. The bottom stage will extend until it is fully stroked before the top stage will begin to extend.

2. A check valve is installed in the bottom-stage piston (figure 22). This check valve functions to allow hydraulic fluid to flow from the bottom stage to the top stage to cause the top stage to extend during the setting operation whenever the bottom stage is fully extended. It also isolates the bottom stage from the top stage to allow the hydraulic fluid to be intensified in the top stage once the support is actively set against the mine roof and floor. This is necessary to allow the top stage to carry the same load as the bottom stage.

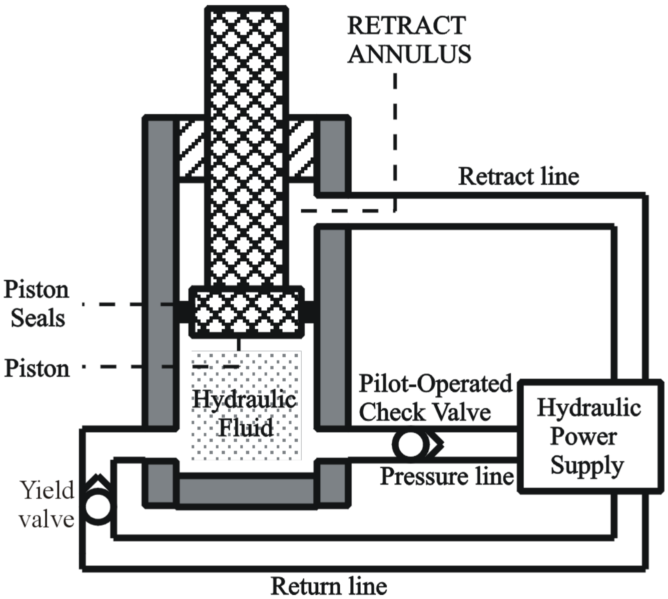


Figure 21.—Functional diagram of hydraulic cylinder operation.

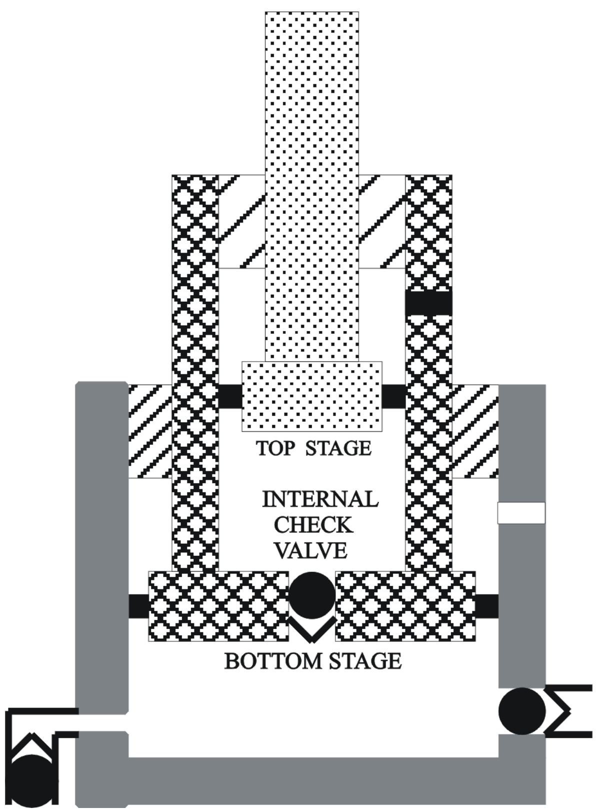


Figure 17.—Failures of caving shield due to stress concentration in sharp corners where lemniscate links connect.

Understanding how the staging functions to provide the necessary support capacity is critical to a proper understanding of multistage leg cylinder operation. For proper operation of the support to occur, force equilibrium must be satisfied for each stage, which means that each stage must carry the same load. This requires different hydraulic pressures in each stage, since the area of each stage is different. Mathematically, this requirement is expressed by equation 1. If the top stage area is one-half that of the bottom stage area, the pressure in the top stage will be twice that of the hydraulic fluid in the bottom stage.

$$A_1 * P_1 = A_2 * P_2, \tag{1}$$

where A_1 = area of largest diameter (bottom) stage, in^2;
P_1 = pressure in largest diameter (bottom) stage, psi;
A_2 = area of the top stage, in^2; and
P_2 = pressure in the top stage, psi;

The extension of specific stages at a particular point in time (cycles of operation) depends on the history of the operating heights of the support. When the support is initially raised from a fully collapsed position, the bottom stage will extend first. If the mining height is greater than the stroke of the first stage, the top stage will extend until roof contact is made. On subsequent cycles, the top stage will remain at the initial extension until (1) the mining height is increased beyond that of the initial cycle, or (2) the support is lowered such that the bottom stage is fully collapsed. Assuming that the support is not lowered to the point where the bottom stage is fully collapsed, the top stage will extend beyond the initial extension only when the mining height is increased. In essence, the bottom stage will be fully stroked (1) when the support is initially raised from a collapsed position and (2) on the mining cycle that establishes a new maximum operating height, whenever the support height is higher than it has been on all previous mining cycles. On all other cycles, the bottom-stage extension will reflect changes in mining height relative to the initial mining height and the maximum operating height at which the support was utilized.

An example is used to illustrate these concepts. As shown in figure 23, the initial setting of the support causes the bottom stage to be fully extended 24 in (stroke) and a 12-in (partial) extension of the top stage. If the roof-to-floor mining height is reduced on the second operating cycle by 3 in, the bottom-stage extension now will be 21 in, while the top-stage extension will remain at 12 in. If the mining height is increased by 6 in from that of the second cycle, the bottom stage will again be fully extended (24-in stroke) and the second-stage extension will increase to 15 in on the third cycle. This exercise can continue as shown in figure 23. As shown in this exercise, the top-stage extension on any cycle will equal the initial extension plus the incremental increase in mining height beyond the initial mining height. The top-stage extension will never be less than the initial extension unless the support is lowered to the point where the bottom stage is fully retracted. The bottom-stage extension will fluctuate with changes in mining height on each cycle and will be fully stroked whenever a new maximum operating height is attained.

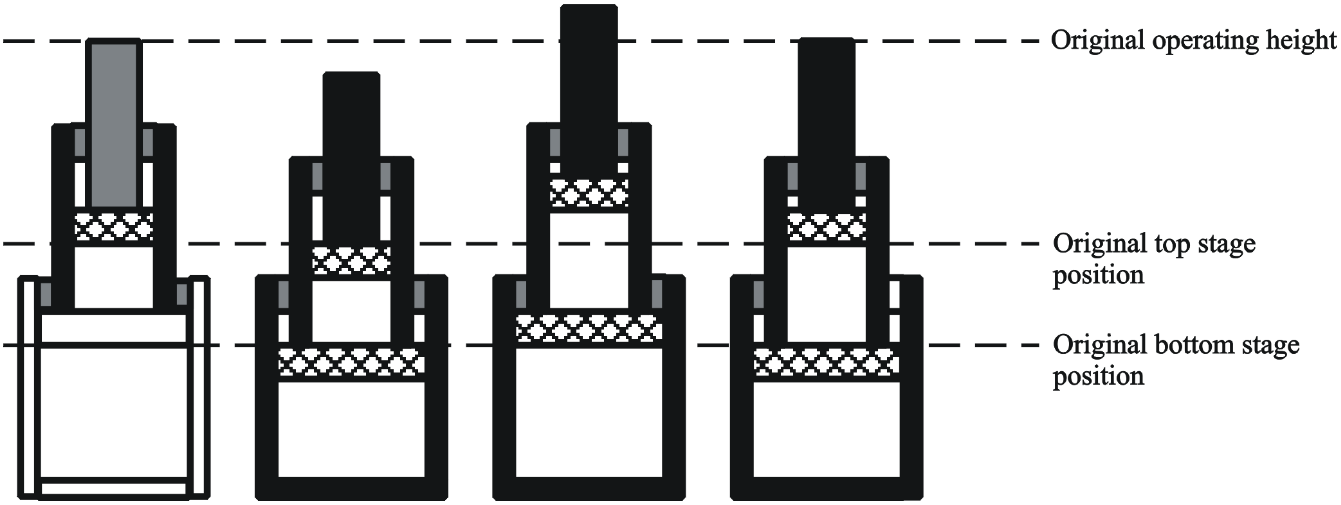


Figure 23.—Example of stage extension for changes in mining height.

Active (Setting) Loads

The shield is actively set against the mine roof and floor by pressurizing the hydraulic cylinders. The force exerted against the mine roof and floor is dependent upon the size of the cylinder and the hydraulic (pump) pressure. In multistage cylinders, the setting force can be determined by measuring the pressure in any stage and multiplying it by the (piston) area of that stage. It is important to remember that the pressure in the top stage is intensified during the setting operation, and only the bottom stage will be at pump pressure once the support is set (assuming that none of the stages are fully stroked). In longwall shields, the hydraulic pressure is measured only in the bottom stage; thus, the pump pressure times the bottom stage area should be used to calculate the setting force. The total setting force for the support is simply the sum of individual hydraulic cylinder forces.

If the bottom stage is fully stroked, the setting force will be diminished. When the bottom stage that is fully extended no longer transfers all of its force to the mine roof and floor, part of it is consumed by tensioning the cylinder casing as the piston is forced against the mechanical stops that limit its travel. The pressure in the top stage will not be intensified, but instead will be at pump pressure. The setting force will always equal the pump pressure times the area of the largest diameter stage that is not fully extended. Thus, the setting force will equal the pump pressure times the top stage area whenever the bottom stage is fully extended.

The reductions in setting force due to full extension of the bottom stages are typically between 40% and 50% for most shield designs. Thus, a 1,000-ton shield with a setting load of 670 tons at 4,200-psi pump pressure would have a setting force of only 268 to 335 tons when the bottom stage is fully extended. Recalling the operation of the hydraulic cylinder previously described, the setting force will be diminished whenever the bottom stage is fully extended, which occurs on the operational cycle where a new maximum operating height is attained. The setting force will be restored to its full capability on all other cycles or whenever the support is operated at a height less than the highest operating height on any previous operating cycle.

Yielding Behavior

Yield valves are normally connected to the bottom stage of multistage hydraulic cylinders to provide overload protection. These valves open whenever the pressure exceeds the design threshold and allow the pressurized fluid to escape from the bottom stage of the cylinder. This loss of fluid causes the pressure to drop in the bottom stage until the valve reseats. The reseating pressure is typically about 90% of the yield pressure; thus, the support capacity will drop by approximately 10% when the yield valve opens. As the pressure drops in the bottom stage, a force imbalance occurs between the top and bottom stage, which causes the bottom stage to lower until force equilibrium is attained. The top-stage extension does not change during yielding until the bottom stage is fully collapsed.

How much the bottom stage lowers because of yielding depends on three factors: (1) the area of the bottom stage, (2) the extension of the bottom stage, and (3) the yield pressure and reseating pressure of the yield valve. The reduction in support height due to yielding will be more for large extensions of the bottom stage than for small extensions of the bottom stage, decreasing in direct proportion to reductions in the bottom-stage extension at the time of yielding. Thus, on each successive yielding during any one operating cycle, the lowering of the bottom stage will be progressively less. The magnitude of bottom-stage lowering due to yielding can be calculated using equations 2 and 3. Using a 700-ton longwall shield with a bottom-stage diameter of 11.8 in and a yield pressure of 6,389 psi as an example, the bottom stage of each hydraulic cylinder will drop approximately 0.028 in during a single yield event if the bottom stage was extended 14 in when yielding began. A 70-ton reduction in support loading would be caused by the yielding.

$$\Delta V = \frac{\Delta P * V}{\beta}, \qquad (2)$$

where ΔV = change in volume of fluid in bottom stage of cylinder, in^3;
ΔP = change in hydraulic pressure in bottom stage of cylinder, psi;
ΔP = yield pressure minus reseating pressure, psi;
V = volume of fluid in bottom stage at time of yielding, in^3;
V = area of bottom stage times extension of bottom stage, in^3; and
β = bulk modulus of oil, psi (i.e., 320,000 psi).

$$\Delta D = \frac{\Delta V}{A}, \qquad (3)$$

where ΔD = reduction in bottom-stage extension due to yielding, in;
ΔV = reduction in volume of fluid in bottom stage due to yielding, in^3; and
A = area of bottom stage, in^2.

DETECTION OF HYDRAULIC FAILURES

It is important to realize that all shields will experience leakages due to seal wear and/or component failures several times during their life expectancy in the mine. Also, many of these failures will go undetected for extended periods of time, resulting in significantly degraded shield capacity and ground control capability. Furthermore, if shield life is to be maximized, then recognition and correction of hydraulic failures are important. If both leg cylinders are not functioning properly, eccentric loading in the canopy and caving shield-lemniscate assembly, for example, will be created, causing increased probability of structural failure in these components. Additional loading may be transferred onto adjacent shields, thereby reducing their life expectancy. Degraded support capacity may induce more severe weighting of the shields, creating a snowball effect that further degrades the support capability.

Observations of the relative positions of the cylinder staging can be used to identify cylinder problems and the cause of hydraulic leakages. One indication of internal leakages is when the bottom stage is consistently fully extended. The bottom stage should be fully extended only on operating cycles that establish a new maximum operating height. Thus, on the majority of operating cycles, the bottom stage should not be at full extension. Another indication of hydraulic leakage is when there is a large difference in stage extensions of leg cylinders on the same support (figure 24). Observation of the bottom-stage position can help to identify the component failure (table 1).

Table 1.—Stage movements associated with internal component failures

Component failure	Bottom-stage movement
Staging check valve	Up.
First-stage seals	Down.
Second-stage seals	Up.
Pilot-operated check valve . . .	Down.
Yield valve	Down.

Problems with the staging check valve can be isolated by fully collapsing the shield and monitoring the leg pressures on the operating cycle after the support is reset against the mine roof and floor. The requirement is to have the bottom stage fully extended when the support is reset. In this configuration with a staging check valve that is functioning properly, the pressure in the bottom stage will not change significantly until the force in the top stage due to additional roof loading overcomes the setting force developed in the bottom stage (figure 25). An immediate increase in pressure in the bottom stage indicates that the check valve is leaking sufficiently to not allow the pressure in the top stage to be intensified.

Two factors that reduce hydraulic life expectancy are contamination and corrosion. It is very important to change filters as needed to maintain a clean emulsion. Debris in the leg cylinder is a leading cause of stage valves failures, which reduce shield capacity by 40% to 60%. Debris also reduces seal life and can cause check valves and yield valves to malfunction. Corrosion is also a primary factor in leg cylinder life. A chemical analysis of the emulsion fluid should be done periodically. Bacteria growth or poor water quality can significantly reduce the life of the leg cylinders.

From the design and repair perspective, there have been improvements in plating technology, which has reduced the corrosion problem in recent years. Early shield designs used chrome plating for the leg cylinder bores. Chrome is a very porous material, and even if the thickness is increased, the structure, which is analogous to placing layers of chicken wire on top of each other, remains quite porous. Bronze is a much better material, and most shield cylinders are now plated with

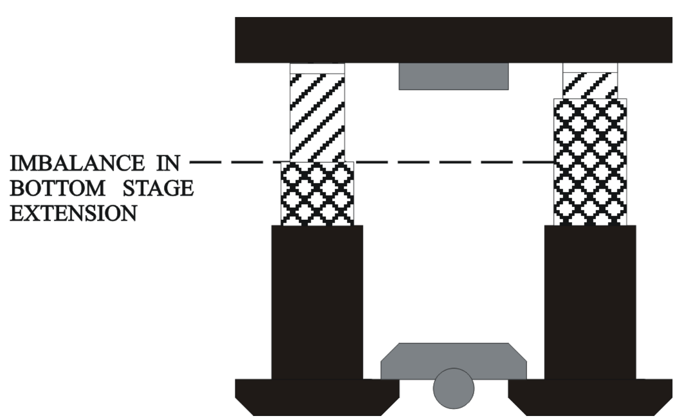


Figure 24.—Imbalance in leg cylinder staging due to internal leakage.

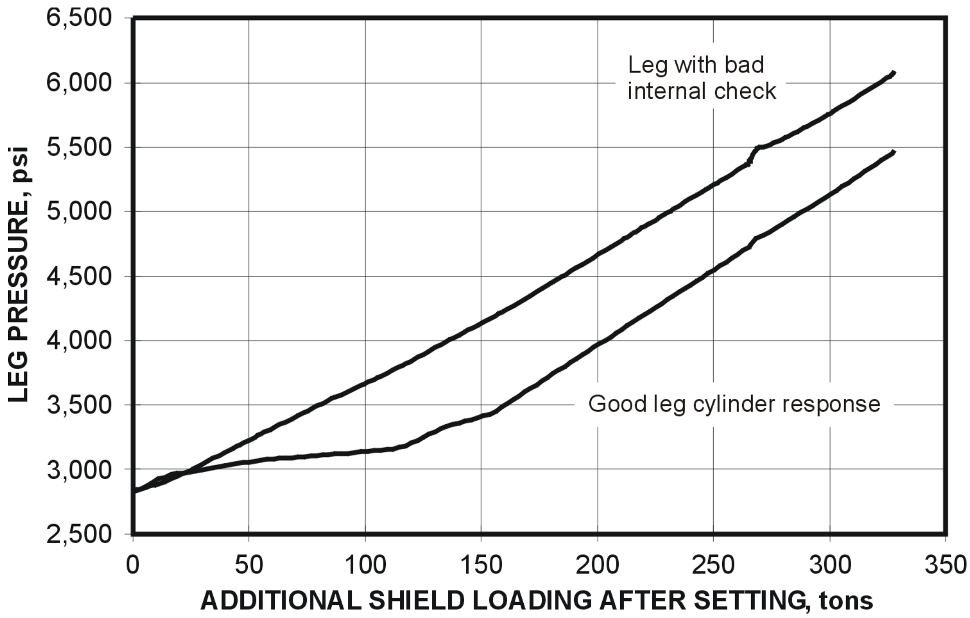


Figure 25.–Identification of defective leg cylinder staging valve.

bronze. The harder the bronze is, the better the life that can be expected. An alternative to bronze is to use chrome over top of nickel plating. The electrolysis of the nickel creates a very hard material that is highly corrosion-resistant, but the process is also considerably more expensive. Another problem with using nickel as the base metal in the plating process is that the two metals tend to react with one another, reducing the bond between them, which causes the chrome to peel off in time.

An impending problem with current plating technology are the environmental hazards associated with the plating process. Both chrome and bronze plating present environmental hazards that are likely to drive up the cost of these conventional plating methods in the near future. Since rebuilding of shield leg cylinders is a routine part of extended shield operation on every longwall, this can have a significant impact on the mining industry, and new plating technologies need to be explored. Swanson Plating, Morgantown, WV, is now offering laser-controlled plating as an alternative. The laser technology, in addition to being more environmentally friendly, reduces the heat affected zone by nearly two orders of magnitude and allows for much more controlled plating using materials that cannot be applied by conventional plating practices. More innovative solutions to consider would be the use of ceramic or composite materials that are much more corrosion-resistant than the heavy metals currently used in cylinder construction.

ERRORS IN ASSESSING SUPPORT LOADING

Support loading is determined from measurement of the pressure in the bottom stage of the hydraulic cylinder. Since the pressure increases in direct proportion to the increase in support loading, this provides an accurate assessment of the roof loading through the full loading cycle. However, when the bottom stage is fully stroked, a portion of the increase in roof loading after the support is set against the mine roof and floor will not be detected by changes in hydraulic pressure in the bottom stage. The reason for this period of undetected roof loading is that when the bottom stage is fully extended, the bottom-stage piston is being held against the mechanical stops with a force exerted by the pump pressure at the time the support was set. The pressure in the bottom stage will only increase when this piston is moved off of the stops and begins to compress the hydraulic fluid in the bottom stage. In order for this to happen, the pressure in the upper stage, which is at pump pressure when the support is set, must increase to cause a force in the upper stage that exceeds the setting force in the bottom stage. The additional roof loading that is required to produce this additional force in the upper stage is the roof loading that is undetected by the pressure gauges measuring hydraulic pressure in the bottom stage.

The mechanics of the pressure development in the various support stages can be described by examination of equilibrium requirements for each individual stage, realizing that each stage must carry the same load. The undetected roof loading with the bottom stage fully extended can be calculated using equation 4. An examination of these principles indicates that the period and magnitude of undetected roof loading will increase as the setting pressure increases for a particular support.

where URL = undetected roof load, tons;

$$\mathrm{URL} = \frac{(A_2 * P_2) - (A_1 * P_1)}{2000}, \quad (4)$$

A_1 = area of first (bottom) stage, in^2;
A_2 = area of second stage, in^2; and
P_1 = P_2 = setting (pump) pressure, psi.

Figure 26 depicts undetected roof loading for a 700-ton (two-stage) longwall shield set at full pump pressure. It is seen

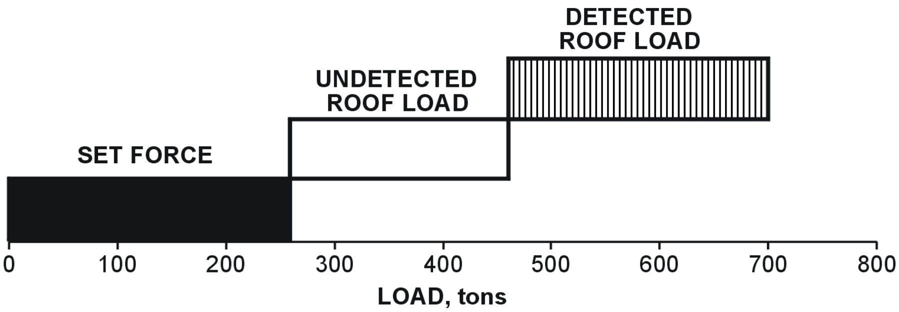


Figure 26.–Undetected roof loaded by bottom-stage pressure measurements when bottom stage is fully extended during setting of the shield.

from this figure that the unrecorded roof loading is quite large. For example, approximately 200 tons of undetected roof loading, which represents 84% of the remaining shield capacity, will occur on the 700-ton longwall after it is set against the mine roof and floor.

CONCLUSIONS AND RECOMMENDATIONS

The evolution of improved shield design continues with the current generation of shield supports. The previous generation of shield supports (1985-95 era) experienced several serious structural failures. These included canopy sections falling off, bases cracking in half, leg sockets pushing through the base and canopy into the mine roof and floor, lemniscate links breaking, and pins bending and breaking. Generally, these problems have been addressed by the support manufacturers. However, as the capacities continue to grow, each new generation of shield supports must also experience the growing pains of design deficiencies. This is occurring now as isolated failures of modern shield design are again cropping up.

While proper shield design is not magic, there is an experience factor that cannot be overlooked. Unlike designing a building or a bridge where the load conditions are well known and fail-safe design philosophies can be readily employed, the mining environment is not well defined or easily understood. Mining engineers typically design to allow failure, while conventional structural engineers design to prevent failure. Furthermore, as the demand for extended shield life continues to grow, continued improvements in quality control and innovations in shield design must be developed if this requirement is to be realized.

Although the design of longwall shields is a complex issue, it would be beneficial to prospective shield buyers to learn the basic engineering principles associated with shield design issues that are addressed in this paper. Although it is not necessary to know all of the answers, knowing what questions to ask is important. Hiring a structural engineering consultant is another option, but again one must be aware of the uniqueness of the mining environment and shield design issues.

Corrosion is by far the most overlooked factor in both shield design and performance testing. Corrosion control practices are limited, but they need to be followed. Simple things such as using corrosion-resistant paint and repainting of shields can be effective. Design changes need to be made to the pin joints and clevises, which are subjected to high wear rates, to avoid corrosion in these areas. Advanced plating technologies and more corrosive-resistant materials need to be employed to improve hydraulic cylinder life, which is by far the weak link in a shield's lifeline.

Support failures are site-specific, and the magnitude of the problem must be considered in making judgments of support safety. The information provided in this paper will help identify problems and suggest solutions. However, the final judgment must be made with considerations of the severity of the problem, number of supports affected, past performance history, face conditions at the time of failure, and the overall situation at the mine site. Finally, the best policy is always to correct problems as soon as they occur. Although this is often impractical and at times impossible, these generic recommendations regarding safety precautions of problem support systems are made with the realization that they are not universal for all circumstances:

1. When failures occur, procedures to reduce shield load should be implemented.

A good practice to reduce face weighting is to maximize the rate of advance. Idle faces generally have a tendency to create higher shield loads. Also, setting forces should be optimized. Conventional wisdom has been to increase the set pressures or to ensure that full setting (pump) pressure is maintained. However, in displacement-controlled loading where the load development is proportional to the shield stiffness and the support resistance is not fully controlling the ground movements, reducing the setting loads may also be an option to reduce the total loading on the support. Another possibility is to reduce the depth of the shearer cut. Much of the shield loading is generated from the shearer pass. A reduced cut may reduce shield loading, although there are no data to substantiate this hypothesis.

2. Ensure proper operation of the support once it is put into service.

Another issue related to lowering the design stress is to ensure proper operation of the support once it is put into service in the mine. Stress developments are greatly enhanced whenever the support is stood on the toe of the base. When a shield is set with the canopy tip up, the leg forces will typically rotate the canopy into full roof contact and in the process lift the rear of the base off of the ground. This base-on-toe configuration increases the magnitude of stress in the lemniscate links by as much as 300%. Modern two-leg shields employ a control system for the canopy capsule cylinder that prevents setting the shield with the canopy tip up. Thus, if this system is deactivated by the mine personnel, the life expectancy of the pin joints and the entire shield can be significantly shortened.

3. Recognize the cumulative effect of degraded performance and noncatastrophic failures.

As previously discussed, it is quite common for aging shields to experience hydraulic leaks that lead to degraded support capacity, but typically do not interfere significantly with the stability of the support or its capability to remain functional. Whenever the degraded support capacity is accompanied by a lack of ground control, obviously the problems must be corrected immediately. However, with modern high-capacity supports this is generally not the case, and the question is how soon the degraded leg cylinder performance should be corrected. In many cases, leakage will occur in only one leg or be considerably worse in one leg compared to the other. From a load distribution viewpoint, loads will transfer down the side of the structure with the active leg. This imbalance in leg forces will cause some increase in component stresses, particularly in the caving shield and lemniscate assembly. Although this generally will not pose an immediate threat, the cumulative effect of the increased loading, if left unattended, will more than likely decrease the shield life by accelerating fatigue-related failures. Furthermore, if structural failures do occur, the asymmetric loading caused by imbalanced leg forces will likely make these failures more severe.

This same logic applies to structural failures. There is considerable redundancy built into a shield support in the sense that there are multiple load paths to transfer roof loading through the support structure and into the mine floor. Most failures when they first occur will not immediately affect the performance of the support. However, once a failure initiates, a domino effect will likely occur wherein additional loading is transferred elsewhere, and the probability for the failure to grow or spread to other components as time progresses is also likely.

4. While fatigue failures are difficult to judge, they can lead to catastrophic failure with little or no warning.

Modern shields of high-strength steel are more susceptible to unstable failure than previous generations of shields constructed from mild steel. Crack formation in any part of the support structure should be viewed as a sign of potentially imminent danger. The crack indicates that the steel has failed. Whether this crack will propagate to cause destruction of a support component depends on many things, most notably, the ability of the member to effectively redistribute loading primarily within that component or, to a lesser degree, to other components. This makes judgments of support safety in these situations difficult. In any event, cracks in a support structure should be closely monitored, and attention should be given to any increase in the growth and particularly the rate of growth of the crack. Cracks that arrest themselves after formation can often be ignored, particularly when they are in sections of the support that do not carry the bulk of the loading. In this case, load was adequately transferred elsewhere to alleviate the localized stress condition that initiated the crack. Conversely, a crack that continues to grow after formation is likely to develop into a critical situation where the safe performance of the support is threatened. Obviously, as soon as the structural integrity of the support (component) is threatened, the support should be taken out of service and modifications made. Once a critical crack length is reached, the growth rate can accelerate in relatively few additional loading cycles. Particular attention should be paid to weldments, since localized stress developments are likely to be higher in these areas and the potential for rapid crack growth is enhanced.

5. Recognize failures that can lead to instability and correct them immediately.

It is unlikely that the support will show any signs of diminished load-carrying capability while structural problems are developing, but the stability of the support may be threatened. Failures to the caving shield or lemniscate link pins or to the links themselves are most serious since they control the horizontal stability of the shield. Failure of any set of link pins or links will cause the support to collapse under its own weight from instability. There is redundancy built into the shield in that there are pairs (left and right side) of both the upper and lower links. Failure of one side may not result in immediate instability, but will significantly increase loading in the adjacent link and therefore should be viewed as very serious, with immediate action taken to remove the failed pin and install a new one.

Another serious failure that can lead to instability is failure of the leg sockets. The leg sockets must transfer loading from the canopy to the base. If the sockets break lose, the cylinder will eventually punch through the bottom plate on the base and render the support inoperable and perhaps unstable.

6. Understand the hazards of automated control systems.

During the early development of the electrohydraulic control systems with automated and shearer-initiated advance, several injuries occurred because of unplanned or unexpected shield movements. Although failures in the electronics have largely been eliminated in recent years, occasional injuries still occur where a miner is unaware of the pending shield move, which was initiated at a remote position by another miner. Close attention should be paid to the audible warning of the shield advance. A good practice is to walk on the bases of the support whenever possible to avoid being pinched by an unexpected shield move.

REFERENCES

American Institute of Steel Construction [1980]. Manual of steel construction. 8th ed. Chicago, IL: American Institute of Steel Construction, pp. 5-89.

Barczak TM, Gearhart DF [1998]. Performance and safety considerations of hydraulic support systems. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: University of West Virginia, pp. 176-186.

Barczak TM, Chekan GJ, Babich DR [1998]. Shield design, construction, and operation. In: Proceedings of Longwall USA International Exhibition & Conference, pp. 5-47.

Barsom JM, Rolfe ST [1987]. Fracture and fatigue control in structures. Englewood Cliffs, NJ: Prentice-Hall Inc.

Fiscor S [1999]. U.S. longwall census '99. Coal Age *Feb*:30-35.

FACTORS TO CONSIDER WHEN PURCHASING A NEW SET OF LONGWALL SHIELDS

By Thomas M. Barczak[1]

ABSTRACT

Purchasing a new set of longwall shields requires a substantial investment. A poor shield design can lead to economic hardships, safety concerns for the mine workers, and closure of the mine. This paper addresses several key points that should be considered in the procurement process: (1) understanding your goals and the logic in selecting a higher capacity shield, (2) the importance of completing performance testing before production shield fabrication begins, (3) making sure performance testing is properly done, (4) measuring load (stress) development during performance testing, (5) testing a shield to failure, and (6) the value of buying extra shields. In addition, several challenges are proposed for consideration in future shield designs. These include (1) 100,000 life-cycle expectancy, (2) improved hydraulic diagnostic capability, (3) smart load control by optimizing setting pressures, (4) lubricated link joint design concepts, (5) composite material applications to reduce shield weight, (6) incorporating periodic weighting predictive algorithms into the data-processing software, (7) advanced component load measurement on two to three specially instrumented shields to detect loading in the structural components, and (8) constant set leg cylinder design.

[1]Research physicist, Pittsburgh Research Laboratory, National Institute for Occupational Safety and Health, Pittsburgh, PA.

INTRODUCTION

A new set of longwall shields can cost over $20 million. This represents a major investment for any mine operator. Currently, these new shields are expected to last for more than 60,000 loading cycles, representing 18-20 panels of mining [Barczak 1999]. A poor design or even a marginal design that reduces the operating life of the shield can be the difference between a mine staying in business or closing. In addition to the economics pertaining to the initial shield investment, time and money spent in making repairs to damaged shields can be very costly.

There are also safety issues when failures occur. Poor shield performance can contribute to instability in the overlying rock mass leading to roof falls in the immediate face area, cavity formation above the shields, or even "iron-bound" shields when excessive yielding occurs. The well-planned removal of shields in properly designed recovery rooms is difficult enough. There are hazards associated with increased exposure of the mine workers to unsupported ground and with moving very heavy equipment in a confined work environment. Unplanned removal of shields due to unexpected failures that require complete removal of the shields or changing out of shield components before the completion of the panel are that much more difficult and hazardous. The safety issues and ground control issues are interrelated. Poor shield performance can lead to ground control problems, and stoppage in face advancement to remove and/or change out shield components can cause the ground conditions to worsen and create a "snowball" effect.

Thus, proper shield design is essential to any longwall operator. Engineering issues pertaining to shield design are addressed by the author in another paper in these proceedings [Barczak 2000a]. This paper provides several key points pertaining to the procurement process, including the reasons for buying a higher capacity shield, performance testing strategies to ensure that the shield design is adequately evaluated, and the benefits of buying extra shields and implementing a well-planned preventive maintenance program once the shields are purchased. In addition, traditional strategies of testing a shield through a limited number of cycles are challenged with recommendations to test a shield to failure. The paper concludes with proposed ideas and challenges for future shield designs.

EVALUATE YOUR CURRENT DESIGN FIRST

Before new shields are purchased, your current shield design should be evaluated. Loading histories, if kept, can provide valuable information to properly size your next support system. Questions to answer include: How often were the supports at yield pressure and when did this occur? Were there any ground control problems, and what were they? Were the shields rebuilt? Were any structural modifications made to the shields? To what degree were the pin bores worn, and when did the wear occur? What was the reliability of the leg cylinders? How often did leakages occur? How often were the cylinders rebuilt? How reliable was the control system for operating the shields? Were there any warranty issues?

Performance testing of an aging shield can provide insight into how much life expectancy is left in your current shields, as well as reveal valuable information about making improvements in the design of the new shields when they are purchased. The National Institute for Occupational Safety and Health (NIOSH) has developed a series of tests using the unique Mine Roof Simulator load frame (figure 1) for conducting safety performance testing of aging longwall shields. These procedures are described by the author in another paper in these proceedings [Barczak 2000b]. The Mine Roof Simulator is an active load frame that can apply both vertical and horizontal loading to a shield and accurately simulate the in-service loading conditions.[2]

[2]Further information on using the Mine Roof Simulator for shield testing may be obtained by contacting the author at (412) 386-6557 or by e-mail at: thb0@cdc.gov

Figure 1.—The Mine Roof Simulator at NIOSH's Pittsburgh Research Laboratory.

KEY POINTS TO CONSIDER WHEN BUYING NEW SHIELDS

1. Identify your goals in buying a bigger shield.

Shield manufacturers now provide two-leg shields with capacities approaching 1,200 tons, nearly four times the capacity of the first shields installed in the United States in 1975 [Fiscor 1999]. Likewise, shield life expectancies are now 60,000 to 70,000 cycles, more than double what it was just a decade ago. Although there is a natural tendency to buy the biggest shield available, you should really consider your goals when making this decision. Are you looking for more support capacity to improve ground control, or is your primary goal to extend the life of the shield, or do you wish to achieve both goals?

If your primary goal is to extend shield life and your previous support of lower capacity was providing adequate ground control, then consideration should be given to derating the higher capacity shield if you choose to purchase one. Derating can be accomplished in two ways: (1) reducing the setting pressure and (2) lowering the yield pressure. The goal is to reduce the total loading on the support as a percentage of the yield load rating of the support. The total shield load will be

the sum of the setting load and load developed in response to the convergence. This total load will be limited by the yield pressure of the leg cylinders. The load developed in response to the convergence will be controlled primarily by the stiffness of the leg cylinders. Since the stiffness increases with increasing leg cylinder diameter, a higher capacity shield will develop more load than a lower capacity shield for the same convergence. Thus, unless the increased setting force results in decreased face convergence, the increased setting force will unnecessarily increase the overall shield loading.

Derating a shield by reducing the operating yield pressure will control the total loading and associated component stress development. This will provide an additional margin of safety and extend the life of shields where high component stresses are observed. Whether yielding is detrimental or not probably depends on the rock mechanics and source of the shield loading. If the convergence is caused by main roof weighting that is irresistible in terms of the available shield capacity, then promoting yielding at a slightly lower yield pressure is not likely to have any significant impact on roof stability.

If you need the extra capacity for ground control *and* you want to improve the life expectancy, then be prepared to pay a premium for the shield and insist on a conservative design. A conservative design can be qualified in terms of the component stress developments, recognizing that the chance for failure increases as the component stresses increase and that fatigue and corrosion failures can occur for nominal stress developments well below the yield strength of the steel. Quantifying stress limitations can be difficult since there are several factors involved. However, a reasonable rule of thumb is that no component should be stressed beyond 80% of the yield strength of the steel under *any* in-service load condition, and stresses should be below 60% of the yield strength for *typical* load conditions. Classical structural design often requires that stresses be kept below 50% of the yield strength to prevent the occurrence of fatigue-related failures [Barczak 1999].

In summary, there is a common misconception that higher capacity shields will be loaded to a lesser degree than lower capacity shields simply because they have a higher support capacity. Since the setting pressures have remained constant and the stiffness increases as the capacity of the shield increases because of the larger leg cylinder diameter, higher capacity shields are just as likely to be fully loaded as lower capacity shields under the current operating practices.

2. Make sure that performance testing is completed before production shield fabrication begins.

A prototype shield should be fabricated and thoroughly performance tested *before* fabrication of the production shields begins. A shield consists of five major components (canopy, base, caving shield, lemniscate links, and leg cylinders) all connected together. Each component must be properly designed to effectively transfer roof loading through the support structure. If one component is underdesigned, additional load may be transferred to other shield components, potentially creating a domino effect that will likely reduce the life expectancy of the support. Also, once a fabrication is made, modifying the existing fabrication to correct the fundamental design deficiency will generally not be possible or at best be less effective than redesigning the component completely. Additionally, modifying an existing fabrication can be difficult when complex geometries and high-strength steels are involved. Furthermore, the weight of a modern-day shield needs to be minimized. Correcting a design deficiency by adding a reinforcement plate to a fabrication generally will not result in an optimum strength-to-weight ratio for that component. Once the production shields are fabricated, any change in design can be costly, and the tendency will be to minimize the cost by avoiding the modification if possible or implementing the lowest cost modification, which may allow the support to pass the performance testing, but will not provide the operator with the best design possible.

3. Make sure that performance testing is properly done.

Performance testing must properly simulate the in-service load conditions. NIOSH has developed a set of safety performance testing protocols that take advantage of an active load frame called the Mine Roof Simulator. This biaxial load frame can apply both vertical (compression) and horizontal (racking) loads simultaneously to simulate the ground movements and shield interaction. These test procedures are described in detail by the author in another paper [Barczak 2000b] and provide the most direct simulation of the in-service loading conditions [Barczak 1999]. In comparison, tests conducted in a static load frame rely on external pressurization of the leg cylinder to generate support loading. The limitation imposed by the static load frame is the inability to directly simulate the forward translation of the canopy due to slippage of the support on the mine roof or floor and the face-to-waste racking of the shield by the strata caving into the gob. These conditions can increase the load development in the caving shield and lemniscate links by several hundred percent.

Slippage of the canopy on the mine roof due to loss of frictional resistance at the roof interface can be simulated in static and uniaxial active load frames by placing rollers on the canopy to create a "frictionless canopy test." Unfortunately, the zero-friction test with rollers on the canopy is not very practical for extensive cyclic loading since the rollers may require frequent adjustments due to frictional effects. Thus, this test is typically not included in the manufacturer's performance testing protocol. If the frictionless canopy test cannot be conducted, then the support should be configured so that it is standing on the toe of the base; this will require horizontal constraints at the toe of the base and rear of the canopy (at the caving shield connection). The horizontal force couple generated by this configuration allows load transfer to the caving shield-lemniscate assembly, similar (although not as severe) to that obtained with the zero-friction test.

Likewise, face-to-waste racking of the shield cannot be directly simulated by a static or uniaxial active load frame. The contact configuration that most closely duplicates the caving shield-lemniscate assembly response for this condition in static or uniaxial load frames is to configure the support so that it is simply supported on the rear of the base. Face-to-waste racking of the shield canopy causes a reversal in the state of stress in the lemniscate links from compression to tension.

Because reversal of the loading accelerates fatigue failures, both the base-on-toe *and* the base-on-rear configuration should be incorporated into safety testing protocols conducted in static or uniaxial load frames.

Longwall operators in cooperation with the shield manufacturers have developed and refined performance testing procedures for longwall shields using static load frames. Consolidation Coal Co. in particular has led this effort. The so-called "Consol Test" consists of several combinations of canopy and base contact configurations that are categorized as follows:

(1) Offset yield loading;
(2) Base toe loading;
(3) Three-point canopy torsional loading;
(4) Two-point canopy torsional loading;
(5) Side shield loading and base torsional loading;
(6) Diagonal base contact;
(7) Three-point base contact torsional loading;
(8) Symmetric base edge loading;
(9) Asymmetric base edge loading;
(10) Leg socket loading; and
(11) Canopy dishing.

These tests are designed to be used in static- or single-axis load frames. These test configurations have proven to be among the most effective protocols for performance testing of shield supports in a static frame. The operator should consult the support manufacturer and discuss these or equivalent tests to be used for the performance testing program.

4. Measure the load development as part of the performance testing.

It is not enough to simply measure the leg pressures during performance testing. Some measurement of load transfer through all the support components should be conducted as part of the test program to verify the effectiveness of the test procedures and to evaluate the stress developments within the shield. This can be done with a few strain gauges to measure nominal load (stress) development in each component. Photoelastic plastic can be used to isolate areas of expected stress concentration.

5. Consider testing a shield to failure.

Most companies conduct performance testing only through a limited number of cycles, generally equal to the warranty period provided by the shield manufacturer, which is typically 50% to 60% of the life expectancy of the shield. This practice is adequate to discover fundamental flaws in the shield design. However, for a well-designed shield, this approach will not evaluate fatigue failures that occur near the end of the shield life. In addition to providing a more definitive estimate of the life expectancy of the shield, testing a shield to failure will provide insight into the nature and severity of the fatigue-related problems that will eventually occur. It will also provide the mine operator with insight as to when to look for these problems because they are often difficult to see when they first develop, but can be catastrophic if left unattended. In addition, by testing the shield to failure, the impact of the failure(s) on the shield structural integrity and performance of the shield can be determined beforehand.

In summary, by testing the shield to failure, the mine operator will be in a much better position to (1) plan for the next shield procurement cycle, (2) develop a monitoring plan for inspecting the shields for failures, and (3) develop a strategy for repair and modification of the shields when failures do occur. The manufacturers, and in turn the mine operators, will also benefit from failure testing by providing critical information on the limitations of current shield design that can lead to design improvements in the future.

6. Consider buying several extra shields.

There is clear evidence that mines that purchase additional shields and institute good preventive maintenance program will maximize the useful life of their longwall shields. A good rule of thumb is to purchase 10% more shields than is necessary for the face. For example, a 1,000-ft face would require 174 shields (1.75 m wide). If 17 extra shields were purchased so that 17 shields could be removed from operation during each longwall move, the entire face would be recycled in 10 panels of mining. Mining a 10,000-ft-long panel will require about 3,300 shield cycles; thus, each shield will be changed out at least once during the shield's warranty period. More importantly, once the warranty period is over and the shields have been in operation for 35,000-40,000 cycles, a detailed inspection and rebuild program can be instituted. Problems such as leg cylinder rebuild and restoring pin tolerances to design specifications could be corrected as they arise, and preventive measures, such as painting and crack inspection and repair, could be done on a regular basis. Although this adds cost to the initial purchase, a good preventive maintenance program will pay big dividends in maximizing the life of a shield.

CONTINUE PUSHING THE ENVELOPE

Improvements in shield design have been made largely through mine operators pushing the envelope in terms of expectations. Ten years ago, a life expectancy of 60,000 loading cycles was unheard of, yet today it is the standard. Likewise, 1,200 tons of support in a two-leg shield was unthinkable 20 years ago, yet shield capacities continue to increase as each new generation of supports is developed. Although there certainly are limitations in any engineering design and factors such as the weight of the shield will soon pose a barrier to increased size with current technologies, one must remember that the unthinkable 20 years ago is now the standard. Additional improvements in shield design will be made as long as mine operators continue to push the envelope in shield design.

Below are some items to consider for setting goals for the next generation of shield supports.

1. 100,000 Life Cycle Expectancy.—The increases in life expectancy realized during the past 10-20 years should continue in the near future. It is not inconceivable for a shield to survive 100,000 loading cycles. To reach this goal, some improvements in design may be necessary or at least closer attention will need to be paid to fundamental design practices. In particular, components will need to be sized such that stress levels are kept at moderate levels with respect to the yield strength of the steel. Stress concentrations due to sharp changes in geometry must be avoided, and weldments must be of high quality. More corrosion-resistant materials may be used in certain areas to reduce stress corrosion. Make sure that the link bores have sufficient bearing areas to avoid high levels of stress, and allow for the additional stresses caused by corrosion in the design of the link bores.

2. Hydraulic Diagnostic Capability.—All shields will experience leakage in the leg cylinders or other hydraulic components that degrade the support capability. An algorithm should be specifically written into the control computer that monitors leg pressure histories that will provide the longwall coordinator with a record of bad leg cylinders that need to be repaired. Another option is to have the computer reset leg cylinders that are leaking below a designated setting pressure. While this capability exists on some modern faces, it should become a standard part of the operating system.

3. Smart Load Control.—The benefits of derating a shield by reducing the total loading on the shield were discussed earlier in this paper. A challenge for future shield designs would be for the control computer to optimize the set pressure by monitoring load development history and adjusting the set pressure to minimize overall shield loading once a ground reaction curve for the longwall face is established. The set pressure could be adjusted as the conditions change to continually optimize the support capacity utilization.

4. Lubricated Link Joints.—Pin joints are necessary to accommodate the kinematics of a shield, but these joints are by far the leading cause of structural rebuild. Efforts should continue to enhance the use of wear-resistant materials, such as impregnating the pins with zinc phosphate, but consideration should also be given to a lubricated joint to reduce the wear.

5. Composite Material Design.—Even with the use of high-strength steels, modern shields weigh as much as 30 tons. Weight is a major barrier in increasing the shield widths beyond 2 m with the current steel constructions. Composite materials of equivalent strength weigh only one-fourth that of steel fabrications. Although several engineering and economic issues need to be explored before the feasibility of using high-technology composite materials for shield construction can be determined, this could lead to a new generation of shield supports in the future.

6. Forecasting Periodic Weighting and Heavy Roof Loading.—Most modern shield systems are capable of capturing shield loading histories to the point where we are overloaded with data. These data, if properly managed, can be useful in evaluating leg cylinder leakage and optimizing setting pressures, as suggested in items 2 and 3 above. Another area of value is the prediction of periodic weighting intervals. There has been some research in this area already. For example, NSA Engineering, Inc., developed a shield monitoring program that couples with its GeoGuard™ software [Sanford et al. 1999]. The system was recently tested in an Australian mine where severe face weighting was observed, resulting in hazardous face conditions and weeks of lost production.

7. Advanced Component Load Measurement.—Currently, only the leg cylinder pressure is monitored. No loading information is obtained on any other components. It would not be difficult in the design process to include strain gauges on selected components that could be monitored by the computerized data acquisition system. This need not be done on every production shield; in fact, two or three shields could serve as instrumented shields and provide valuable information on the actual load conditions observed underground. This information could then be used to refine performance testing, as well as help to identify load conditions that actually contribute or cause structural failures. Even measurement of the link loading alone would provide valuable information that is currently not available.

8. Constant Set Leg Cylinder Design.—Current shields use a two-stage cylinder design where the first stage extends and retracts first. This operation typically causes the first stage to be near full extension most of the time. In addition, seal leakage, which occurs on almost all aging shields, causes the bottom stage to extend outward and further promotes full extension of the bottom stage. Since the setting force and the subsequent capacity of the shield to resist roof loading is reduced in proportion to area differential between the top and bottom stage when the bottom stage is fully extended, this is not a good design to ensure that the full capacity of the shield is used. In other words, why buy a 1,000-ton shield when its performance is degraded to that of a 500-ton shield most of the time? There are alternative designs available, most notably that employed by Fazos, Inc., which do not use the conventional two-stage design used in most modern shields. The Fazos design uses a central core to eliminate the use of independently acting stages. Although this design works, it has not been adopted by the major shield manufacturers. An alternative approach would be to configure the standard two-stage design with a system that would cause the top stage to extend first. This could be done by using a nominal hydraulic pressure in the retract annulus of the bottom stage during the extend operation. For current shields, a mechanical device (strap or chain holster) could be built that would attach to the cylinder casing and the base that would prevent the bottom stage from extending. This would not be used on every mining cycle, but would be done periodically as part of a preventive maintenance program to restore full setting capability to the leg cylinders, which are routinely at full bottom-stage extension.

CONCLUSIONS

Improvement in shield design continues to be made with greater life expectancies than ever before. This can certainly benefit mine operators, but the increased cost of modern shields also places greater emphasis on ensuring that the shield design is good since the consequences a poor design can be catastrophic from both an economic and a mining perspective.

Proper planning is essential for a new shield procurement. Sufficient time must be built into the procurement process to ensure that a prototype shield is adequately tested before fabrication of the production shields begins. Otherwise, design changes may not be able to be made in time and/or the modifications will not be as effective as changing the original design. Performance testing needs to be done properly, and every effort should be made to simulate the in-service conditions that occur in the mine. Testing a shield to failure, even if it incurs additional costs, will give insight into when and what to expect when fatigue-type failures plague the support as it ages and needs to be rebuilt or nears the end of its useful life. Purchasing additional shields and implementing a preventive maintenance plan that allows for scheduled rebuild of hydraulic cylinders and inspection and repair of structural components at the onset of problems will pay big dividends in extending the life of the shields.

One misconception is that modern shields by virtue of being higher capacity will last longer than lower capacity shields of previous generations. Modern-day shields are just as likely as lower capacity shields of previous generations to be fully loaded. In fact, the higher strength steels used in modern-day shields generally are prone to more brittle and catastrophic failures when excessive loading occurs than previous generation shields. This fact, coupled with the higher cost and longer life expectancy, makes it more important than ever that shields be properly designed. Thus, it is important to determine the design quality and limitations of the shield during the procurement process.

Improvements in shield design will continue as long as operators continue to demand improvements to which there are engineering solutions. A life expectancy of 100,000 cycles is a reasonable goal that can be accomplished with current technology. Modern longwall faces all have computers collecting data and providing automation of the support operation. The next logical step is to use the data to further improve the diagnostic capabilities and optimize the shield performance. This will be a challenge to shield operators and will require some experimentation on operating longwall faces, but again is within the realm of current technology. The next major improvement in shield design may be the use of composite materials to lighten the weight of the shield. Reducing weight could break the 2-m width barrier that limits current designs using high-strength steel fabrications.

REFERENCES

Barczak TM [1999]. Modern shield technology: better than ever, but still not perfect. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 149-163.

Barczak TM [2000a]. Examining longwall shield failures from an engineering design and operational perspective. In: New Technology for Coal Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Barczak TM [2000b]. NIOSH safety performance testing protocols for standing roof supports and longwall shields. In: New Technology for Coal

Mine Roof Support. Pittsburgh, PA: U.S. Department of Health and Human Services, Public Health Service, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health, DHHS (NIOSH) Publication No. 2000-151, IC 9453.

Fiscor S [1999]. U.S. longwall census '99. Coal Age *Feb*:30-35.

Sanford J, Mahoney S, Conover DP, DeMarco MJ [1999]. Shield monitoring to forecast severe face weightings at the South Bulga Colliery, NSW, Australia. In: Peng SS, Mark C, eds. Proceedings of the 18th International Conference on Ground Control in Mining. Morgantown, WV: University of West Virginia, pp. 164-176.

INSTRUMENTS FOR MONITORING STABILITY OF UNDERGROUND OPENINGS

By Mark K. Larson,[1] Douglas R. Tesarik,[2] J. Brad Seymour,[1] and Richard L. Rains[3]

ABSTRACT

For several years, researchers from the Spokane Research Laboratory of the National Institute for Occupational Safety and Health (NIOSH) have been using geotechnical instruments in underground mines to study ground control problems and develop means of reducing accidents and fatalities caused by ground falls. This paper describes many of the different types of instruments, sensors, and data acquisition equipment that have been used for these studies; briefly explains the advantages and disadvantages of various sensor technologies; provides practical recommendations regarding the use of specific instruments and data acquisition systems; and outlines a general approach to the design and implementation of a successful instrumentation plan.

A wide variety of instruments are commercially available for measuring deformation, strain, stress, and/or load. If used correctly, these instruments can provide important quantitative information regarding the mining-induced behavior of the host rock, the performance of ground support systems, and the safety and stability of underground workings. Data collected from these instruments can warn mine staff of impending ground control failures or hazardous working conditions, as well as provide valuable information for the design of ground support systems and the configuration and sequencing of mining excavations. We hope that the practical information presented in this paper will encourage a more widespread use of geotechnical instruments in underground mines and lead to reliable measurements that aid the engineering decisions affecting the safety of miners.

[1]Mining engineer.
[2]Mechanical engineer.
[3]Geologist.
Spokane Research Laboratory, National Institute for Occupational Safety and Health, Spokane, WA.

INTRODUCTION

Researchers at the Spokane Research Laboratory (SRL) of the National Institute for Occupational Safety and Health (NIOSH) have been placing instruments in mines for several years to study means of reducing accidents and fatalities caused by ground falls. This paper describes ways of using these instruments. The goal of this paper is to share information and tips that encourage mining operators to use instruments to aid in decisions affecting the safety of miners.

In today's world of mining, layout and roof support are designed so that pillar and entry safety factors are only a little above the critical point of failure. Under varying conditions, the amount and type of roof support may greatly affect stability. It is therefore desirable to optimize roof support, i.e., the density and/or type should be sufficient to minimize the risk of ground fall, yet minimize costs. Technical personnel must determine this point using good engineering judgment.

Numerical models can be valuable tools in providing answers to "what if" questions, but they may provide poor information if erroneous assumptions or poor input values are used. For example, a homogeneous continuum model may not be sufficient if joints play a significant role in roof falls. Verification and tuning of models is a step that is quickly forgotten in the rush for an answer.

Instruments have been used increasingly in mines to measure deformation, stress, strain, and load. Such measurements serve two purposes. First, they provide quantitative information that can be useful in determining the mechanics of stability and in aiding engineering decisions. Second, they can be useful in verifying and tuning numerical models.

Bauer [1985] briefly described many types of available ground control instruments. Hanna [1985] comprehensively described instruments and measurement theory as applied to foundation engineering and construction. Dunnicliff [1988] updated information on instruments and how to plan monitoring programs. While new instruments have been designed and improvements made in older ones, it is not the purpose of this paper to give a comprehensive update. Rather, the purpose of this paper is to outline an approach to instrumentation that we have used for several years, describe the basic types of sensor technology, describe our experience with various instruments, list manufacturers of these instruments known to the authors, and provide pointers on continuous monitoring with a data acquisition system.

INSTRUMENTATION PLANNING

Design and implementation of a successful instrumentation plan requires careful consideration of the following factors:

- A fundamental understanding of the overall rock mechanics, geology, and general site conditions;
- Identification of the critical mechanisms or geotechnical parameters that need to be measured;
- Clearly defined objectives for each of the measurements to be taken;
- Estimates of the magnitude or anticipated range for each of these measurements;
- Selection of instruments, transducers, and data acquisition equipment appropriate for the desired measurements and site conditions;
- Availability of experienced personnel to install, troubleshoot, and monitor the instruments; and
- Established procedures for data collection, analysis, and reporting.

In general, instruments should not be installed until appropriate provisions have been made for collecting, analyzing, and reporting the instrument data.

Programmable data acquisition systems with multiplexers provide the user with the opportunity to scan and record measurements frequently from a large number of instruments. To ensure that the maximum benefit is derived from instrumentation programs, especially those capable of collecting enormous amounts of data, careful planning is necessary. Entire chapters in geotechnical instrumentation books are devoted to the subject of planning instrumentation programs and interpreting recorded data [Dunnicliff 1988; Hanna 1985]. Similarly, the number of papers presented at instrumentation conferences that address planning issues is sometimes large enough to comprise a technical session [Leung et al. 1999].

A successful instrumentation program starts by defining the purpose of installing the instruments and ends with implementing the decisions derived from interpretations of the data. Dunnicliff [1988, 1999] offers a 21-step checklist in which each step in a successful instrumentation program is described. Because comprehensive information about instrumentation planning is available in geotechnical literature, the following discussion focuses specifically on experience gained from installing instruments and data acquisition systems in underground hard-rock and coal mines, maintaining these systems, reducing the collected data, and interpreting the data for immediate and future use.

Because one of the goals of an instrumentation program is to monitor ground stability and take corrective action for any imminent problem, instruments that sense displacement and stress changes are commonly used. Instruments should be checked to ensure that they have not been damaged during shipping and handling. In general, displacement transducers should be recalibrated after they have been received, especially if the sensor is a potentiometer or a Wheatstone bridge circuit and if cable is added between the sensor and monitoring device. This action is helpful because the new calibrations will account for resistance caused by the cable. For vibrating-wire transducers, factory calibrations are generally sufficient because calibration constants are not changed by adding cable.

However, operation of the transducers should be verified before they are used underground. It is useful to have calibration sheets for displacement transducers at installation sites to assist in positioning transducers in the desired initial positions.

It is important to place the instruments at locations where the expected parameter changes can be measured accurately. For tabular ore bodies, estimates of change in values, such as stress or strain in pillars, abutments, and backfill, can often be calculated using the tributary load method. More complicated mine geometries may require numerical models to identify areas in the mine that will undergo high stress changes. For extremely critical areas near shafts or shops, it may be necessary to calculate changes in stress and displacement at specific points using a numerical model before installing instruments. This information can then be used to select the appropriate instrument range. If a two-dimensional numerical model will be used with instrument data collected during mining to evaluate mine stability, it is important to place an adequate number of instruments along the modeled cross section to provide calibration points for the model.

Installing two instruments that measure different parameters at the same location increases the probability that collected data will be useful. For example, pillar stability could be assessed with a stress-change measuring device such as a biaxial stressmeter and a displacement measuring instrument such as a horizontal borehole extensometer [Seymour et al. 1998]. Stress changes in the pillar can be verified by a borehole extensometer, which measures pillar dilation. Similarly, stress changes in backfill could be measured by embedment strain gauges and cross-checked with earth pressure cells [Tesarik et al. 1995]. Borehole extensometers in the mine roof may help analyze loading mechanisms [Tesarik et al. 1999]. Clustering instruments in this manner was useful in numerous instrumentation programs and served to verify measurements and aid in data interpretation. Clustering also provided backup data when one of the instruments or its cable was destroyed or malfunctioned.

SENSORS AND MONITORING

The authors have primarily used the following sensor technologies for instruments installed in underground mines: strain gauges configured in Wheatstone bridge circuits, potentiometers, and vibrating-wire transducers. These sensors can be read either manually with a portable electronic readout box or at programmed intervals using a computerized data acquisition system set up in a central location near the instruments. Manual data collection methods are typically used in situations where only a limited number of instruments are monitored; individual instruments are installed in remote, isolated locations; it is impractical to route the instrument cables to a central location; personnel experienced with the use of dataloggers are not available; or the additional expense of a data acquisition system is prohibitive. Although manual readings can provide reliable information about the stability of underground openings [Tesarik et al. 1991, 1999], these readings are usually taken too infrequently to allow a meaningful analysis of measurement quality and are subject to human error. If possible, instruments should be monitored with a data acquisition system, and manual readings should be taken periodically with an electronic readout box to verify the datalogger measurements. Frequent readings not only permit better judgment about the quality of the measurements, but also help associate trends in the instrument data with discrete mining events. The use of dataloggers and their ancillary components is described below in the "Data Acquisition System Basics" section, along with several recommendations for improving the likelihood of quality measurements.

SENSOR TECHNOLOGIES

Wheatstone Bridge

A Wheatstone bridge is a circuit (figure 1) that provides a signal output directly related to strain. If R_1, R_2, R_3, and R_4 are the respective resistances of arms 1 through 4 in figure 1, and $R_1R_3 = R_2R_4$, then the differential output voltage, ΔE, may be represented by

$$\Delta E = V_x \frac{R_1R_2}{(R_1+R_2)^2}\left(\frac{\Delta R_1}{R_1} - \frac{\Delta R_2}{R_2} + \frac{\Delta R_3}{R_3} - \frac{\Delta R_4}{R_4}\right) \quad (1)$$

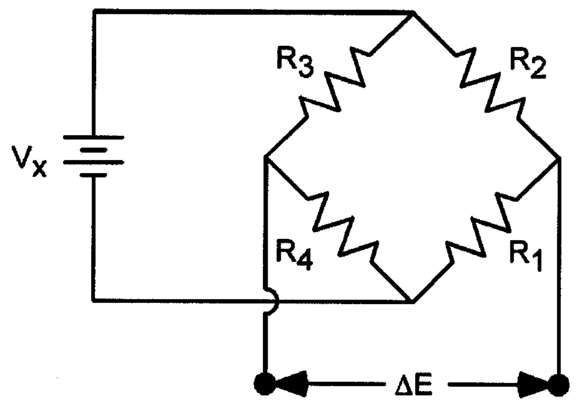


Figure 1.—Wheatstone bridge wiring diagram.

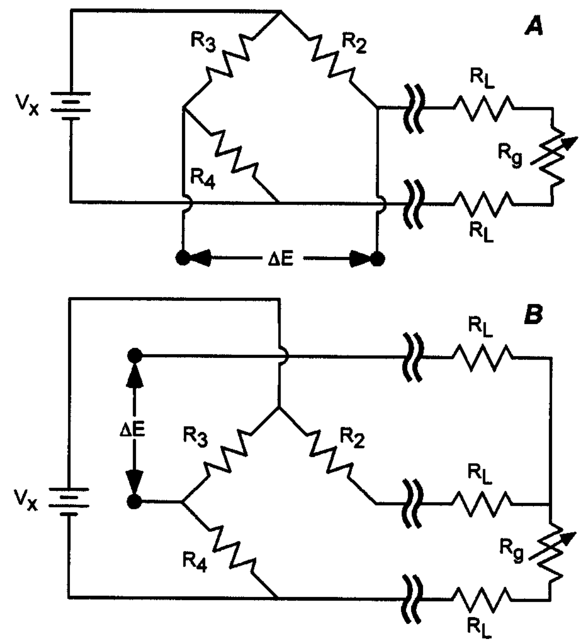


Figure 2.—Quarter-bridge wiring diagram. *A*, Two-wire leads; *B*, three-wire leads.

if second-order terms are neglected [Dally and Riley 1991]. In equation 1, V_x is the applied excitation voltage and ΔR_1, ΔR_2, ΔR_3, and ΔR_4 are the changes in resistance of strain gauges 1, 2, 3, and 4, respectively. If all arms of the bridge are active (or all are subject to strain-induced resistance change), then the circuit is called a full Wheatstone bridge. A half-bridge is described when R_3 and R_4 are fixed and R_1 and R_2 vary with strain. A quarter-bridge is described when only R_1 varies with strain and the other arms of the bridge are fixed.

Load cells and pressure transducers are examples of typical full-bridge sensors. An instrumented bolt or cable has strain gauges positioned at various distances along its length on opposite sides of the bolt. Each strain gauge is read by completing the bridge with fixed precision resistors (quarter-bridge circuit), which are usually located near the data acquisition unit. It is possible to use one set of completion resistors (R_2, R_3, and R_4) by using switches on the strain gauge leads. The switches to a particular strain gauge (R_1) are closed while the switches to the other strain gauges are left open. The next gauge may be read after opening the closed switches to the current strain gauge and closing the switches to the next strain gauge. The completion resistors should be kept as close to the temperature of the active gauge(s) as possible to avoid thermovoltaic errors.

In the case of a quarter-bridge circuit, long wire leads may affect the circuit by reducing sensitivity (see figure 2*A*, where $R_1 = R_g + 2R_L$). This sensitivity loss is typically compensated for by a reduction in the gauge factor used to convert voltage signal to strain. The loss resulting from signal attenuation may be cut approximately in half by using the three-wire quarter-bridge circuit, as shown in figure 2*B*.

Advantages of Wheatstone bridge technology are that response is immediate and very sensitive and the instrument can be used for measuring static or dynamic strain. Two disadvantages are that the datum-state reading shifts over time (zero drift) and that the instrument may respond secondarily to temperature. Much of the drift may be attributed to glue creep.

Another disadvantage of the Wheatstone bridge is the low level of the signal, ΔE. In a mine environment, there are several possible sources of external current or noise that may overshadow or drown the signal, including high-voltage power cables, leaky-feeder-type communication systems, machine noise, and multiple grounds. In most cases, such effects from external sources are not present. Instrument cables should be hung away from other power cables. Power cables should be crossed only when necessary. A shield should always be placed around the power cable, and the instrument cables should be routed so that they cross the power cable at a right angle.

Multiple grounds have periodically been a problem, particularly with instrumented bolts, where a connection to an earth ground may run through the bolt. If another earth ground has already been established (for example, near the datalogger), a ground loop is formed, which may drive bridge signals up or down, often completely out of its expected range. For this reason, the authors now choose to not connect the datalogger's power supply ground to an earth ground.

Potentiometer

Figure 3 shows a typical potentiometer circuit used to sense displacement. A resistive element is excited with a constant, known voltage V_x. A wiper contacts the resistive element along its length. One-dimensional movement of the wiper with respect to the resistive element is sensed by measuring the voltage of the wiper with respect to the ground. The measured wiper voltage ranges from 0 to the excitation voltage, depending on its contact location on the resistive element.

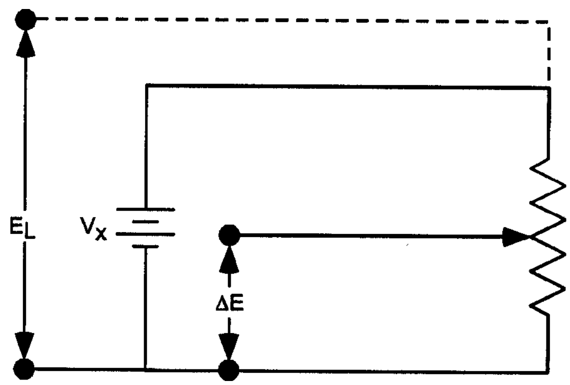


Figure 3.—Potentiometer circuit.

Potentiometers may be manufactured to cover a variety of displacement ranges. Sensitivity depends on the excitation voltage and the voltage monitoring device.

The authors have not encountered noise problems with potentiometers because the level of signal used has typically been 2 to 2.5 orders of magnitude above that of the Wheatstone bridge circuit. However, the user should be aware of the potential to burn up the wiper-resistive element contact if a large electrical current is discharged through the wiper following measurement. One precautionary measure is to connect a resistor to the wiper to restrict the amount of current that can pass through it. The authors have used a 1-kΩ resistor on the wiper with a 20-kΩ resistive element.

In the case of long lead wires, the user can choose to use the circuit as is. However, the circuit must be calibrated with the long lead wires before installation. The authors have determined applied excitation voltage using a fourth wire (dashed line in figure 3) to measure the voltage drop across one lead length. The voltage applied at the resistive element is the excitation voltage applied at the datalogger less the drop across two lead lengths. The voltage drop across one lead length is determined with the electric potential between the dashed wire and analog ground. By assuming equal voltage drop in both lead wires, the applied excitation at the resistive element is determined using the following equation:

$$\text{Applied excitation at the resistive element} \approx V_x - 2(V_x - E_L), \quad (2)$$

where V_x = excitation voltage applied at the datalogger; and

E_L = the measurement in volts between the dashed lead and analog ground.

The potentiometer reading may be normalized with this applied excitation voltage. The raw reading, applied voltage, and normalized reading should be stored in the measurement data file.

Vibrating-Wire Transducer

Vibrating-wire transducers are used in many instruments, including load cells, deformation gauges, surface and embedment strain gauges, earth pressure cells, pressure sensors for piezometers, and liquid level settlement gauges [Dunnicliff 1988; McRae and Simmonds 1991; Choquet et al. 1999]. A steel wire is tensioned between two clamps so that it can vibrate at resonant frequency. As the clamps are physically moved relative to each other by applied force to the sensor (displacement transducers use a linear spring to convert displacement to a proportional force on the vibrating wire), the resonant frequency changes. A coil is positioned near the wire. When current is run through the coil, the resulting magnetic field in effect plucks the wire, causing it to vibrate at a resonant frequency reflective of the external force. The vibrations, in turn, cause voltage fluctuations in the coil that correspond to the vibrations. A handheld readout box or automated data acquisition system measures the time it takes to complete a given number of vibration cycles and computes the frequency. Sometimes an interface unit is used to boost the excitation voltage. Signal conditioning and conversion to digits of frequency squared or engineering units are performed by a datalogger.

In the case of a continuous readout, two coils are used. One coil electronically plucks the wire and senses the vibration-caused voltage fluctuations. This frequency is calculated. The second coil vibrates the wire at the same frequency. As frequency changes, so does the plucking frequency of the second coil.

There are several distinct advantages to using vibrating-wire transducers. First, the readings are not sensitive to long lead wire lengths because frequency, rather than voltage level, is being measured. Low-capacitance cable may be required for lead lengths greater than 200 m (650 ft). This means that a cable can be cut in the field to the length needed without offsetting the output signal. Moreover, Choquet et al. [1999] report that manufacturers have perfected the manufacturing process to the degree that there is very little drift in the reading at no load or zero displacement over time and only small changes occur in calibration slopes (within ±0.5%) after nearly 4 years. McRae and Simmonds [1991] report that all of their tests, some lasting for over 10 years, show that if wire stress is kept below 30% of yield, long-term drift will be minimal. For pressure transducers where minimum stress is only 13% of yield, long-term drift is insignificant. McRae [2000] reports that his tests show virtually no drift after 15 years. This performance makes vibrating-wire technology very desirable for long-term monitoring.

One problem that has surfaced from time to time is that the datalogger will fail to obtain a meaningful reading when it is monitoring a vibrating-wire transducer that incorporates a vibrating-wire interface to the datalogger. One explanation may be that the selected frequency sweep range did not include the frequency to be read. More often, however, it is a contact problem where wires are clamped at terminals. These contacts must be tight and of good quality. In the authors' experience, when a poor connection was discovered during sensor checks or calibrations, improving the contact generally solved the problem. A vibrating-wire readout box provides a good backup, as it provides more signal filtering. The authors have never encountered a problem when using a vibrating-wire readout box instead of a datalogger. Experience has helped minimize problems encountered when the sensors are monitored with a data acquisition system.

DATA ACQUISITION SYSTEM BASICS

The authors have used dataloggers from Campbell Scientific, Inc., for several years. Models 21XQM (now discontinued by the manufacturer) and CR10QM are approved by the Mine Safety and Health Administration for use in methane-air atmospheres. Although operation of one of these dataloggers is relatively simple, learning to use the system may require a little effort. There is a learning curve for the dataloggers and related components, but unattended instrument monitoring and recording are usually well worth the effort required to learn to use the system. In this section, basics of using these dataloggers and associated components are discussed to help the user considering sensor monitoring with an unattended data acquisition system.

The dataloggers collect two basic measurements: single-ended voltage and differential voltage. The difference between these types of measurements will be aided by reference to figure 4, which shows a CR10QM wiring panel with labeled terminals.

A single-ended measurement is a measurement of voltage with respect to ground. For example, a single-ended measurement would be measured between two wire leads. One lead must go into one of the analog ground (AG) terminals. The other lead must connect to one of the input channels (1H (high), 1L (low), 2H, 2L, etc.). The input channel must be specified with a measurement command in the datalogger program. For example, a potentiometer is read with a single-ended connection to the datalogger. A vibrating-wire sensor with an in-line interface is connected to the CR10QM in the same way; however, instead of a single-ended measurement, the datalogger measures time for a certain number of vibration cycles and converts that measurement to frequency squared or engineering units.

A differential measurement is a measurement of voltage in one lead with respect to that in another lead. The two leads are connected to differential channel input terminals (1H and 1L, or 2H and 2L, or 3H and 3L, etc.). The voltage of the high (H) terminal is measured with respect to the low (L) terminal. For example, Wheatstone bridge circuits are measured by connecting the signal leads to a set of differential channel input terminals on the datalogger.

A CR10QM datalogger has 6 differential inputs or 12 single-end inputs. The 21XQM has 8 and 16 inputs, respectively. The number of sensors monitored may be extended by using multiplexers. A multiplexer is an electronic switching unit with mechanical relays. It has 4 common terminals and 16 channels of 4 terminals each. A multiplexer is activated by sending and maintaining a voltage that is greater than 3.5 V from a datalogger control port (setting the port high) to the multiplexer's "reset" terminal. When a pulse is sent from a control port of the datalogger to the multiplexer's "clock" terminal, switches close between the multiplexers' four common terminals and the four respective terminals of channel set 1. The next pulse opens these switches and closes the switches between the common terminals and the four terminals of channel set 2. The multiplexer is deactivated by removing the current through the control port (setting the port low).

Wires are run from datalogger terminals (inputs, excitation, and ground) to the four common terminals on the multiplexer. Sensor leads are connected to the appropriate channel set terminals.

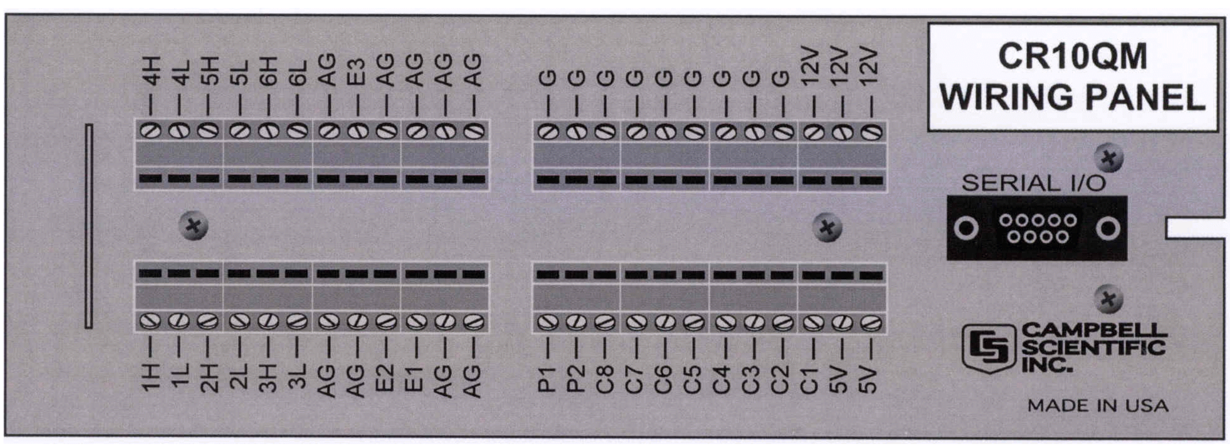


Figure 4.—Campbell Scientific CR10QM wiring panel.

The multiplexer increases the number of possible sensors that may be monitored through a datalogger input terminal or set of terminals. For example, the number of vibrating-wire transducers that may be monitored through a multiplexer is 32 (2 per channel set). Potentiometers require an excitation lead, and Wheatstone bridges require excitation and ground leads. Therefore, it is possible to monitor 16 potentiometers or 16 Wheatstone bridges through 1 multiplexer. Sensor types are generally not mixed on the same multiplexer.

The addition of multiplexers to a datalogger adds some complexity to reading sensors. When a clock pulse is sent to a multiplexer, a set of mechanical relays is closed. To ensure that contact has been made, the user must insert a programmed delay before the sensor is read. Typically, a delay of 0.02 to 0.03 sec is sufficient. Also, a delay of 0.05 sec is good practice follow-ing multiplexer deactivation.

PC208W software and interfaces are used for communications, viewing data, and programming. For example, one subprogram provides a real-time connection to the datalogger from a personal computer through an optical interface. Another subprogram is used to communicate with a storage module through an RS-232 cable and interface. Another application is used to view data, and another to process information from those data.

A good programming structural sequence is to—

1. Read the sensors, usually in groups (in a loop) by multiplexer (readings are stored in input locations);
2. Read sensors internal to the datalogger (such as cold-junction temperature, battery voltage, program signature, etc.);
3. Set the output flag;
4. Read time and date;
5. Set resolution (usually high) and write readings from input locations to final storage; and
6. Send the data to an output device, such as a storage module.

Before writing a datalogger program, all circuits should be diagramed from the datalogger to the sensor so that it is clear what programming instructions and parameters are to be used. With this information, programming is relatively simple. For the user's first program, it may be a good idea to contract with Campbell Scientific for programming assistance. Once the user understands that program, additional programming is easy.

Once the system is installed, one potential operating problem involves electrical grounds. Until recently, it was the authors' practice to connect all cable shields to shield terminals and connect the terminals to one earth ground near the datalogger. In addition, the authors would connect the datalogger power supply ground to the same earth ground. While this practice seems logical for protecting transducers and the datalogger when a cable is compromised, it can also affect readings if the system is connected to a Wheatstone bridge. Ground potentials can vary significantly between sensor locations. In fact, one of the authors has measured a difference in ground potential of approximately 1 V between points in a coal mine roof just a few feet apart. When the instrument itself becomes part of a ground, as can happen with instrumented bolts and cables, any difference in ground potential between the instrument and the ground near the datalogger induces a ground loop and affects measurements. In such cases, it is not a good practice to connect shields at the multiplexers or datalogger. Recently, the authors began leaving the datalogger ground floating (not connected to an earth ground).

The authors always collect and store raw analog measurements. Sometimes the raw data are also processed and stored, but raw data are always saved. This procedure provides a backup in case of an inadvertent mistake in processing.

Lastly, the authors strongly recommend spending some time in the laboratory performing checks on the instruments and datalogger before installing the instruments in the field. This check avoids many problems that may be more difficult to solve underground or after installation. Monitoring sensors over a period of time in the laboratory can identify many problems. Similarly, instruments should also be checked at the installation site before they are installed. Experience has shown that use of connectors should be avoided if possible. The authors prefer to hard-wire instrument cables to datalogger and multiplexer terminals. This avoids problems such as cold solder junctions and the abuse that connectors always suffer underground as cables are run. In the event that use of connectors is unavoidable, cold solder junctions may be found by identifying erratic sensor readings when the connector is heated with hot air.

INSTRUMENTS

Experience is important to increase the quality and probability of success in installing instruments. In the sections that follow, specific instruments and instrument types that the authors have used are described. Comments and tips are offered.

DISPLACEMENT INSTRUMENTS

The authors have used both potentiometer (linear and rotary) and vibrating-wire sensors in instruments that measure displacement. Some instruments were purchased from various manufacturers, while the hardware of some other instruments was manufactured in-house. The type of instrument used should depend on how much displacement is expected, number of required in-borehole anchors, instrument accuracy, stability over time, and operational constraints. For small expected displacements over a long period of time, grouted anchors or hydraulic anchors may be more desirable than spring anchors. For larger expected displacements, a spring with attached piano wire running to string potentiometers mounted at the borehole collar may be sufficient.

One of the authors used rotary potentiometers with a geared spindle that was supposed to mesh with a threaded rod. However, the gears sometimes skipped because the gear teeth and thread profile did not match well. The skipping problem was also very dependent on the force between these members. Force adjustments did not solve the problem.

Simple instruments that require manual measurements have been tried by the authors with varying degrees of success, because several possible sources of error may be introduced. Therefore, the authors generally avoid instruments that require manual measurements except where necessary (such as closure measurements).

Instruments are available from several manufacturers. The authors have used vibrating-wire sensors, and linear and rotary potentiometers. Variations on these instruments are available from Geokon, Inc., Lebanon, NH; Roctest, Inc., Plattsburgh, NY; RST Instruments, Vancouver, British Columbia, Canada; and Slope Indicator Co., Bothell, WA. SRL researchers have used string (rotary) potentiometers made by Celesco Transducer Products, Inc., Canoga Park, CA, with hardware manufactured in-house to monitor sag at different depths in a coal mine roof [Signer et al. 1993].

INSTRUMENTED BOLTS

An instrumented bolt [Serbousek and Signer 1987; Johnston and Cox 1993] (figure 5) is an instrument that has been used increasingly in recent years by SRL researchers [Signer 1990, 1994; Larson et al. 1995; Larson and Maleki 1996]. It consists of strain gauges bonded to milled surfaces on opposite sides of a bolt. In addition to measuring strain, the gauges may be calibrated in the elastic range to calculate load directly as long as load has not progressed past the yield point of the bolt steel. If gauges are monitored over time, a row of these instrumented bolts reveals useful information about roof deformation and roof conditions. The information may aid in determining potential roof hazards and justifying changes in roof support to optimize or increase safety. The instruments are commercially available from Jenmarr Corp., Pittsburgh, PA, and Rock Mechanics Technology, Ltd., Staffordshire, U.K., although the authors have only used bolts instrumented by SRL technicians.

Because of the low level of the signal, the potential of the instrument serving as a ground, and differences in ground potential from point to point in an underground roof, external noise may be introduced to the gauge signal. The user should follow suggestions found in the "Data Acquisition System Basics" section to minimize the likelihood of this problem.

If long cables link the gauges to a data acquisition system, a three-wire, quarter-bridge circuit should be used.

INSTRUMENTED CABLE BOLTS

A new development in cable bolt sensor technology is replacement of the bolt's original king wire with an instrumented king wire [Martin et al. 2000] (figure 6). Similar to an instrumented bolt, an instrumented cable bolt is a support device as well as a rock mass strain-measuring device in underground mines. Characteristics of the rock mass in an underground mining environment can be determined by these devices, which provide a means for mining engineers and mine inspectors to predict roof falls.

The gauges are mounted on a narrow strip of steel at intervals determined by expected roof conditions, the instrumented metal strip is placed in a steel mold, and the mold is injected with epoxy. This results in a new king wire. The remaining six cables are then rewound around the new king wire, creating a seven-stranded instrumented cable. Maximum length of the cables is 6 m (20 ft) because of the physical constraints of the wires connecting the gauges. The maximum number of gauges is 10, or 5 on each side.

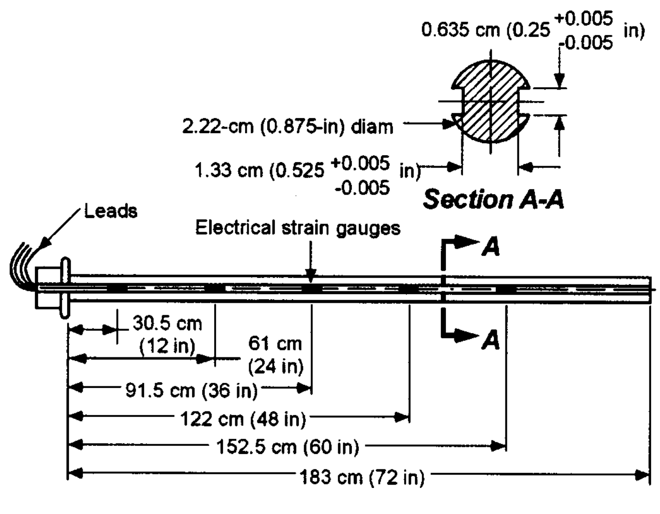


Figure 5.—Instrumented bolt (after Signer and Lewis [1998]).

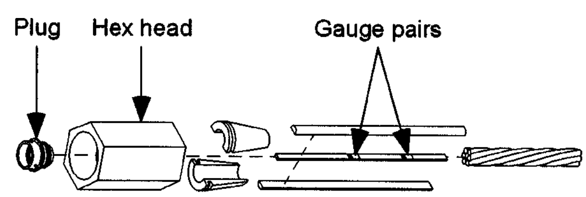


Figure 6.—Instrumented cable bolt (from Martin et al. [2000]).

The gauges can either be set in pairs in the same plane to detect shear or at 90° angles to detect lateral loading. The cables are inserted in a mine roof using cement or resin grout. They are capable of holding an ultimate load of 214 kN (48,000 lb). Data readings are reliable to bolt loads of 178 kN (40,000 lb).

The cables are fitted with a special hex head and inserted into the rock with either a jackleg drill or a conventional roof bolting machine. The cable bolts are spun into the drill hole, mixing the grout for adhesion. The special head also has a connector that allows for data acquisition after installation.

The same cautions and limitations that apply to instrumented bolts apply to instrumented cable bolts.

HOLLOW INCLUSION CELL

Worotnicki and Walton [1976] developed the hollow inclusion stress cell (HICell) (figure 7) at the Commonwealth Scientific and Industrial Research Organization (CSIRO) in Australia. These cells are available from Mindata, Seaford, Victoria, Australia, and from Reliable Geo, LLC, Yakima, WA, in the United States. The HICell is actually a group of strain gauges and strain gauge rosettes embedded in a thin epoxy shell. It is bonded to the wall of an EX-size borehole. Installers drill to the approximate installation depth with a 152-mm (6-in) barrel. An EX hole is extended for approximately 61 cm (24 in). Glue is displaced by a piston and forced into the annulus between the shell and the borehole wall. When stress is relieved through overcoring [Hooker and Bickel 1974], the stress state may be inferred from strain measurements collected from one installation.

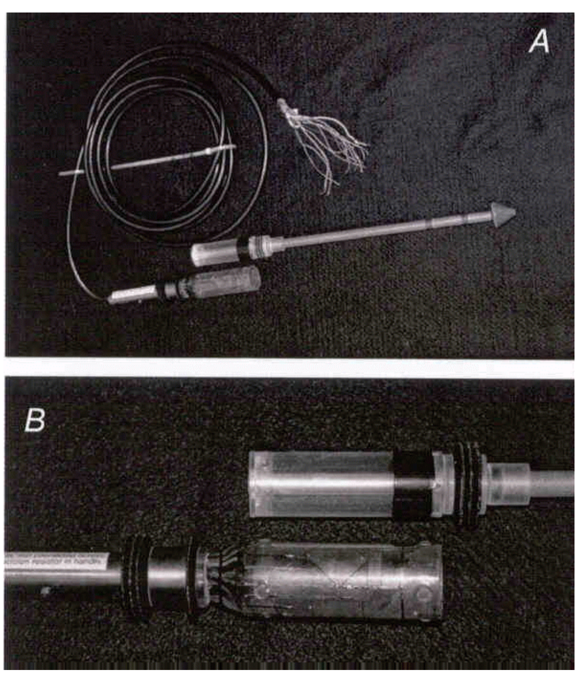


Figure 7.—Hollow inclusion stress cell. *A*, Strain gauge shell with piston, dowel, and cable; *B*, closeup of shell and piston.

Isotropic elastic properties are determined using results of a biaxial test on a recovered rock core and instrument. For isotropic rock, Larson [1992] developed a data reduction code, STRESsOUT. For transversely isotropic rock, five independent elastic constants are needed, and Amadei [1983] has developed a data reduction solution. However, laboratory tests on cores with orientations along the axes of anisotropy are required to determine all elastic constants. These constants can be estimated, but such estimates decrease the quality of stress state determination.

Tesarik and McKibbin [1989] installed and overcored four HICells at the Magmont Mine, a lead mine in Missouri. The stresses they determined from the overcoring measurements were subsequently used as initial stresses in a two-dimensional, finite-element model employed to analyze stress redistribution during pillar recovery.

Johnson et al. [1993] overcored four HICells and a borehole deformation gauge in three boreholes at one site and two boreholes at another site in the Homestake Mine, Lead, SD. Because results from overcoring varied, they discussed causes, such as variability of geology and material properties, anisotropy, and use of statistics with the least-squares approach.

In an underground platinum and palladium mine, Johnson et al. [1999] used HICells to measure in situ stress and subsequent stress change ahead of the face during advance of a footwall drift. Stress components calculated by multiple linear regression fit the strain measurements with little error. The correlation coefficients for each determination were in the range of 0.90 to 1.00.

Usually it is difficult to obtain good results in coal-measure rock because most sedimentary cores fracture easily during drilling. However, if good cores can be obtained, overcoring the HICell will usually result in good-quality measurements.

Conducting a dry run before HICell installation is very important. This is accomplished by using nails instead of lead pins in the shear pinholes. This practice allows an installer to learn to sense the right technique for inserting an instrument in an EX borehole.

A problem was recently encountered while installing a HICell. Three seal rings are positioned close together on the base of the instrument and on the piston. Recently, it was found that the grooves for those seals were positioned closer together than had been done in the past. In a 38.1-mm (1.500-in) hole, these seals did not have room to bend and thus were forced over the top of each other. This made it impossible to push the piston into the EX hole. A solution discovered by the SRL installation team and verified by Mindata [Walton 1999] was to remove the middle seal on both the base of the instrument and the piston. We have completed two successful installations with this modification. Also, Mindata now recommends not cutting V-shaped grooves in the seals, as is encouraged in the installation manual.

BOREHOLE DEFORMATION GAUGE

The three-component borehole deformation gauge (figure 8) was designed by the former U.S. Bureau of Mines (USBM) [Merrill 1967]. Pistons that contact the borehole wall press against cantilevers mounted inside the body of the instrument (figure 9). Strain gauges bonded near the base of the cantilevers (two on the top surface and two on the lower surface) are connected as a full Wheatstone bridge and calibrated to reflect changes in the diameter of the borehole. Like the HICell, this gauge is installed in an EX hole and overcored [Hooker and Bickel 1974; Bickel 1993]. During overcoring, borehole diameter changes are measured in three different orientations spaced at 60° intervals. Pistons that contact the borehole wall displace a cantilever. Strain gauges bonded near the base of the cantilever form a full Wheatstone bridge circuit. Changes in signal are calibrated to calculate changes in diameter. After overcoring, a biaxial test is performed to measure the Young's modulus of the overcore.

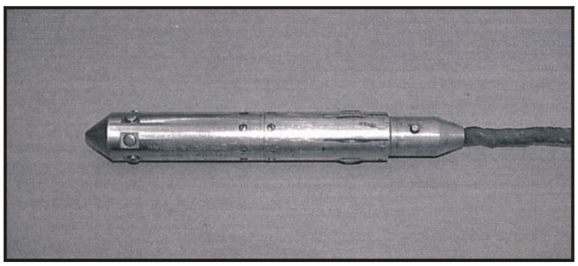


Figure 8.—Borehole deformation gauge.

The borehole deformation gauge is easy to use and install. There is no curing time as there is for the HICell. However, much drilling is required to determine all independent components of the stress tensor because the instrument must be overcored three different times, each time in a different non-coplanar borehole. Stress components are fit to the measurements using the least-squares method, as demonstrated by Panek [1966]. This calculation may be performed with the STRESsOUT code [Larson 1992].

NIOSH researchers have recently used this instrument for measuring horizontal stress in a coal mine roof [McKibbin 2000]. In this case, one borehole orientation was used. The results from multiple overcores were consistent. The method and data reduction are described by Bickel [1993], but in his analysis, he assumed that the borehole axial strain was zero because it was not measured. That assumption must be evaluated. Equations used in data reduction are discussed by Obert and Duvall [1967].

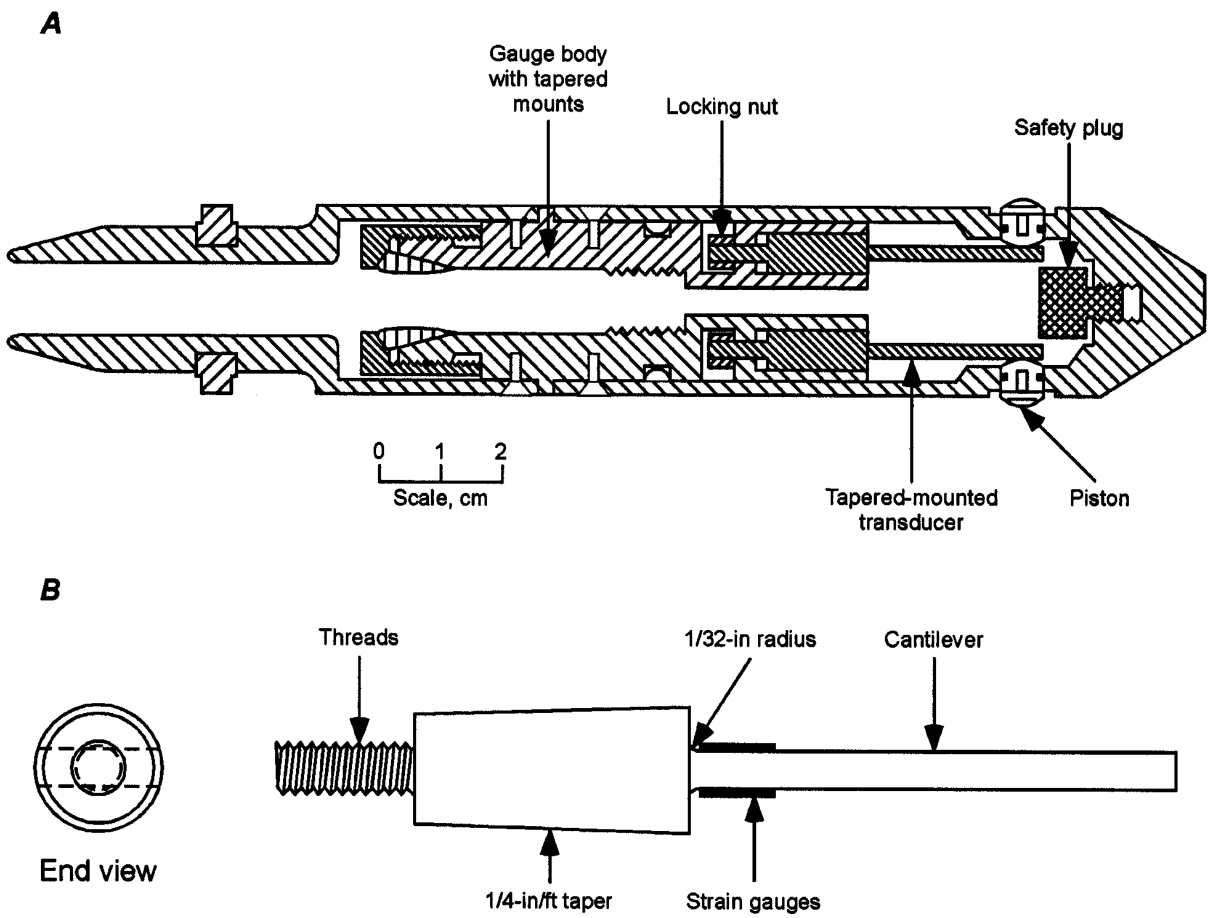


Figure 9.—Schematic of three-component borehole deformation gauge. *A*, Main body showing location of tapered-mounted cantilevers and pistons; *B*, closeup of a cantilever showing location of strain gauges (after Bickel [1993]).

Girard et al. [1997] overcored a borehole deformation gauge at the 7400 level of the Homestake Mine. The stress state determined from these measurements compared well with stresses calculated from equations developed by Pariseau [1986], which represented principal stresses as a function of depth. Measurements by Johnson et al. [1993] in conjunction with HICell measurements were mentioned in the previous section.

The borehole deformation gauge is available from Geokon.

BIAXIAL STRESSMETER

The biaxial stressmeter is a rugged and reliable stress monitoring instrument manufactured by Geokon. The stressmeter is typically grouted in a BX-size (60-mm (2.36-in)) drill hole to measure compressive stress changes in a plane perpendicular to the instrument's longitudinal axis. As the walls of the drill hole deform in response to stress increases induced by mining, deformation in the host rock is transferred through a thin, grouted annulus to the stressmeter's body (a thick-walled steel cylinder approximately 34 cm (13 in) long). Radial deformation of the steel cylinder is measured by one or two sets of three vibrating-wire gauges oriented 60° from each other and closely spaced along the center of the instrument's longitudinal axis (figure 10). From these radial deformation measurements, the magnitude and orientation of changes in major and minor secondary principal stresses can be calculated using equations developed by Savin [1961]. The biaxial stressmeter can also be equipped with two longitudinal vibrating-wire gauges that measure overall change in the length of the instrument. To account for the effects of stress changes outside of the biaxial measurement plane, the longitudinal measurements are used to calculate axial strain and correct the computed secondary principal stress changes for the Poisson component of stress changes acting in the direction of the drill hole. In addition, the stressmeter can also be equipped with two vibrating-wire temperature sensors that measure the temperature at the instrument's location. Because the vibrating-wire gauges have a slight temperature sensitivity, the manufacturer provides temperature calibration factors for each of the radial and longitudinal gauges. Data from the temperature sensors are used in conjunction with these calibration factors to correct the computed radial and longitudinal strains and compensate for the influence of temperature fluctuations near the instrument.

The manufacturer currently supplies various models of the biaxial stressmeter with either three or six radial gauges and with or without the longitudinal gauges and temperature sensors. Because consistent readings from all three of the radial gauge orientations (i.e., 0°, 60°, and 120°) are required to calculate the magnitude and orientation of the secondary

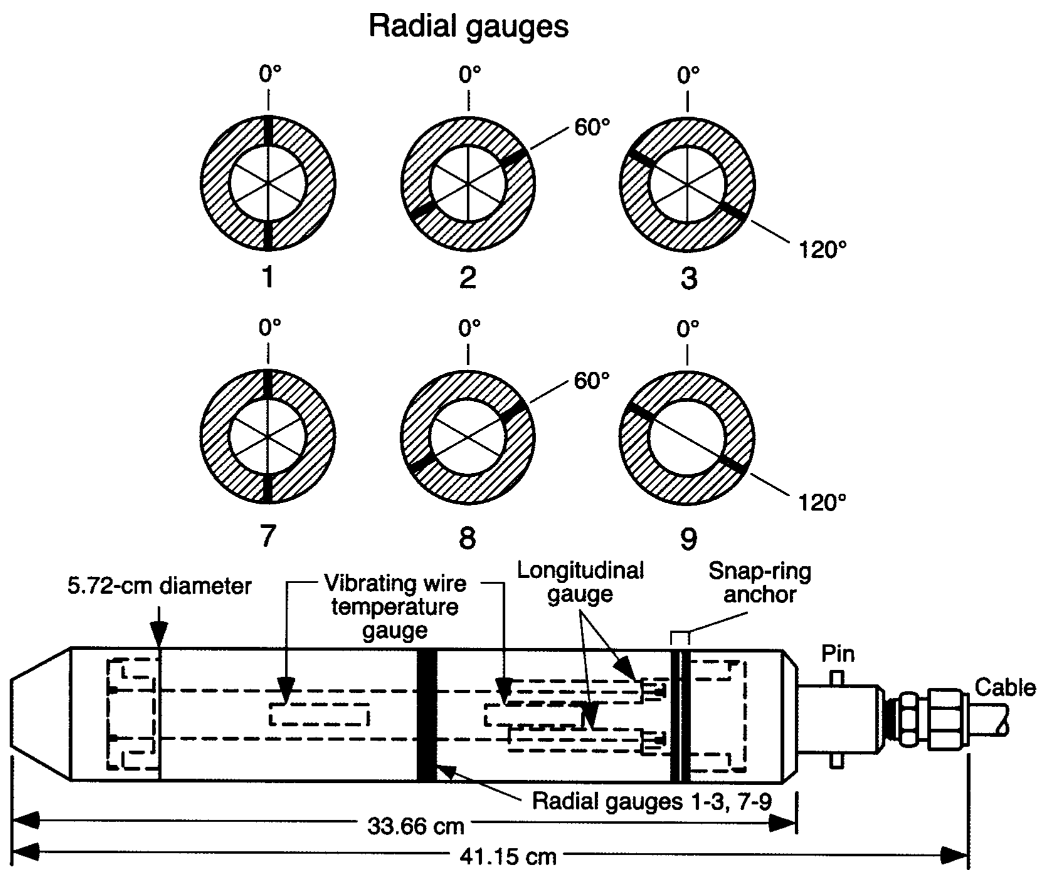


Figure 10.—Biaxial stressmeter (from Seymour et al. [1999]).

principal stress changes, the authors highly recommend using a duplicate set of radial gauges for data redundancy and added reliability. Although the amount of longitudinal and temperature corrections have generally been small, the authors also prefer to order the biaxial stressmeter with two longitudinal gauges and two temperature sensors so that these corrections can be quantified accurately and the stress changes adjusted if necessary. For situations where maximum accuracy is desired or the temperature changes are expected to be large, the instrument should be ordered with the full complement of radial, longitudinal, and temperature gauges.

Before installing the biaxial stressmeter, the authors typically perform a point-load test with the instrument to confirm internal wiring configurations, verify the operation of the radial gauges, and check the configuration and programming of data acquisition equipment. Usually the electronic readings collected from the biaxial stressmeter are analyzed using spreadsheet software because of the complexity and number of data-reduction equations. As a general rule, the response of the vibrating-wire transducers should be graphed and analyzed prior to converting the electronic readings to engineering units and calculating the magnitude and direction of the stress change. Graphing the electronic readings helps identify anomalous data, diagnose the performance of the datalogger components, and interpret the response of individual gauges that are not functioning as expected. Inconsistent data can be difficult to detect and interpret from stress change calculations alone.

To measure stress changes in a vertical plane, the biaxial stressmeter is typically installed in a drill hole inclined slightly below horizontal to ensure that the grout flows to the bottom of the hole and completely fills the annulus around the instrument. The installation hole is usually drilled with a diamond core drill for about 1.5 m (5 ft) past the target depth for the instrument. After initially filling the hole with about 3 m (10 ft) of grout, the biaxial stressmeter is inserted into the drill hole, pushed through the grout to the target depth, and carefully rotated into position (i.e., radial gauge 1 oriented vertically) using self-aligning setting rods or a leveling device. To lock the instrument securely at this desired position in the drill hole, two opposing snap-ring anchors are activated from the collar of the hole by pulling a cable attached to an anchor-release pin.

The remainder of the hole is then filled with grout to help ensure that the biaxial stressmeter is completely covered with grout. Grouting the entire hole also alleviates problems with stress concentrations or creep in the drill hole near the instrument's location, which can introduce errors in the stress change measurements. The biaxial stressmeter is generally installed using a relatively thick mixture of Five Star Special Grout 400, an expansive cement-based grout manufactured by Five Star Products, Inc., of Fairfield, CT. A thick grout mix helps deter grout migration through fractures or discontinuities near the drill hole and provides sufficient viscosity to displace extraneous water from the hole.

Using these procedures, Seymour et al. [1999] have installed biaxial stressmeters in a variety of underground mines and host rock formations to monitor and evaluate mining-induced stress changes in mine pillars and abutments. By measuring both the magnitude and direction of the secondary principal stress changes, the biaxial stressmeter provides valuable insights into the behavior of the host rock and the redistribution of mining-induced stresses. Data from these instruments have been used to determine the transfer of mining-induced loads to cemented backfill pillars [Tesarik et al. 1991], evaluate the ground support capability of frozen gravel pillars [Seymour et al. 1996], and monitor stress redistribution ahead of a longwall [Seymour et al. 1998]. The instrument's robust design and vibrating-wire transducers are particularly well suited for monitoring long-term changes in stress. Because the biaxial stressmeter is securely grouted in the host rock, the instrument does not seem to be adversely affected by vibrations caused by blasting or mining equipment.

To measure stress changes in a horizontal plane, the biaxial stressmeter is installed in a vertical drill hole. At a proposed nuclear waste repository in Sweden, several biaxial stressmeters have been installed in boreholes drilled in the bottom of a test drift to measure both thermal and mining-induced stress changes around cylindrical excavations that may eventually house canisters containing radioactive waste material [Röshoff 1999]. In addition, Geokon is currently developing a borehole packer for installing the biaxial stressmeter in overhead drill holes. SRL researchers recently used a prototype version of this borehole packer to install biaxial stressmeters in the roof of a coal mine.

BOREHOLE PRESSURE CELLS

Borehole pressure cells are used to monitor stress change in one direction (e.g., vertical stress change profiles at several points through a pillar). They consist of rectangular flatjacks embedded in a cylinder of grout (figure 11). The cylinder is placed in a 60-mm (2.375-in) diameter borehole and set to a pressure about 10% more than the estimated absolute stress in the host rock. The viscoelastic-viscoplastic nature of the coal or rock allows a pressure equilibrium to be reached. Thereafter,

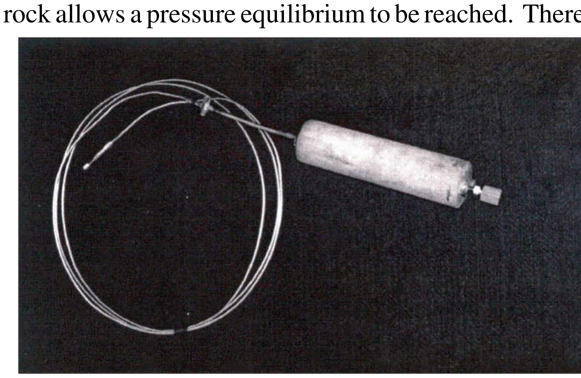


Figure 11.—Borehole pressure cell.

any changes in stress of the host rock are reflected directly in equal changes in fluid pressure in the flatjack.

These gauges may be monitored with hydraulic gauges. Typically, we have used both hydraulic gauges and pressure transducers and monitored them with a datalogger.

Three cells may be installed in the same borehole. After a cell is placed and oriented in the borehole, the cell is seated by pumping hydraulic fluid into the flatjack. The cell is then pressurized to a predetermined level before the valve is turned off. Resetting the pressure is sometimes necessary after 24 hr.

The hydraulic failure rate appears to be about 10% to 20% when the cells are manufactured by experienced hands. Good-quality cells can only be achieved if made by a careful, experienced manufacturer. The authors used cells manufactured only by employees of the Denver Research Center of the former USBM. However, Geokon manufactures a flatjack intended to be grouted into a borehole.

EARTH PRESSURE CELLS

Hydraulic earth pressure cells (figure 12) are commonly used to measure stress changes in unconsolidated or cemented materials. This instrument consists of two rectangular or circular stainless steel plates welded together around their edges. A thin cavity is left between the two plates and filled with a fluid such as antifreeze or hydraulic oil. Pressure applied to the cell induces an equal pressure on the internal fluid that, in turn, is sensed by a transducer connected to the cavity between the two welded plates with high-pressure stainless steel tubing [Dunnicliff 1988].

Earth pressure cells are generally used to identify loading trends and are not relied upon for precise measurements. Because the stiffness of the instrument and the medium in which it is placed most likely will not be equal, the earth pressure cell may not sense the same stress change as the surrounding medium. Furthermore, the modulus of the in-place material may vary considerably from one location to another as a result of unequal compaction or heterogenous components, such as coarse or fine aggregate. This modulus variability may cause discrepancies or scatter in the stress change measurements. Finally, it is difficult to calibrate the instrument accurately in the laboratory in a medium other than fluid.

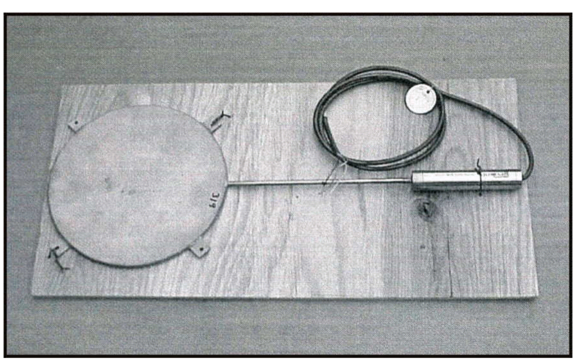


Figure 12.—Earth pressure cell.

The authors generally use earth pressure cells with vibrating-wire transducers to measure stress changes in cemented backfill. To avoid point loading or damaging these instruments when they are installed in cemented rockfill, the earth pressure cells are precast in wood forms using a cemented backfill mix with minus 6.35-mm (0.25-in) aggregate. The forms are constructed large enough to form a protective layer several inches thick on all sides of the instruments. As soon as the cast instruments can be handled without breaking the protective layer, the forms are removed, and the instruments are placed at their desired orientation in the backfill stope. Casting the instruments helps maintain alignment during installation and protects the cells when wet backfill is placed over them. To ensure that the desired orientation of the earth pressure cells is maintained during the bulk filling of an underground opening, an additional protective layer of cemented rockfill approximately 0.6 m (2 ft) thick is placed over the instruments and allowed to cure for at least 24 hr before heavy equipment is driven over the in-struments. Earth pressure cells installed using these procedures have supplied important information regarding the mining-induced loads that were transferred to cemented backfill stopes as adjacent ore panels or pillars were mined [Tesarik et al. 1993, 1995]. They have also been used to evaluate the long-term ground support provided by cemented backfill [Tesarik et al. 1999].

In mines that use hydraulic fill, earth pressure cells are anchored to wire mesh using cable ties or bailing wire. If wire mesh or some other means of anchoring the instrument is not available at the desired location in the stope, the instruments can be suspended with wire in a wooden frame without a top or bottom and placed directly in the fill by hand. However, care should be taken to maintain the desired orientation of the instrument. This technique was successfully used at an underground coal mine; the earth pressure cells were installed in in-panel entries as they were backfilled with an air-entrained mixture of fly ash and cement [Seymour et al. 1998].

Earth pressure cells are made by several instrument manufacturers, including Geokon, Roctest, RST Instruments, and Slope Indicator Co.

EMBEDMENT STRAIN GAUGES

The authors generally use embedment strain gauges (figure 13) with vibrating-wire transducers to measure relative displacement in cemented backfill. Strain is calculated by dividing the relative displacement between the instrument's two end flanges by the length of the instrument, typically 15 to 25 cm (6 to 10 in). Typically these instruments are installed

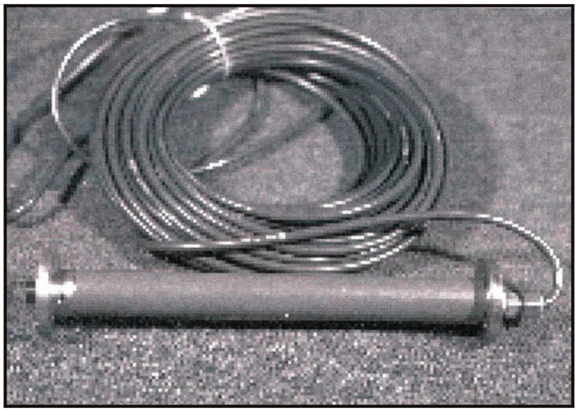


Figure 13.—Embedment strain gauge.

near the location of an earth pressure cell to provide an estimate of the backfill's in situ modulus (i.e., stress change divided by strain change).

For cemented backfill with a modulus ranging between 690 and 1,380 MPa (100,000 and 200,000 psi), a 25.4-cm (10-in) long embedment strain gauge with a 6.35-mm (0.25-in) maximum displacement range can be used to determine strain changes up to a stress change level of between 17 to 34 MPa (2,500 to 5,000 psi). Problems have been encountered using embedment strain gauges with smaller displacement ranges because the maximum displacement range was exceeded. On the other hand, erratic results were obtained from 1-m (39-in) long instruments with 12.7-mm (0.5-in) displacement ranges that were placed vertically in cemented fly ash. The reason for these variations is not fully understood, but the embedment strain gauges may have buckled as mining-induced loads were transferred to the backfilled entries because of the longer length of the instruments.

Installation procedures for embedment strain gauges are the same as for earth pressure cells, except waxed cardboard cylinders are typically used to precast the instruments instead of wooden forms. Except for the above-mentioned problems, these instruments have generally provided good results and have typically given a more precise indication of backfill loading changes than the earth pressure cells.

Embedment strain gauges are made by Geokon, Roctest, and Slope Indicator Co.

VERTICAL BACKFILL EXTENSOMETERS

To measure relative displacement over large vertical distances in backfill, vertical extensometers can be constructed as shown in figure 14. For applications where boreholes can be drilled in placed backfill, sections of 15.9-mm (0.625-in) diameter steel rod coupled together serve as the extensometer measurement rod, and steel rebar welded to a small length of rod is used as the downhole anchor. A mixture of cement and water is poured down the hole to set the anchor, followed by dry sand to prevent pieces of backfill from spalling and wedging between the steel rod and the side of the drillhole. The head assembly, which consists of a vibrating-wire displacement transducer, bearing plate, and a short length of 38-mm (1.5-in) steel pipe welded to the plate, is placed over the steel measurement rod, and the transducer is screwed into the rod. The transducer is set at the desired position using an aluminum extension rod and is held in place with a compression fitting. Finally, the excess portion of the aluminum rod extending above the compression fitting is cut off, and a protective head is bolted onto the bearing plate. These instruments have successfully monitored compressive strains in backfill caused by overburden stress redistribution, as well as tensional strains caused by undercutting backfill [Tesarik et al. 1993].

When drilling in cured backfill is not feasible, extensometers can be assembled as successive backfill lifts are placed. In this situation, the bottom anchor is constructed of a 45.7- by 45.7-cm (18- by 18-in) steel baseplate with a 51-mm (2-in) steel coupler welded to the top of the plate. The first section of a 15.9-mm (0.625-in) diameter steel measurement rod is inserted through a hole in the baseplate and secured with a nut. The first section of 51-mm (2-in) pipe is threaded into the steel coupler to protect the measurement rod as backfill is placed around the instrument. The outside surface of this pipe is greased during construction to help reduce friction between the outer surface of the pipe and the cured backfill. As successive lifts of backfill are placed, sections of rod and pipe are coupled together until the desired instrument height is achieved. The 38-mm (1.5-in) steel pipe on the head assembly is then inserted inside the 51-mm (2-in) protective pipe, leaving several inches of backfill between the bearing plate and the top of the 51-mm (2-in) pipe. This gap allows the instrument to retract as the backfill compresses under load. The transducer is set and secured at the desired position as described above, and the protective cover is bolted onto the head assembly. Following these procedures, backfill extensometers ranging from 4.3 to 16.5 m (14 to 54 ft) long have been successfully installed in cemented fill stopes and used to determine the vertical strain induced within the backfill by mining [Tesarik et al. 1991, 1995].

HORIZONTAL BACKFILL EXTENSOMETERS

The authors have attempted to measure horizontal strain in cemented backfill stopes (horizontal dilation in response to vertical loading), with only marginal results. At one mine site, two 3-m (10-ft) long commercially available borehole joint meters were hung end-to-end from steel cables that spanned the width of a 7.3-m (24-ft) wide stope [Tesarik et al. 1991]. Some of these instruments were damaged during installation when backfill dumped from the bench above surged forward and distorted the flexible instrument into a slight arc. Data recorded by the jointmeters that still functioned after the stope was completely filled were difficult to interpret because, unlike most of the other rock mechanics instruments installed at this site,

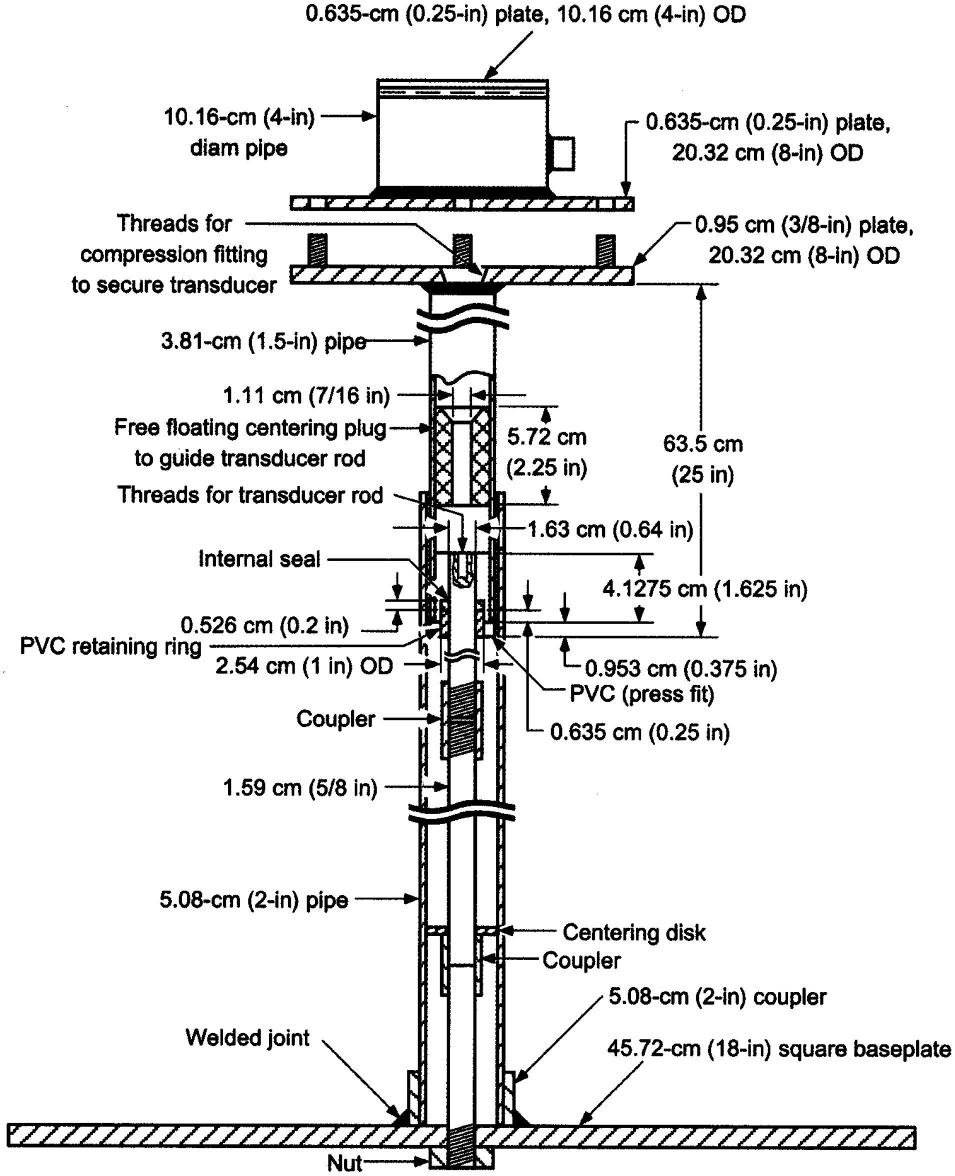


Figure 14.—Vertical backfill extensometer.

changes in instrument readings were not always related to a specific mining event. Compressive strain, which may be attributed to backfill shrinkage, was recorded by some of these instruments after the stopes were first filled.

To help prevent damage from surging or falling backfill, horizontal extensometers were fabricated at SRL with steel, rather than plastic, protective pipe. Steel-bearing plates at each end of the extensometer were also enlarged to increase the contact area between the instrument and the backfill. These instruments were placed directly on the floor of partially backfilled stopes and covered with wet backfill using a front-end loader. This practice protected the instruments from the weight of mining equipment and the large backfill loads that were subsequently placed over the instruments. These more robust extensometers recorded horizontal compressive trends from 73 to 145 days after installation, followed by relatively small tensional strain changes that were difficult to interpret [Tesarik et al. 1993].

CONCLUSION

Design and implementation of a successful instrumentation plan requires careful planning and consideration of site-specific factors, such as geology, identification of critical mechanisms and parameters, clearly defined objectives of measurements, estimates of the range of these measurements, types of transducers, ease of installation, how the sensors will be monitored and monitoring interval(s).

An overview of sensor technologies has been presented. Precision and the ability to sense sudden changes is a distinct advantage of Wheatstone bridge technology, but zero drift often makes it unsuitable for long-term monitoring. Vibrating-wire technology has been proven to provide reliable long-term measurements. Potentiometers have been used successfully to measure displacement.

The authors have summarized the use of several instruments and offered tips to help the user obtain better measurements. Also, the experiences of coworkers are cited. It is hoped that the information conveyed here will enable mine operators to make measurements on their own or improve on past measurement programs and that measurements obtained might be useful in making decisions that improve the safety of miners.

ACKNOWLEDGMENTS

The authors express their gratitude to John McRae of Geokon, Inc., Lebanon, NH, and Jeff Goalen of Campbell Scientific, Inc., Logan, UT, for reviewing this paper and offering corrections and suggestions for improvement.

REFERENCES

Amadei B [1983]. Rock anisotropy and the theory of stress measurements. Lecture Notes in Engineering No. 2. Brebbia CA, Orszag SA, eds. Berlin: Springer-Verlag.

Bauer ER [1985]. Ground control instrumentation: a manual for the mining industry. Pittsburgh, PA: U.S. Department of the Interior, Bureau of Mines, RI 9053.

Bickel DL [1993]. Rock stress determinations from overcoring: an overview. Denver, CO: U.S. Department of the Interior, Bureau of Mines, Bulletin 694.

Choquet P, Juneau F, Debreuille PJ, Bessette J [1999]. Reliability, long-term stability and gage performance of vibrating wire sensors with reference to case histories. In: Leung CF, Tan SA, Phoon KK, eds. Field Measurements in Geomechanics: Proceedings of the Fifth International Symposium on Field Measurements in Geomechanics—FMGM99 (Singapore, December 1-3, 1999). Balkema, pp. 49-54.

Dally JW, Riley WJ [1991]. Experimental stress analysis. 3rd ed. New York: McGraw-Hill.

Dunnicliff J [1988]. Geotechnical instrumentation for monitoring field performance. New York, NY: John Wiley & Sons.

Dunnicliff J [1999]. Systematic approach to planning monitoring programs using geotechnical instrumentation: an update. In: Leung CF, Tan SA, Phoon KK, eds. Field Measurements in Geomechanics: Proceedings of the Fifth International Symposium on Field Measurements in Geomechanics—FMGM99 (Singapore, December 1-3, 1999). Balkema, pp. 19-30.

Girard JM, McKibbin RW, Seymour JB, Jones FM [1997]. Characterization of in situ stress conditions at depth—Homestake mine, Lead, South Dakota (Abstract). Paper No. 104. Int J Rock Mech Min Sci *34*:3-4.

Hanna TH [1985]. Field instrumentation in geotechnical engineering. Trans Tech Publications, pp. 687-759.

Hooker VE, Bickel DL [1974]. Overcoring equipment and techniques used in rock stress determination. Denver, CO: U.S. Department of the Interior, Bureau of Mines, IC 8618.

Johnson JC, Larson MK, Brady TM, Whyatt JK, Langston RB, Kirsten H [1999]. Stress measurement of a jointed rock mass during drift development. In: Hagan TO, ed. SARES 99: 2nd Southern African Rock Engineering Symposium. Implementing Rock Engineering Knowledge (Johannesburg, Republic of South Africa, September 13-15, 1999). Int Soc Rock Mech, pp. 94-101.

Johnson JC, Pariseau WG, Scott DF, Jenkins FM [1993]. In situ stress measurements near the Ross shaft pillar, Homestake Mine, South Dakota. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9446.

Johnston JL, Cox DJ [1993]. Instrumentation procedures for fully grouted rock bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, IC 9341.

Larson MK [1992]. STRESsOUT—a data reduction program for inferring stress state of rock having isotropic material properties: a user's manual. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, IC 9302.

Larson MK, Maleki H [1996]. Geotechnical factors influencing a time-dependent deformation mechanism around an entry in a dipping seam. In, Ozdemir L, Hanna K, Haramy KY, Peng SS. Proceedings of the 15th International Conference on Ground Control in Mining. Golden, CO: Colorado School of Mines, pp. 699-710.

Larson MK, Stewart CL, Stevenson MA, King ME, Signer SP [1995]. A case study of a deformation mechanism around a two-entry gate-road system involving probable time-dependent behavior. In, Peng SS, ed. Proceedings of the 14th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 295-304.

Leung CF, Tan SA, Phoon KK, eds. [1999]. Management of instrumentation in geomechanics. In: Field Measurements in Geomechanics, Proceedings of the 5th International Symposium on Field Measurements in Geomechanics—FMGM99 (Singapore, December 1-3, 1999). Balkema, pp. 109-162.

Martin LA, Pakulnis R, Curtin RP [2000]. Field tests of cable bolts using instrumented king wires. Presentation at Canadian Institute of Mining, Metallurgy and Petroleum (CIM) 2000 meeting, March 5-10, 2000, Toronto, Ontario, Canada.

McKibbin RW [2000]. Personal communication, January.

McRae JB [2000]. Personal communication, May 5.

McRae JB, Simmonds T [1991]. Long-term stability of vibrating wire instruments: one manufacturer's perspective. In: Sørum G, ed. Proceedings of the 3rd International Symposium on Field Measurements in Geomechanics (Oslo, Norway, September 9-11, 1991). Balkema, pp. 283-293.

Merrill RH [1967]. Three-Component Borehole Deformation Gage for Determining the Stress in Rock. U.S. Department of the Interior, Bureau of Mines, RI 7015.

Obert L, Duvall WI [1967]. Rock mechanics and the design of structures in Rock. New York, NY: John Wiley and Sons, pp. 413-417.

Panek LA [1966]. Calculation of the average ground stress components from measurements of the diametral deformation of a drill hole. U.S. Department of the Interior, Bureau of Mines, RI 6732.

Pariseau WG [1986]. Research study on pillar design for vertical crater retreat (VCR) mining. Salt Lake City, UT: University of Utah. U.S. Bureau of Mines contract No. J0215043.

Röshoff K [1999]. Personal communication, December.

Savin GN [1961]. Stress concentrations around holes. New York, NY: Pergamon Press.

Serbousek MO, Signer SP [1987]. Linear load transfer of fully grouted roof bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9135.

Seymour JB, Tesarik DR, Larson MK, Shoemaker J [1998]. Stability of backfilled cross-panel entries during longwall mining. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 11-20.

Seymour JB, Tesarik DR, McKibbin RW [1996]. Stability of permafrost gravels in an Alaskan underground placer mine. In: Myrvang AM, Vik J, eds. Proceedings of the Fourth International Symposium on Mining in the Arctic (Longyearbyen, Svalbard, Norway, July 27-30, 1996). Sintef Rock Min Eng, pp. 147-156.

Seymour JB, Tesarik DR, McKibbin RW, Jones FM [1999]. Monitoring mining-induced stress changes with the biaxial stressmeter. In: Leung CF, Tan SA, Phoon KK, eds. Field Measurements in Geomechanics: Proceedings of the Fifth International Symposium on Field Measurements in Geomechanics—FMGM99 (Singapore, December 1-3, 1999). Balkema, pp. 55-60.

Signer SP [1990]. Field verification of load transfer mechanics of fully grouted roof bolts. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9301.

Signer SP [1994]. Field evaluations of grouted roof bolts. In: New Technology for Longwall Ground Control; Proceedings: U.S. Bureau of Mines Technology Transfer Seminar. Pittsburgh, PA: U.S. Department of the Interior, U.S. Bureau of Mines, SP 01-94, pp. 91-101.

Signer SP, Lewis JL [1998]. A case study of bolt performance in a two-entry gateroad. In: Peng SS, ed. Proceedings of the 17th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 249-256.

Signer SP, Mark C, Franklin G, Hendon G [1993]. In: Peng SS, ed. Comparisons of active versus passive bolts in a bedded mine roof. In: Peng SS, ed. Proceedings of the 12th Conference on Ground Control in Mining. Morgantown, WV: West Virginia University, pp. 16-23.

Tesarik DR, McKibbin RW [1989]. Instrumentation and modeling of the North 140 Section of Magmont Mine, Bixby, MO. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9215.

Tesarik DR, Seymour JB, Mudlin ME [1993]. Analysis of backfill and pillar performance in the B-Neath orebody at the Cannon mine. In: Glen HW, ed. Minefill 93, Fifth International Symposium on Mining with Backfill (Johannesburg, Republic of South Africa, September 1993). S Afr Inst Min Metall, Symp Ser S13, pp. 55-64.

Tesarik DR, Seymour JB, Yanske TR [1999]. Long-term stability of a backfilled room-and-pillar mine. In: Leung CF, Tan SA, Phoon KK, eds. Field Measurements in Geomechanics: Proceedings of the Fifth International Symposium on Field Measurements in Geomechanics—FMGM99 (Singapore, December 1-3, 1999). Balkema, pp. 431-435.

Tesarik DR, Seymour JB, Yanske TR, McKibbin RW [1995]. Stability analysis of a backfilled room-and-pillar mine. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9565.

Tesarik DR, Vickery JD, Seymour JB [1991]. Evaluation of in situ cemented backfill performance. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9360.

Walton RJ [1999]. Personal communication, November.

Worotnicki G, Walton RJ [1976]. Triaxial 'hollow inclusion' gauges for determination of rock stresses in situ. In: Proceedings of the ISRM Symposium on Investigation of Stress in Rock—Advances in Shear Measurement. Int Soc Rock Mech, pp. 1-8.

EVALUATION OF GROUND SUPPORT AT A TRONA MINE USING INSTRUMENTED CABLE AND REBAR BOLTS

By Lewis A. Martin,[1] Catherine A. Dreesbach,[2] and Rimas Pakalnis, Ph.D.[3]

ABSTRACT

Instrumented cable bolts developed at the Spokane Research Laboratory of the National Institute for Occupational Safety and Health were used in conjunction with existing ground control systems to monitor rock mass loads at Tg Soda Ash's trona mine, Granger, WY. Axial and shear loads were determined to levels of strain gauge accuracy of ±5 N or ±5 microstrain. These gauges were embedded in a remanufactured king wire that replaced the conventional king wire. Cable bolt performance, quality of grout, and installation techniques were also assessed. By using instrumented cables, a mine operator can determine axial load along the cable at predefined gauge locations. These data provide necessary input for an operator to design a safer working environment for miners.

[1]Mechanical engineer, Spokane Research Laboratory, National Institute for Occupational Safety and Health, Spokane, WA.
[2]Mine engineer, formerly with FMC Granger, Granger, WY.
[3]Associate professor, Department of Mining and Mineral Process Engineering, University of British Columbia, Vancouver, British Columbia, Canada.

BACKGROUND

Until recently, external measurement devices such as the "tensmeg" or resistance-wire cable strain gauges [Hyett et al. 1997; Windsor 1987] were used to determine load on cable bolts. Previously, researchers thought that an internal gauge would be too expensive to use with cables as an instrument within mines. Another issue has been that external gauges on cables may fail because of water seepage through the grout before the cable begins to take on load [Goris et al. 1993]. In 1996, the "stretch measurement to assess reinforcement tension" (S.M.A.R.T) cable was developed and patented by researchers at Queen's University in Canada [Hyett et al. 1997]. This system replaces the king wire of a cable bolt with multipoint extensometers enclosed in a tube to measure cable elongation, thereby enabling deformation and load to be determined [Bawden et al. 1998]. The usefulness of a ground control instrument having gauges mounted in the center of the cable or on the king wire was shown by Signer [1990] in research on the use of resin-grouted rebar bolts in coal mines.

In 1998, researchers at the Spokane Research Laboratory (SRL) of the National Institute for Occupational Safety and Health (NIOSH) developed an *instrumented king wire cable bolt* (patent pending). This instrument is manufactured by molding the king wire from epoxy around a flat piece of steel containing strain gauges. Instrumented cable bolts will aid in evaluating geologic conditions, grout quality, effective anchorage length, and cable load.

CONCEPT

The instrumented king wire cable bolt has 12.5-mm-long strain gauges embedded in the king wire at positions defined by the mine operator. Presently, 10 gauges can be installed along a resin-grouted bolt up to 5 m long or a cement-grouted bolt up to 7 m long. These constraints are largely due to current manufacturing processes and not to limits on the system. The gauges can either be set in pairs to detect shear load in the same plane, or the flat stock can be set at 90° angles to detect loads in three directions. The cables are capable of sustaining an ultimate load of 250 kN and can produce reliable readings to 180 kN, as shown by the calibration curve in figure 1.

To create an instrumented king wire cable bolt, strain gauges are first installed on both sides of a 2.4- by 0.5-mm metal strip at various user-specified positions on the bolt (figure 2). The individual strain gauges are connected to an instrument plug by 30-AWG wire. This apparatus is then placed in a steel mold and injected the total specified length of the bolt with a two-part epoxy. This new king wire measures 8.3 mm in diameter.

The original cable is uncoiled strand by strand, the original center wire is replaced by the new epoxy-filled king wire, and the outer six strands are rewound around it, resulting in an instrumented cable bolt. The gauge wires protruding from the center are then inserted into a 12-pin connector. This connector is recessed into a 4.4-cm hex head attached to the cable by means of a barrel-and-wedge assembly (figure 2). The cables are fitted with special heads so they can be inserted by either a jackleg drill or a roof bolter. An instrumentation plug is used to monitor the gauges. A data acquisition system is required to read the loads.

The system has been tested on a Vishay strain indicator box, a Campbell Scientific system, and an Omni Data system.

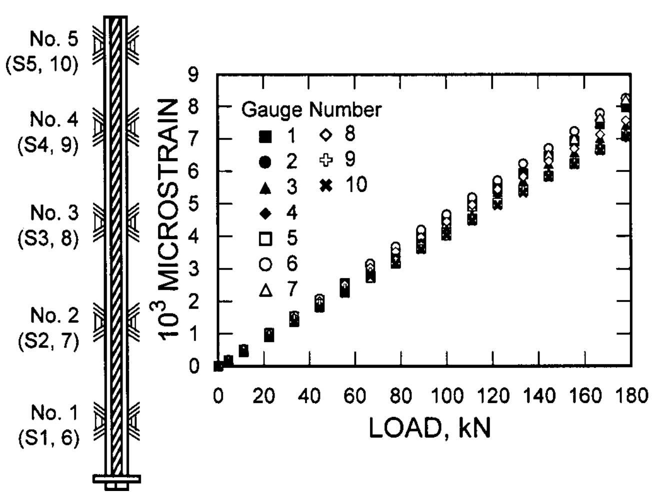


Figure 1.—Calibrated curve for instrumented king wire cable bolt.

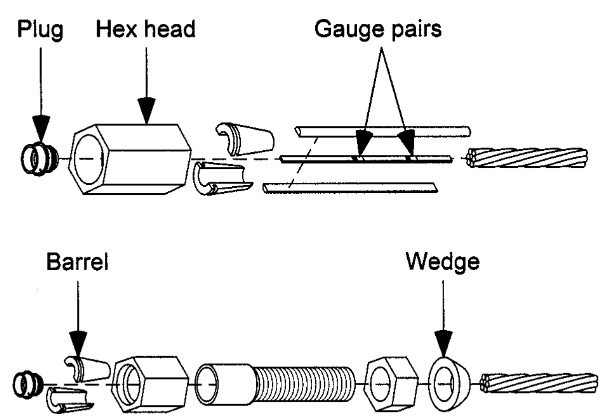


Figure 2.—Instrumented cable bolt assembly with and without pretension head.

MATERIAL DESCRIPTION

Standard cable specifications were the American Society of Testing Materials (ASTM) standard A416, 0.6-in, grade 270 K, low-relaxation, seven-strand cable having a minimum breaking load of 258 kN with 3.5% elongation. Upon removing the original king wire and replacing it with the manufactured one, ultimate load was reduced to 215 kN. Strain readings were reliable to 180 kN with 3.5% strain on the cable at failure. Calibration tests at SRL provided correlation of microstrain to load to enter into the data collection system (figure 1). This technique was used successfully throughout the elastic load-strain range on instrumented rebar bolts by Signer et al. [1997].

CABLE DESIGN

Laboratory tests were conducted on instrumented king wire cable bolts embedded with resin grout in concrete blocks to obtain calibration curves for converting microstrain to load in kilonewtons. Pull tests were also conducted to determine the load-carrying characteristics of the bolts. The strain gauges were mounted 0.15, 0.35, 0.61, 0.91 and 1.22 m from the head of the bolt (figure 3). The cables were loaded to 180 kN using hydraulic pull test equipment. A strain-to-load curve is shown in figure 3 and is typical of what Goris et al. [1994] observed in pull tests with conventional cable bolts.

These tests aid in explaining how load on cable bolts is transferred from the cable to the grout and then into the rock mass. A "shelling" effect is shown whereby the grout-cable interface breaks down and the gauge becomes debonded as load is increased and is taken fully by the cable. Figure 3 shows results of the pull tests as a load profile along the cable length. The curve shows that a 128-kN (~12.8-tonne) pull load on the collar of the cable resulted in measured strain loads of 123 kN along gauges 1 and 6 at a distance of 0.15 m from the collar. Researchers interpreted this measurement, a difference of 5 kN, as the amount of load the cable was not sensing 0.15 m from the collar. At 1.22 m from the collar, the cable did not sense any load, indicating that 1.22 m was the critical bond length. Other researchers (e.g., Goris et al. [1994]) have reported critical bond length to be approximately 1 m for standard cable. This load capability enables an operator to measure failure in real time as debonding of the grout-cable interface is initiated. In addition, the difference between cable load and applied load indicates grout quality, which can be a better field test of the quality of cable bolt installations than field pull tests.

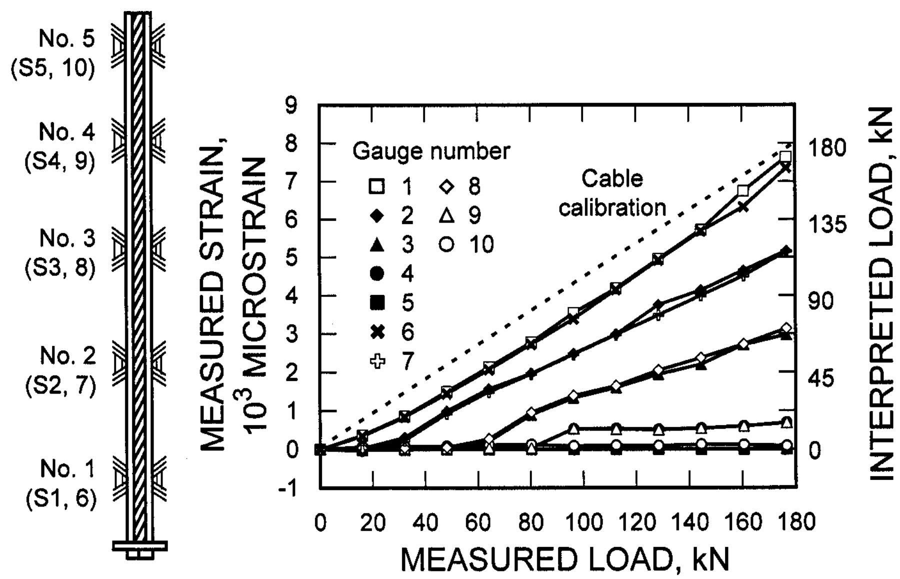


Figure 3.—Calculated load and strain on grouted cable bolt with instrumented king wire.

REBAR DESIGN

The resin-grouted rebar roof bolts used in these tests were standard, grade 60, No. 6 rebar that had been milled with a 6.35-mm-wide by 3.18-mm-deep slot along each side (figure 4). Axial load tests showed that the average yield load of the bolts was 96 kN and the average ultimate load was 158 kN. Before milling the slot, yield load had been 107 kN and ultimate load had been 176 kN. Thus, slotting caused a 10% reduction in strength [Signer 1990].

Strain gauges were installed into the slots as shown in figure 4. The strain gauges were then calibrated in a uniaxial test machine to correlate voltage change to load change. A typical load-strain curve from a tested bolt is shown in figure 5. When the data from the instrumented rebar bolts were reduced, the correlation coefficients from the axial calibrations were used to convert voltage changes to load changes. This process was accurate to ±22 N. When the load levels exceeded the yield point of the steel, voltage readings were converted to strain readings to estimate the degree of plasticity (figure 5).

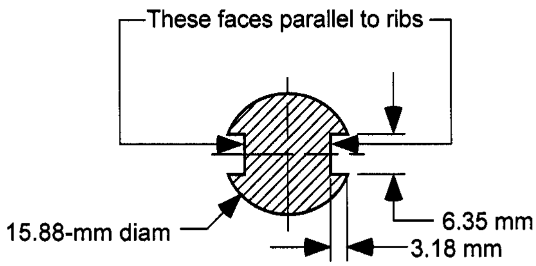


Figure 4.—Cross section of resin-grouted rebar bolt machined for installation of gauges.

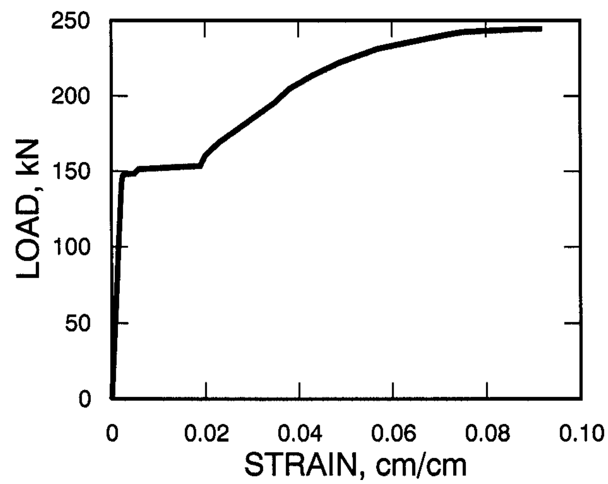


Figure 5.—Typical load strain profile for instrumented rebar bolt.

CASE STUDY: TG SODA ASH, FMC GRAINGER

MINE GEOLOGY

The mine is located in the Green River Basin of southwestern Wyoming. Trona deposits in the basin were formed during the Eocene period within the Wilkins Peak Member of the Green River Formation. The repeated evaporation and recharge of Lake Gosiute, a playa lake occupying the basin during Eocene time, created evaporite layers of trona intermixed with marlstone and alternating layers of oil shale [Leigh 1998].

The areal extent of the basin is marked by the Uinta Mountains in the south, the Rock Springs uplift in the east, the Wind River and Gros Ventre ranges in the north, and the Wyoming Range in the west. There are 42 known trona beds, of which 25 range from 240 to 670 m thick. These beds are numbered 1 through 25, according to the order in which they were deposited (bed 25 being closest to the surface). Beds 1 through 18 consist mainly of fine-grained trona; beds 19 through 25 are composed of a coarse crystalline trona. Mining at Tg Soda Ash takes place in bed 20 approximately 425 m below the surface. The area in which the field study was conducted is overlain by an additional 60 m of overburden.

MINING METHOD

Trona is mined with a drum-and-borer-type continuous mining machine. These machines, traditionally used for coal mining, are custom built to cut the harder trona ore. Tg Soda Ash uses only drum-type continuous miners. Cutting heads are typically 5.3 m wide, but the panel in which the field study was conducted was mined with a 5.6-m-wide cutting head.

The Tg Soda Ash mine uses a modified room-and-pillar mining method. In the field study area, the panel was developed as a four-entry system. An additional two entries were later driven parallel to the existing workings, with an eight-room retreat mined in a chevron configuration 78 m wide by 103 m deep with cross cuts on 12.2-m centers. The retreat consisted of pulling chevrons, a method in which pillars are cut on 12.2-m centers 60° from the direction of advance. This method maximizes the rate of extraction by minimizing ground control problems.

The field study site was chosen for its rapidly changing ground conditions and its proximity to mining activity in this panel. It is believed that the wider cutting head, extra overburden, and weaker geologic material of the roof and floor in the panel contributed to rapidly deteriorating ground conditions in the outside entries of the system, making the area ideal for a field study. During the course of the study, the face was approximately 215 m from the test area and retreated to within 45 m of the test area.

INSTRUMENTED BOLT INSTALLATION

For simplification in the rest of this paper, the term "cable bolt" means a bolt in which the conventional king wire has been replaced with the new remanufactured king wire.

Cable bolts and rebar bolts were installed in the roof of the mine to measure roof loads in the intersection of 350S and 12371E (figures 6 and 7). This location was chosen because of drill access ahead of the mining cycle and because loads would be induced in the bolts as ore was extracted toward the intersection. Mine personnel were interested in determining the potential for instability in the back and the extent of any unstable blocks.

Ground support was installed after development and during rehabilitation. Strain gauges were positioned along rebar and cable bolts at depths that would ensure that loads on the resin-grouted support would be measured. In addition, a set of gauges was mounted at an ungrouted section to measure dead-weight loads in the rock mass. Figure 8 shows the location of the strain gauges with respect to the collar. The longest cable bolts were installed at the center of the intersection because these bolts would receive the greatest loads, while shorter cable bolts were installed north of the intersection closer to the pillar front. Resin-grouted rebar bolts were placed in the south part of the intersection to measure loads where the density of rebar bolts was highest. Sagmeter stations were set at three locations in the intersection (figure 7), and holes were drilled to depths of 2.4, 3.2 and 3.8 m. These sagmeter depths were chosen to correspond with the bolt gauge profiles (figure 8), allowing correlations to be made between deformation recorded by the strain gauges to deformation recorded by the sagmeter stations. This is important because slip expected to occur between the cable-grout-rock interface can be compared to actual rock deformation.

RESULTS OF MEASUREMENTS ON INSTRUMENTED BOLTS

Loads on the cable bolts were measured from April 29 through July 11, 1999. Mining activities influencing the intersection progressed from northeast of the site to the south (figure 6). On June 9, development mining began within one pillar width (8 m) of the intersection. Figure 9 shows all load profiles measured by the 4.27-m-long bolts (bolts 1, 2, 4, and 5) and the 3.66-m-long bolts (bolts 6, 8, 9, and 11). Approximately the last 1.5 m of the bolt was bonded with resin grout. Figure 10 shows load profiles for the 2.44-m-long, fully grouted rebar bolts (bolts 201-204).

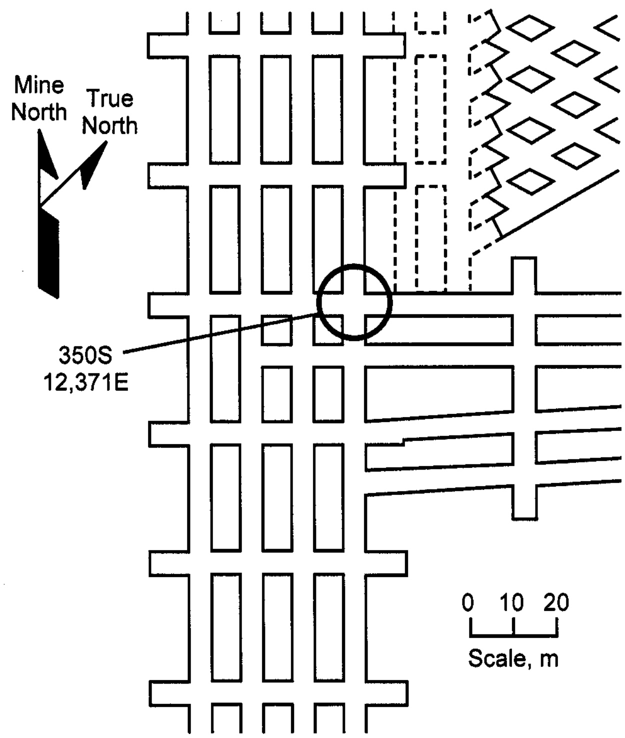


Figure 6.—Instrument locations in mine.

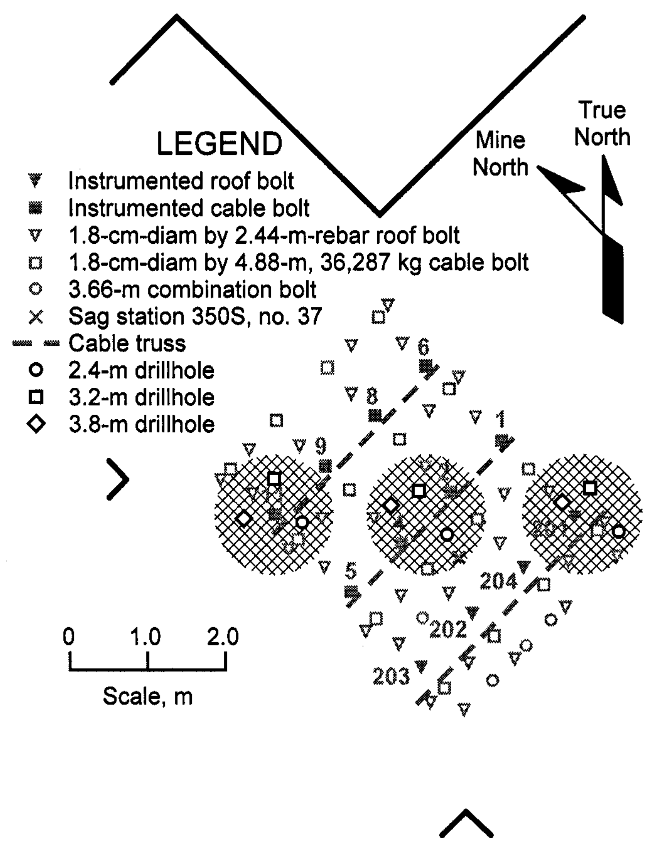
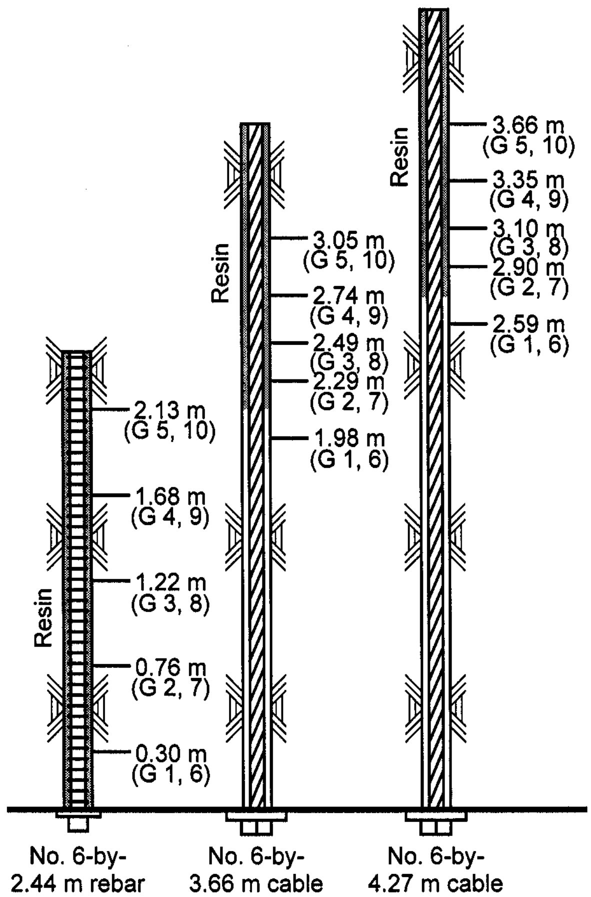


Figure 7.—Bolt and sagmeter locations.



Notes:

1. Assume 1.5-m resin anchor for cable bolts.
2. All gauges are in pairs at given locations.
3. Instrumented cable strength is 213.5 kN, yield at 177.9 kN.
4. Load is 11 kN times microstrain for cables.
5. Current inventory is: four no. 6-by-2.44-m rebar, four no. 6-by-3.66-m cable, four no. 6-by-4.27-m cable.

Figure 8.—Positions of strain gauges on bolts (note two at each position).

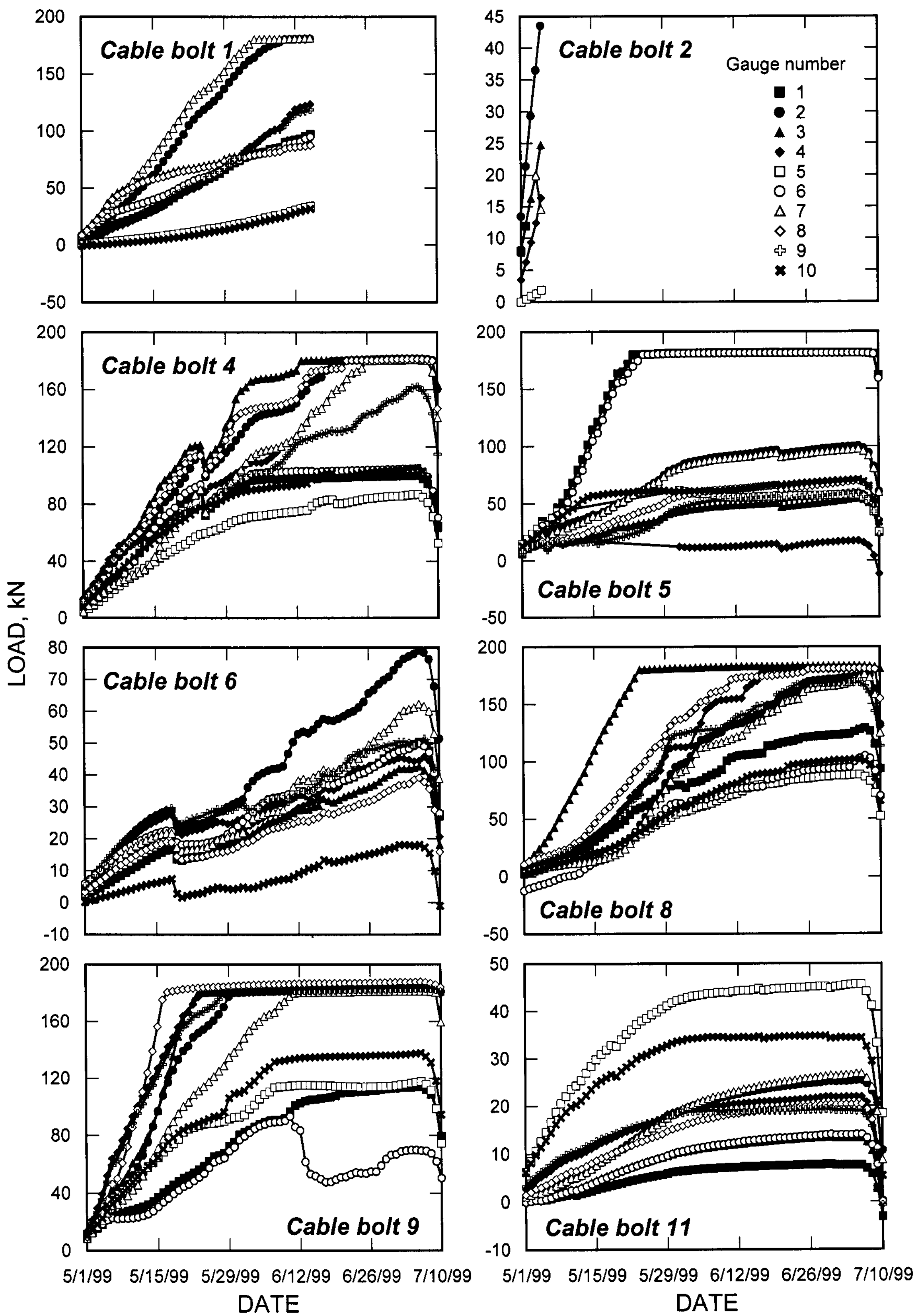


Figure 9.—Load profile versus time on 4.27-m-long bolts (1, 2, 4, and 5) and 3.66-m-long bolts (6, 8, 9, and 11).

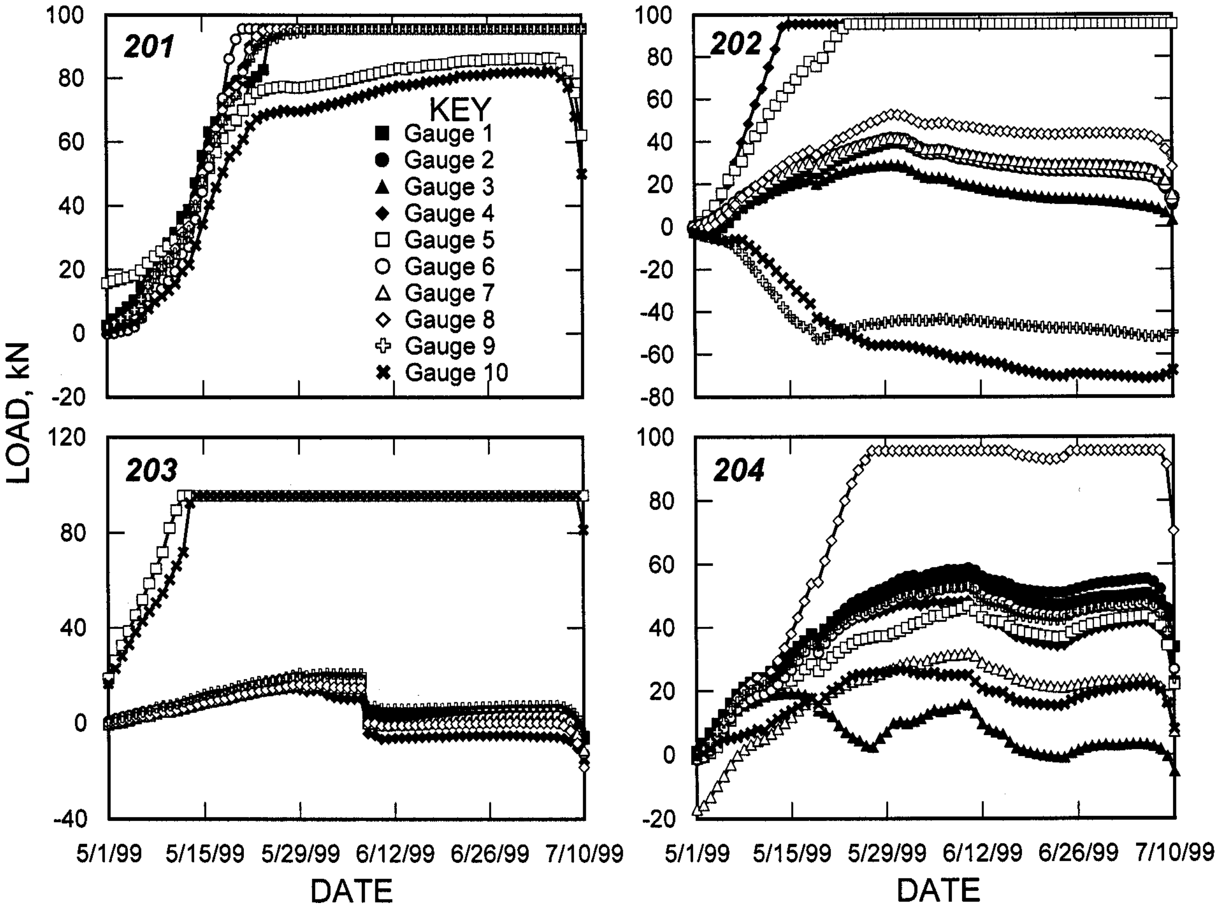


Figure 10.—Load profile versus time on a 2.4-m-long, fully grouted rebar bolt.

Following is the authors' interpretation of loads on 12 bolts. By recording actual point loads in the bolts, it will be possible to understand more readily the likelihood that instabilities may be forming or that there may be problems with the quality of the installation.

- Gauges 2 and 7 on cable bolt 1 indicated loads up to the yield limit of the steel. These loads were higher than the axial dead-weight loads on the ungrouted section of the bolt and loads on the grouted gauges, indicating that some delamination was occurring close to the gauge. The higher readings were most likely a result of the fact that concentrated load is measured over the gauge length by a strain gauge, whereas an ungrouted gauge measures all delamination cracks over the length of the ungrouted section, which was 2.8 m. The result is an average load much lower than the load on a bonded cable over a given distance.
- Bolt 2 was inspected by mine personnel, who determined that it had broken. This could have happened because the bolt was defective or because of unexplained shearing of the bolt.
- Bolt 4 shows a load-shelling profile similar to that generated in the laboratory (see figure 3). The greatest load was recorded 3.1 m above the back by gauges 3 and 8. The ungrouted length (2.8 m) shows a low load primarily because load was distributed over a large distance, as discussed previously. It must be noted that loads were lower on the gauges on either side of the bolt at 3.1 m. This implies that a crack was close by.
- Bolt 5 shows the classic load shelling profile. The highest load was recorded in the ungrouted section within 2.8 m into the back. The next highest load was at the bonded gauge 2.9 m into the back. Readings again indicate that delamination was largely controlled by cracks within 3 m into the back.
- Loads on bolt 6 were uniform, with an average of 40 kN, excepting for gauges 2 and 10, which were located 2.3 m above the back. This implies that delamination may have been occurring at this location. Note that this bolt was close to the pillar boundary and therefore the depth of the fractured zone would be less.

• Gauge sets 2 and 7 (2.3 m deep), 3 and 8 (2.5 m deep), and 4 and 9 (2.7 m deep) on bolts 8 and 9 showed a load profile approaching, and in most cases reaching, the yield strength of the bolt. This reading indicates that delamination was occurring throughout the horizon or that rock mass loading was higher. These two bolts were placed in the north-center section of the site, which experienced high loads such as those on bolt 4, which had also been placed in the center of the intersection. The deepest gauges (5 and 10) at 3.1 m into the back recorded low values, indicating that the bond at depths between 2.7 and 3.1 m was sufficient to absorb most of the load.

• Bolt 11 was located at a pillar boundary and therefore was receiving more support from the mine rock, resulting in less delamination and lower loads. Representative load curves for the 3.1-m-deep gauge show the greatest amount of load. The second set of gauges, at 2.3 m deep, took the next highest load, and the first set of gauges, at 2.59 m deep, took the lowest loads. Delamination was occurring, albeit at low loads, approximately 3 m above the back.

• Typical axial load curves are shown for bolt 201. These types of curves usually result when conducting laboratory pull tests on instrumented bolts. The curves also show a quick loading rate, which should be expected because the bolt was closest to active mining on the east side of the intersection.

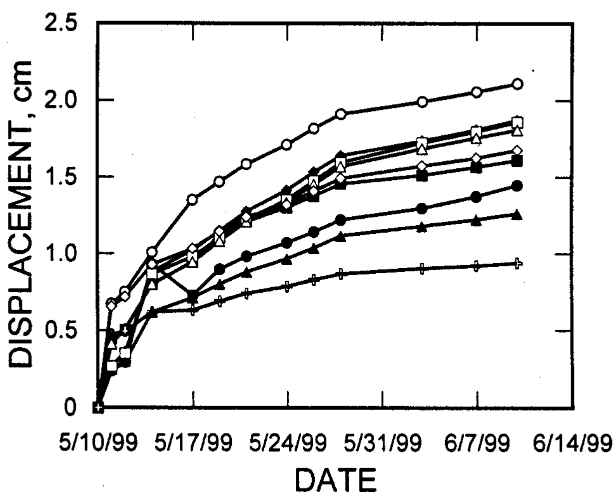
• Strain gauges on bolt 202 were reading both positive and negative strain, which indicate bending of the bolt. Gauges 4 and 9 and gauges 5 and 10 indicated that bolt bending on one side was in the yield zone of that bolt. Loads on gauge 5 showed the same loading horizon as the cable bolts at 2 to 5 m. The remainder of the gauges showed that the nonbending sections of the bolt were under low loads compared to the sections of the bolt showing bending.

• The tip on bolt 203 was undergoing high axial loads in the yield zone. This could be interpreted as an indication that the bolt was just reaching the load horizon at 2 to 3 m into the back. The rest of the gauges were being loaded at relatively low load rates.

• The load curve on bolt 204 was typical except for readings from two gauges. The bolt was supporting ground at all instrumented horizons, with somewhat higher loads at the center of the bolt. These load profiles also indicate that the resin grout was developing a good bond along the entire length of the hole. It is possible that a crack in the back caused grout delamination that affected gauge set 3 and 7. Total lower load readings along the bolt could have resulted from a greater bolt density in the surrounding area.

RESULTS OF MEASUREMENTS AT SAGMETER STATIONS

Three sagmeter stations were installed at three different depths (2.4, 3.2, and 3.8 m) at the intersection of 350S and 12,371E (figure 7). Figure 11 shows that sagmeters at the three stations generated similar curves with displacements ranging from 0.5 to 2.0 cm. In general, the most centrally located station had the greatest displacement at all three depths. Generally, the north-northwest and south-southeast sagmeter stations, which were near the boundary pillars, showed the least amount of deformation. Delamination was observed throughout all horizons with the largest amount of movement between 2.4 and 3.8 m of the back. This roof movement corresponded with data collected from the strain gauges, showing that delamination was generally 2.4 m above the back. Sagmeter readings from the 3.2-m-deep hole at the center of the intersection would be comparable to readings from a 4.3-m-long cable bolt with 1.5 m of resin embedment. This would mean 2.8 m of unbonded cable length.



	Location (relative to center of intersection)	Compare to __	Depth
■	SSE	2.4-m bolt	3.8 m
●	SSE	2.4-m bolt	3.2 m
▲	SSE	2.4-m bolt	2.4 m
◆	Center	4.3-m bolt	3.8 m
□	Center	4.3-m bolt	3.2 m
○	Center	4.3-m bolt	2.4 m
△	NNW	3.7-m bolt	3.8 m
◇	NNW	3.7-m bolt	3.2 m
✣	NNW	3.7-m bolt	2.4 m

Figure 11.—Displacement of back over time.

The above case history is being further evaluated with respect to deformation, mining sequence, and behavior of the back. The depth of failure was most likely 3 m. Employing a support pattern of bolts placed on 1.2- by 1.2-m spacings and a specific gravity of 2.6 for the marlstone-trona-shale back would yield a dead load of approximately 110 kN, which is within the range of loads as measured by the bolts. Although several of the measured loads exceeded the calculated dead load significantly, which could have been the result of mining-induced loading, readings from these cable bolts enable researchers to understand overall support-rock interaction.

CONCLUSIONS

In all installations where the gauges were read, cable bolts provided information about loads in the immediate rock mass. Grout quality could be determined by relating bonding capacity of these cable bolts to the load profile of the ungrouted bolts. The instrumented cable bolt was found to be reliable in indicating load changes and has been correlated to deformation of the adjacent rock mass.

The dangers of assuming that deformation is linear across a bonded cable were also suggested as the load profile showed the relationship of the degree of grout bonding to load along the cable. The simple relationship of strain-to-load calculated with cable modulus is only valid for the ungrouted sections of cable. However, the grouted sections of the cable should be calculated using calibrated data to determine the strain relationship. The above interpretation is only possible if point strains and loads on a cable are known.

This instrumented bolting system is used to aid in determining proper bolt spacings, bolt lengths, and load-carrying characteristics of the bolts, thus allowing a mine manager to design a safer roof support system for miners.

ACKNOWLEDGMENTS

We thank Tg Soda Ash and mine engineers John McDonnell, Christopher Pritchard, and Scott Lehman; without their cooperation, this research would not have been conducted. We also thank Stephen P. Signer, mining engineer at SRL, for his expertise in the application of Campbell Scientific systems; and Richard Curtin, Dennis J. Cox, Richard L. Rains and Joanne L. Johnston, engineering technicians at SRL, for their dedication in constructing the instrumented bolt systems.

REFERENCES

Bawden WF, Hyett AJ, Laush PM, Moosavi M, Degraaf P, Reust M [1998]. The S.M.A.R.T. cable bolt: results from two initial field trials. Presentation at the 100th annual general meeting of Canadian Institute of Mining and Metallurgy, Montreal, Quebec, Canada, May 3-7, 1998, 8 pp. Available from the author and on CD-ROM from the Canadian Institute of Mining, Metallurgy and Petroleum.

Goris JM, Brady TM, Martin LA [1993]. Field evaluation of cable bolt supports, Homestake Mine, Lead, SD. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, RI 9474.

Goris JM, Nickson SD, Pakalnis R [1994]. Cable support technology in North America. Spokane, WA: U.S. Department of the Interior, Bureau of Mines, IC 9402.

Hyett AJ, Bawden WF, Reust M, Pahkala M [1997]. The S.M.A.R.T. cable bolt: an assessment for the determination of tension in seven-wire strand cable bolts. In: Broch E, Myrvang A, Stjern G, eds. International Symposium on Rock Support: Applied Solutions for Underground Structures (Lillehammer, Norway, June 22-25, 1997).

Leigh RT [1998]. Wyoming trona: an overview of the geology. Laramie, WY: Wyoming State Geological Survey, Public Information Circular 40.

Signer SP [1990]. Field verification of load transfer mechanics of fully grouted roof bolts. U.S. Department of the Interior, Bureau of Mines, RI 9301.

Signer SP, Jones, SD [1990]. A case study of grouted roof bolt loading in a two-entry gate road. In: Peng SS, ed. Proceedings of the Ninth Conference on Ground Control in Mining. Morgantown, WV: University of West Virginia, pp. 35-41.

Signer SP, Cox D, Johnston J [1997]. A method for the selection of rock support based on bolt loading measurements. In: Broch E, Myrvang A, Stjern G, eds. International Symposium on Rock Support: Applied Solutions for Underground Structures (Lillehammer, Norway, June 22-25, 1997).

Windsor CR [1992]. Application of instrumentation in underground excavation: rock deformation and rock rein-forcement. In: Kaiser PK, McCreath DR, eds. Rock Support in Mining and Underground Construction: Proceedings of the International Symposium on Rock Support (Sudbury, Ontario, Canada, June 16-19, 1992). Balkema, pp. 349-365.