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FORUM ON ROOF BOLTING

(Annual General Meeting, Edmonton, April, 1963)

The Forum on Roof Bolting, a panel type of presentation organized by the C.I.M.'s Coal Division, was held on the afternoon of April 2, 1963, at the Annual General Meeting in Edmonton. Mr. W. J. Riva, Chairman of the Coal Division, introduced Mr. D. F. Coates, moderator, to a large audience. Mr. Coates observed that the time was late, declared the forum open and called upon Mr. T. S. Cochrane, of the Fuels and Mining Practice Division, Mines Branch, Ottawa, for the first paper.

Pull Tests as a Measure of Roof Bolt Efficiency and of Roof Bolt Design†

By T.S. COCHRANE* and F. GRANT

ABSTRACT

A variety of practical tests have been tried in the mining industry to determine the support properties of mine roof bolts. Such tests include torsion wrench readings, hammer blow tests and the use of plates with built-in tension indicators. This paper gives some details of pull tests with a hydraulic jack, as carried out in coal mines of western Canada.

MANY criteria exist for establishing the efficiency of roof bolting, and the one adopted depends on the nature of the problem and on the individual leanings of the investigators. Consequently, sufficient studies have not been made from a common viewpoint or under similar geological and technological conditions to compare these methods of evaluation. However, regardless of whether certain tests provide sufficient factual data to form a basis for theoretical research on bolting or not, many of these tests can and do provide a positive means of judging the suitability of chosen roof bolting techniques and equipment under definite geological conditions.

Such a test is the short-term anchorage or pull test, carried out to determine the anchorage strength of an assembly. This test, if properly executed and interpreted, can effect considerable savings in time and money during the introduction of roof bolting to any set of conditions. It is appreciated that the time-dependent function of the anchorage dictates success, but, in the experience of the authors, if the anchorage cannot sustain short-term forces it will not sustain long-term forces. On the other hand, if the anchorage passes the short-term pull test, it invariably will pass the long-term test unless some unusual rock behaviour, requiring material strength studies to clarify, is involved.

The short-term pull test consists of subjecting an installed anchorage and bolt, devoid of bearing plate and end nut, to a direct pull by means of a hydraulic pump and a hollow-bodied ram. The load applied to the bolt and anchorage is read from the gauge on the hydraulic pump, and the yield or travel of the anchorage and bolt is measured by an extensometer set for the purpose. The load-yield data is recorded for definite increments of load or yield up to failure of the assembly, which may be characterized by breaking of the bolt or by the exceeding of a predetermined yield limit. For ease in interpretation, this data is plotted as a load-yield curve.

The plots from the pull tests provide information on significant requirements for good anchorage, namely:

1. the seating of the anchorage device;
2. the suitability of the rock — the matching of the rock and anchorage device; and
3. the suitability of the bolting procedure.

Ideally, the anchorage should sustain the yield load of the bolt at an anchorage displacement or travel which is less than the maximum allowable deflection of the roof. Misseating of an anchorage device, mismatching of the rock and anchorage device, or a faulty hole drilling or installation technique will allow a greater percentage of anchorage displacement for an increment of load, and will often prevent bolt strength, or an acceptable percentage of bolt strength, from being developed before the anchorage displacement has exceeded the maximum allowable roof deflection.

A few characteristic pull test results can be presented to illustrate these points. Figures 1 and 2 show load-yield curves taken from the literature for illustrative purposes.* Figures 3 to 5 are the results arising from tests carried out by Mines Branch personnel at a property in western Canada.

†Fuels and Mining Practice Division, Mines Branch, Internal Report FMP 63/52-MIN.

*Acting Head, Mining Research Section, Fuels and Mining Practice Division, Mines Branch, Ottawa.

*Bolting Experience, by Messrs. Middendorf and Janssen, "Gluckauf," 89.33/34, 15 August, 1953.

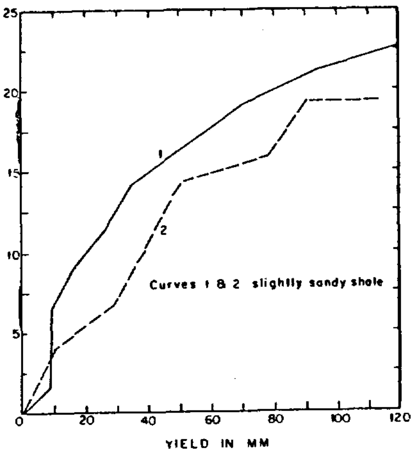

Figure 1.

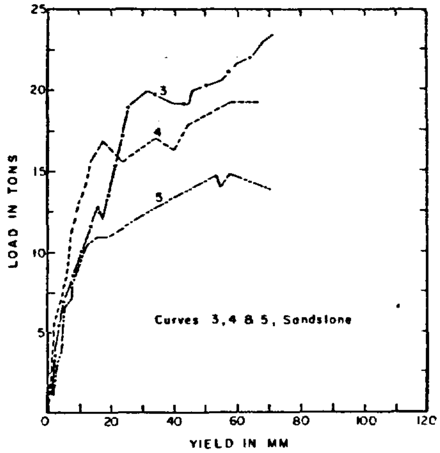

Figure 2.

Examples of load-yield curves.

In Figure 1, the flatness of the initial part of curve 1 draws attention to the effects of improper setting of the anchorage. The remainder of the curve shows the rock-shell relationship in dry-drilled sandy shale. Curve 2 of Figure 1 shows the obvious detrimental effects of wet drilling under these conditions.

In Figure 2, the curves illustrate the importance of bolt dimensions and material stretch. Curves 3 and 4 are drawn for bolts of the same diameter, anchored with similarly designed shells, but of different lengths (4 and 5 feet). Note the flattening of curve 4 at a lower load because of the additional material stretch in a longer bolt. Similarly, curves 3 and 5 compare bolts of the same length but of different diameters. The effect of the smaller diameter, as shown in curve 5, is apparent.

The next series of curves are drawn from pull tests conducted in a mine in western Canada to determine if a proper shell-rock match could be obtained with existing commercial shells. In this instance, a company had purchased a complete bolting unit and a quantity of roof bolt assemblies, complete with two types of anchorage shells, on a supplier's recommendation. The company personnel had bolted over 500 feet of roadway when the Mines Branch was asked to check the installation. A simple torque test was made on 111 of the installed bolts, with the following results. Only seventeen (or 15.3 per cent) of the bolts tested had retained 50 per cent of the installed torque. Twenty-one bolts (or 19 per cent) of the total tested were loose, with bearing plates free. A torque of over 100 ft.-lbs. was recorded in six (or 5.5 per cent), and only one bolt showed an increase in torque.

Because of these results, a series of pull tests was conducted on bolts installed for the purpose. The upper curves in Figure 3 are representative of the results obtained from assemblies installed and tightened under mine installation procedure. It is quite apparent that either the shell and rock were a complete mismatch or the hole was too large in diameter. Under very closely controlled drilling and installation conditions, the anchorage capacity of 4-foot bolts only approached 40 per cent of the bolt strength. Setting this anchorage device in a more competent bond at a 2-foot depth

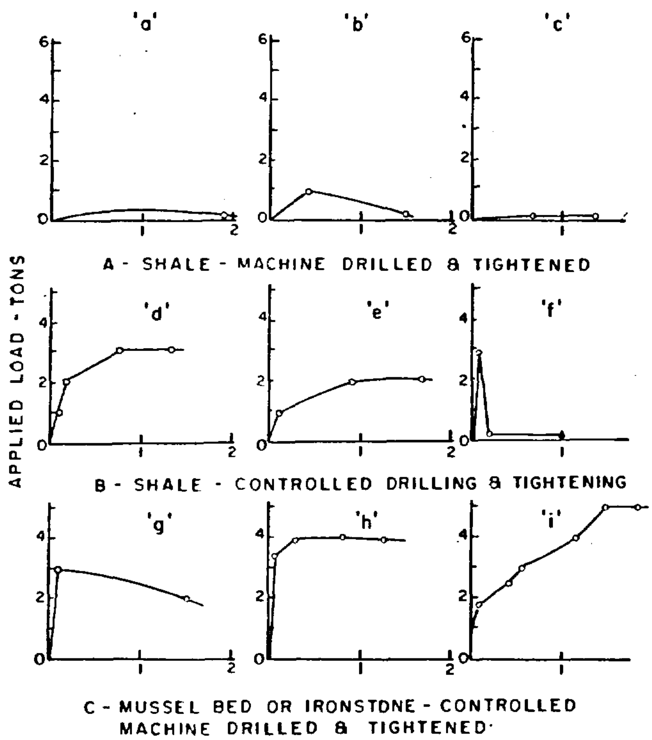

Figure 3.—(above)—Graphs of load vs. yield for the bail-type expansion shells.

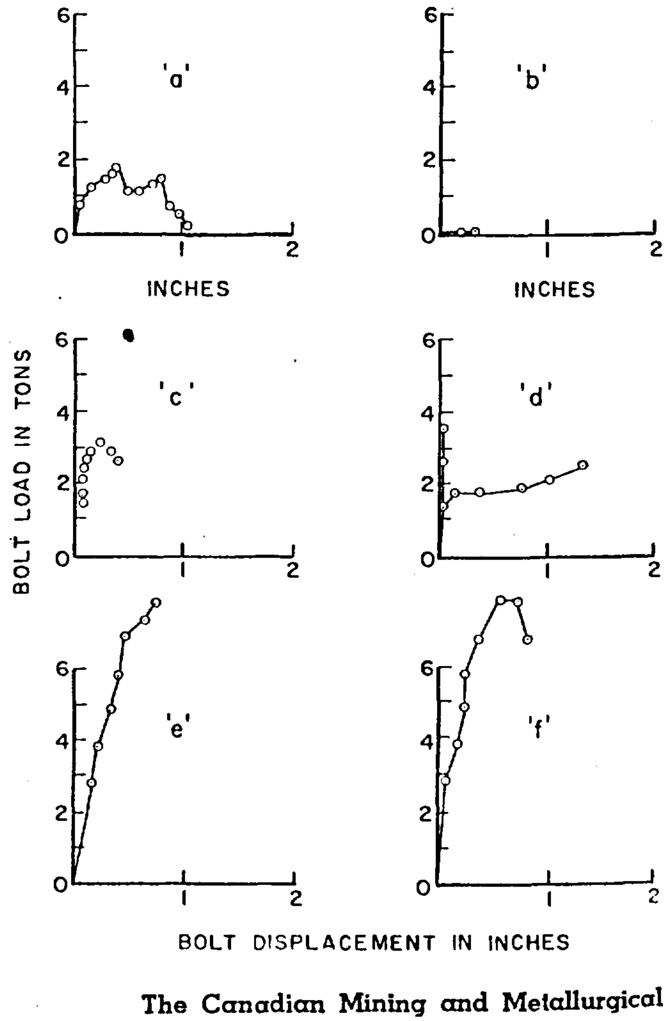

Figure 4.—(right)—Graphs of load vs. yield for the one-piece type of expansion shell.

brought about little improvement in the results (G curves). It was apparent that the shell and rock were not matched and that this means of anchorage was not suitable.

A similar series of tests conducted using the second type of anchorage showed comparable results (Figure 4). Again, under stringent installation conditions, an anchorage capacity of a little more than 50 per cent of bolt strength was developed, as shown in curves e and f of Figure 4.

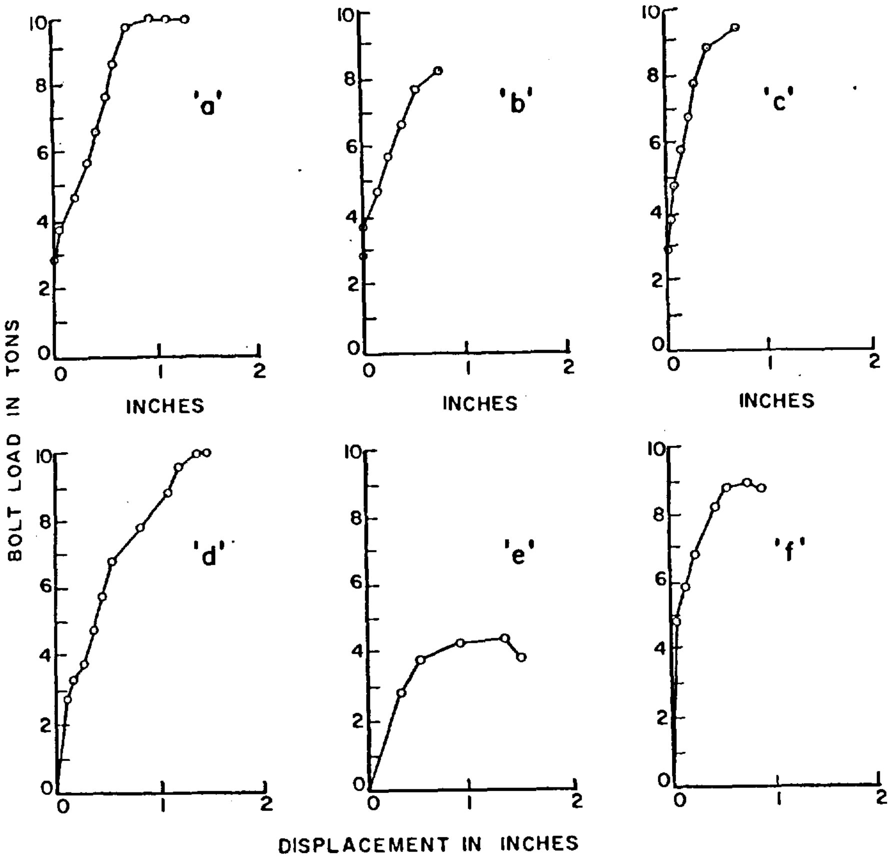

Figure 5.—Graphs of load vs. yield for miscellaneous shells.

A selection was then made of a number of miscellaneous commercial shells, and these were subjected to pull tests. Representative plots for each type selected are shown in Figure 5. Four of the six shells tested were the bail type, and had certain features in common: — well-developed serrations that were favourably orientated, a positive expansion mechanism and parallel expansion. As can be seen, these shells developed anchorage capacities that were 61 to 85 per cent of the bolt strength at the allowable deflection of 0.15 inch. The other two one-piece conical expansion shells were not as effective under these ground conditions. Table I summarizes the results of these tests.

As a consequence of the tests, a clearer understanding of the requirements for good anchorage in this ground was obtained. It was apparent that the anchorage unit, in order to be effective, must embody the following features:

depth of serration — + 3/32 inch
angle of serrations — normal to hole axis
expansion ratio — 1 : 1.3
expansion type — parallel
effective bearing length — 1⅞ inches, preferably 4¼ inches.

An anchorage shell was designed to conform to the above specifications. Unfortunately, however, the mine closed before the shell could be field tested.

TABLE I

SUMMARY OF ANCHORAGE RESULTS

Test No.	*Type of Anchorage*	*Rod Length*	*Setting Torque*	*Jack-Load*, in pounds, *at Initial Yield* (Displ. – 0.15 in.)	*Displacement at Initial Yield* — inches	*Maximum Jack-Load* pounds	*Displacement at Maximum Jack-Load*, inches	*Remarks*
Figure 3								
a	bail	4	120-150	—	—	—	—	No resistance, cont. slip
b	”	4	100	1000	.15	2000	.40	
c	”	4	100	250	.15	1400	.70	Little resistance
d	”	4	130	2000	.10	6000	.75	
e	”	4	120	2000	.10	4000	.90	
f	”	4	100	6000		6000	.10	
g	”	3	120-150	6000	.10	6000	.10	
h	”	2	130	7000	.07	8000	.30	*Reset
i	”	2	130	4000	.10	10000	1.45	
Figure 4								
a	one-piece	4	110	2600	.15	4000	.40	
b	” ”	4	110	250	.15	500	.30	Little resistance
c	” ”	4	110	6000	.15	6400	.20	
d	” ”	4	120	3600	.15	3600	.15	
e	” ”	4	140	6000	.15	19400	1.00	Erratic at 13,600,0.45 in.
f	” ”	4	130	6800	.15	15600	.55	
Figure 5								
a	bail	4	140	8600	.15	20000	.75	
b	”	4	140	9600	.15	16600	.75	
c	”	4	140	11600	.15	18800	.70	Bolt broke
d	one-piece	4	130	6600	.15	19600	1.40	
e	” ”	4	130	3000	.15	8800	1.35	
f	bail	4	130	10600	.15	18000	.75	

Roof Bolt Anchorage at Michel Colliery

*By L.M. DWARKIN**

ABSTRACT

The use of roof bolting gave promise of substantial economic benefits to Continuous Miner operations at Michel Colliery. As the first step in the adoption of bolting, 142 anchorage tests, employing six types of anchor shells, were made in three mines. Significant differences were found in the anchorage capabilities of the various shells and the various roof rocks.

Introduction

SPORADIC attempts at roof bolting had been made at Michel Colliery over a period of ten years. Results were indifferent or inconclusive. A concerted effort was not made, because a compelling need for bolting did not exist. By 1962, however, the advent of Continuous Miners had brought a new significance to roof bolting and, at the same time, the increasing price and scarcity of mine timber made bolting materials more competitive in cost. A fresh appraisal of bolting, therefore, became justified and necessary.

The potential benefits of roof bolting at Michel are several. The first is in the utilization of production time. The Continuous Miners can mine up to 6 tons of coal per minute, so every second is precious and it is imperative that all obstacles to maximum production be removed. Mining conditions at Michel dictate that the permanent roof support be installed concurrently with coal removal. The present timbering methods take up 30 to 40 per cent of the mining cycle, so any gain in that area is extremely valuable. Bolting gives promise of such a gain.

The length of Continuous Miners creates severe problems at turnouts and intersections. Present timbering consists of 3-piece sets, having two rib posts and a cross piece. Starting a turnout requires the removal of several posts at the rib affected, the emplacing of new posts and the substitution of bridge timbers. To resume work in the original heading, the timbering must be changed again. This laborious, slow and costly procedure could be eliminated entirely by roof bolting.

Timbers, particularly posts, restrict the activity of mobile equipment. One instance is the hindrance to cleanup operations by Miners and Loaders.

A substantial advantage of bolting involves the reduction in materials handling and storage effort due to the lesser bulk and weight of bolting materials.

The final gain, and possibly the most important, is the expectation of improved roof control from bolting.

The Bolting Program

Before embarking on a new bolting program, past experience was reviewed. In retrospect, the previous lack of success could be attributed to the absence of a scientific approach and procedure. To avoid repetition of the error, the help of a consultant (Mr. D. F. Coates) was obtained, and, under his guidance, a program was outlined.

The first phase of the program involved a determination of the anchorage capacity of roof bolts. This has been the accomplishment to date. Work was begun in the "A" North mine with the selection of three sites encompassing the proposed bolting area. Drill cores of the roof rock were taken and sent to the laboratory of the Mines Branch in Ottawa. Their tests indicated good anchorage capability, but, unfortunately, subsequent investigation revealed that the selected sites were not fully representative of the mine. A site having a much poorer roof was found, and work was concentrated there in an effort to meet this worst condition.

Work was also done in the "A" West mine and the No. 1 Seam mine. Eventually, a total of 142 tests, employing six types of anchor shells, were carried out in the three mines.

Anchorage Tests

Anchorage was tested by installing bolts in the roof and pulling them with a hydraulic jack. Progressive readings of the applied load and the bolt displacement were taken and plotted on a graph. The latter revealed the character and effectiveness of the anchorage.

The Mines Branch, who have been exceedingly helpful throughout the program, loaned us the test equipment that was used.

Anchor Shells

Figure 1 shows the various anchor shells that were subjected to test. They can be classified into two main groups. The bail type (A to D) suspend the shell from the top of the bolt by means of a bail. The prong type (E and F) support the shell from below by a strippable nut or by lugs. The bail shells tend to remain parallel on expansion; the prong type spread on top, forming a "V".

Another classification involves the number of faces per wedge. Shell A has a single face, shells B and C have two, and shells D, E and F have four.

Shells also differ in dimensions, area of contact between shell and rock, and the type and extent of serrations.

*Chief Engineer, Crow's Nest Pass Coal Company, Fernie, B.C.

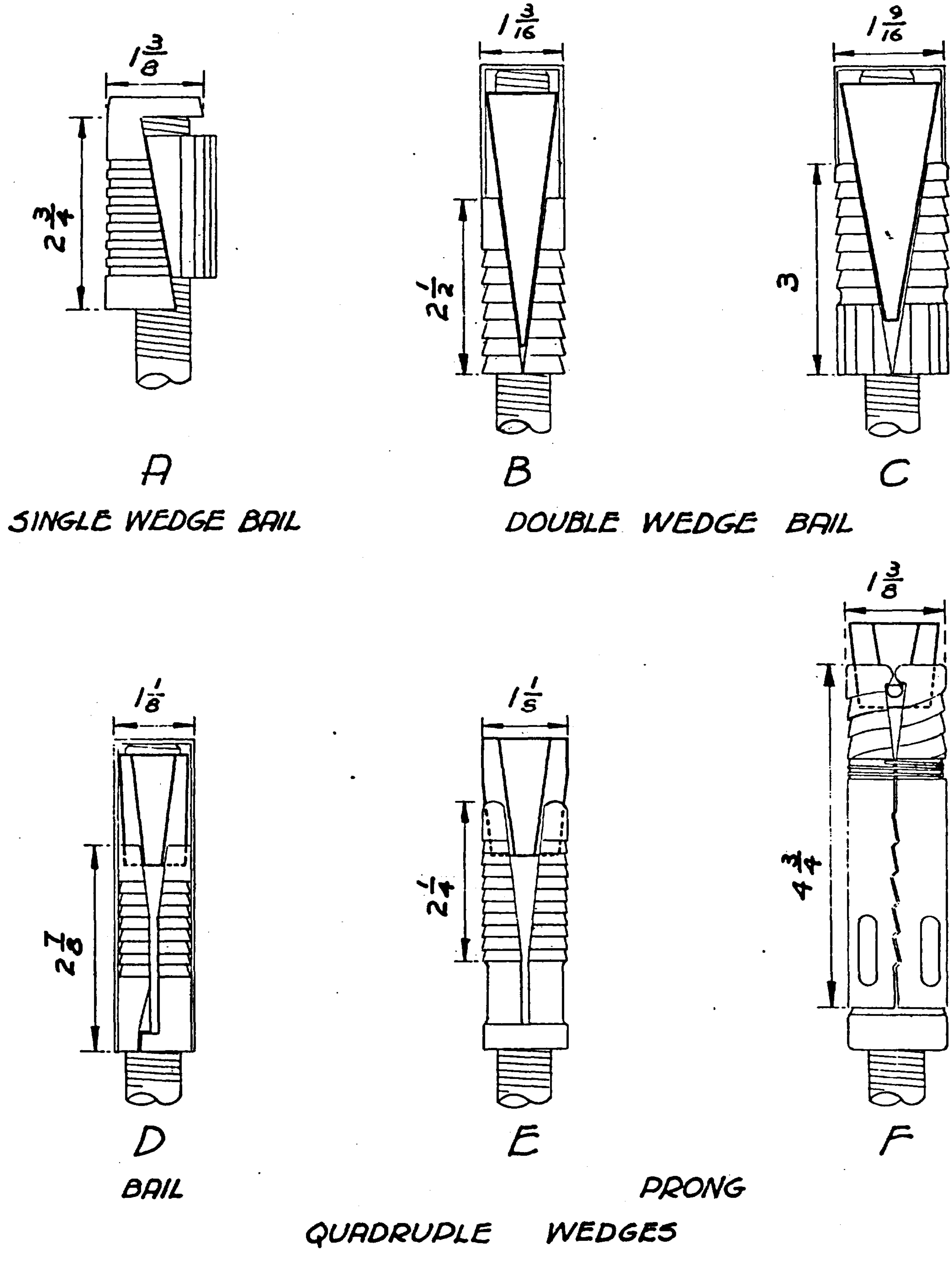

Figure 1.—Various Roof Bolting Shells.

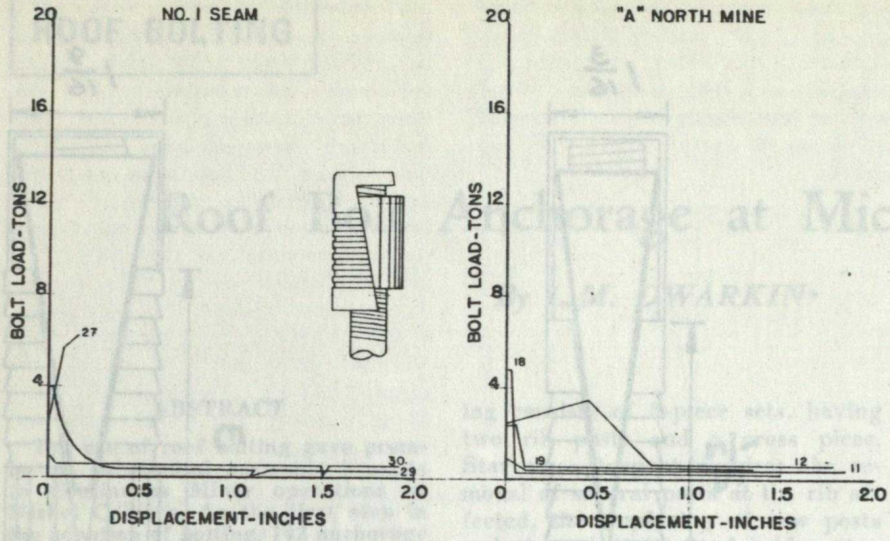

Figure 2.—Anchorage curves for shell "A".

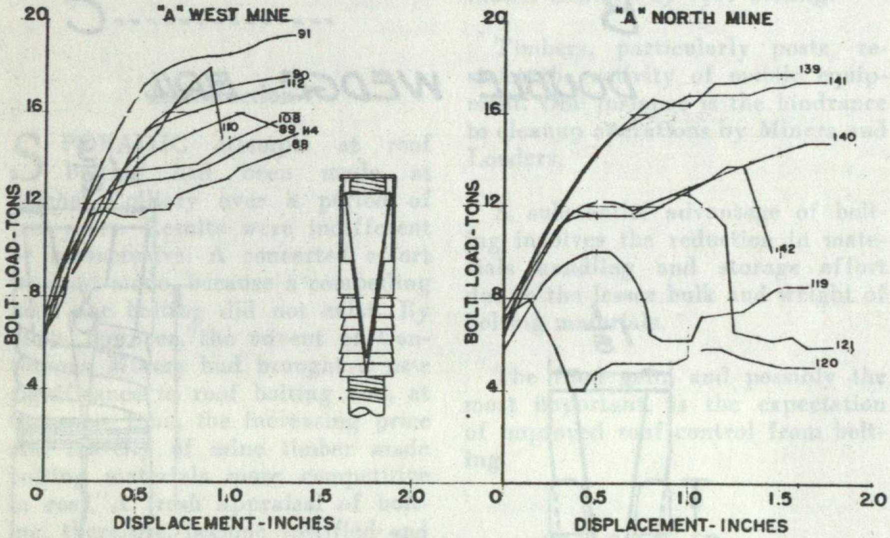

Figure 3.—Anchorage curves for shell "B".

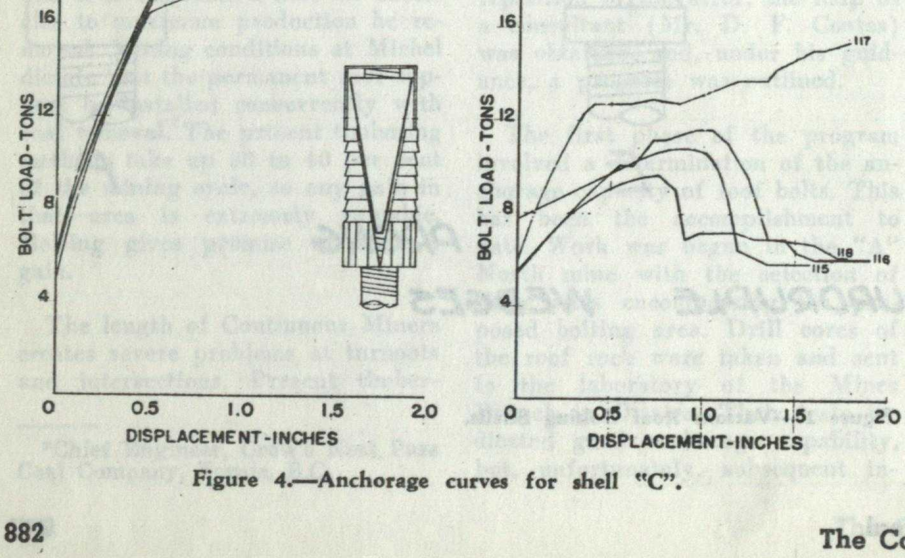

Figure 4.—Anchorage curves for shell "C".

Anchorage Curves

Figure 2.—Shell "A" had the worst anchorage of all the shells tested. It gave poor results both in the soft roof of the "A" North mine and in the hard roof of No. 1 seam. The poor showing may be due to inadequate serrations.

Figure 3.—Shell "B" did very well in the hard rock of the "A" West mine. Its capacity varied from 14.3 to 19.2 tons, with an average of 17. In "A" North, it was inconsistent and gave results ranging from very good to fair.

Figure 4.—Shell "C" gave exceptionally high and consistent anchorage in "A" West. The anchorage exceeded the strength of the ¾-inch, high-tensile bolts. In "A" North, however, its performance was only fair.

Figure 5.—Shell "D" had an anchorage that was fairly good in "A" West but mediocre in "A" North.

Figure 6.—Shell "E" behaved in the opposite manner to the previous shells and gave best results in "A" North. Its performance was not too consistent in either mine.

Figure 7.—Shell "F" was found to be the best of all for the "A" North mine. Anchorage was high and quite consistent. Seven out of eight tests gave very similar results. In "A" West, performance was also good.

Figure 9.—Here, average curves for the various shells are compared. In the hard roof of "A" West, the bail type of shell did well. Those with the largest contact area gave the best anchorage. The small-prong shell was poorest; the large-prong shell was intermediate.

In the softer roof of "A" North, the situation was reversed. The large-prong shell was best and the small-prong shell was second. The bail shells gave relatively poorer results.

Anchorage Theory

Figure 8 shows a typical curve for good anchorage. The form of the curve is intriguing, and the following is suggested in explanation of the configuration.

Point A is obviously the load applied to the bolt during installation. This preload is related to the torque used during tightening. In our tests, 225 foot-pounds of torque resulted in a preload of 4½ to 7 tons.

From A to B, the wedge is being forced into the slot and the load

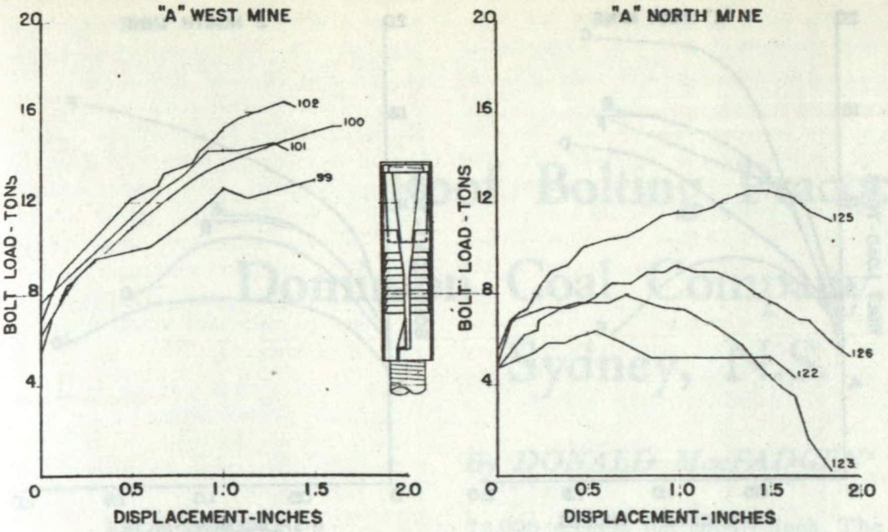

Figure 5.—Anchorage curves for shell "D".

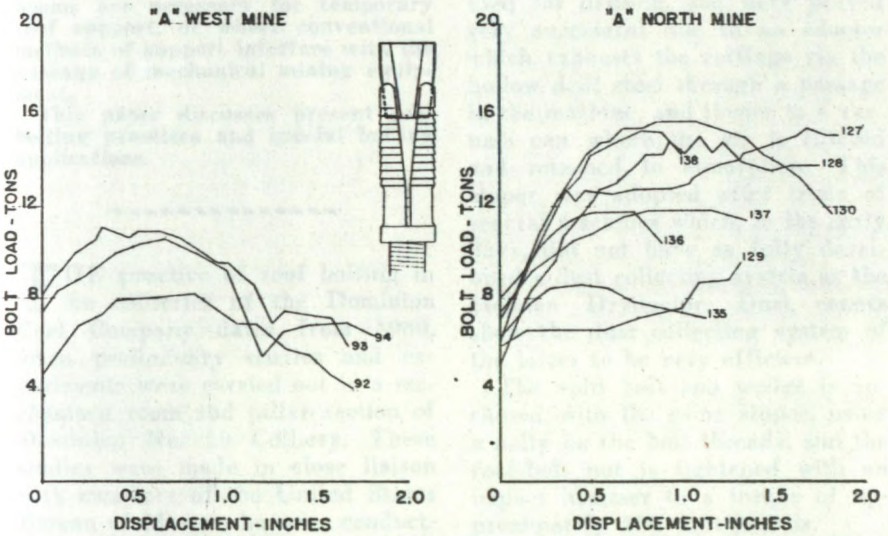

Figure 6.—Anchorage curves for shell "E".

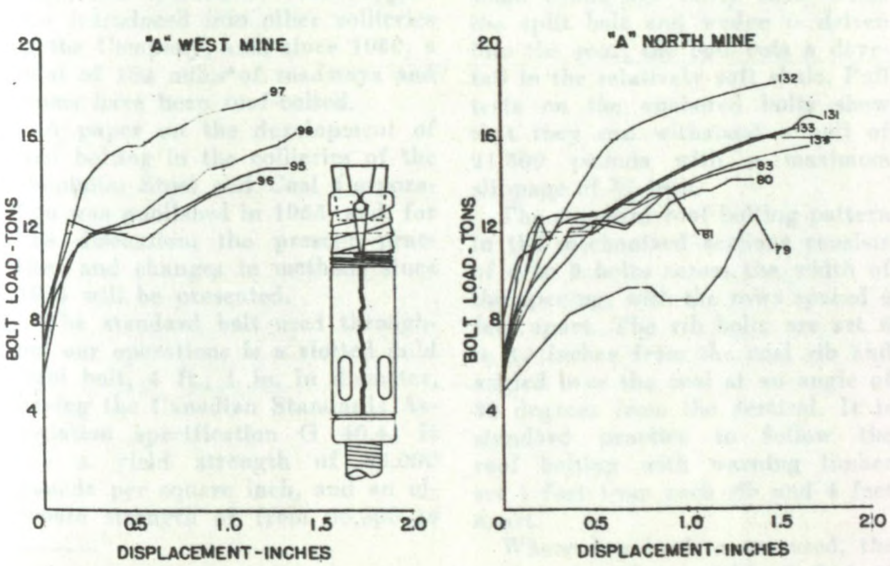

Figure 7.—Anchorage curves for shell "F".

rises because of the increasing resistance of the rock to shell expansion. The slope of the curve depends on the bluntness of the wedge, the area of contact between shell and rock, and the modulus of elasticity of the rock. The displacement is due to the downward motion of the wedge and also to the elongation of the bolt under load.

At point B, the curve flattens due to slippage of the shell down the hole, or to the yielding of bolt steel, or to both.*

From C to D, the load is removed and some of the displacement is recovered. This recovery is due to one or both of two agencies:—the springing of the wedge back out of the slot, and elastic contraction of the bolt. All of the displacement cannot be regained, because the bolt has been stretched beyond its yield point and also because any slippage of the shell in the hole is permanent.

From D to E, the load is reapplied to prove that the maximum load obtained is genuine and not caused by hang-up of the shell on a rock shelf or by some other extraordinary circumstance. The hysteresis loop from C to D is formed principally because friction opposes both the loading and unloading of the bolt.

From E to F a small additional load causes large displacement. The bolt anchorage has reached its limit here.

*The normal inflection point of the curves should be at 12 tons, corresponding to the yield point of the bolts used. Some of the bolts were strain-hardened by re-use, resulting in higher inflection points.

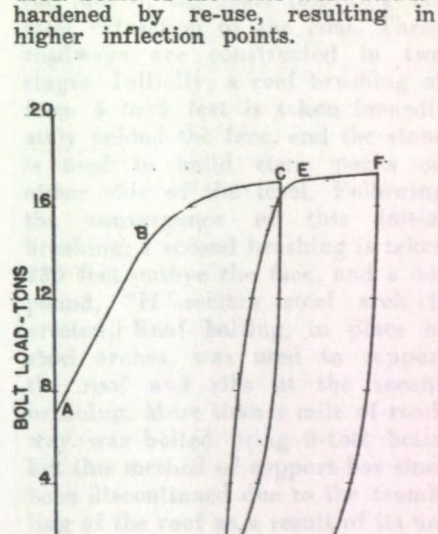

Figure 8.—A typical curve for good anchorage.

Conclusions

The anchorage tests have proven the following:

(1) The anchorage capability of the mine rock varies widely, not only from one mine to another but also in different locations of the same mine.

(2) The available anchorage shells differ significantly in their absolute anchorage ability as well as in their behaviour under different rock conditions.

The conclusion, therefore, is that a suitable shell must be found for each individual case. Although theoretical considerations can serve as a guide, the final choice must be made from actual testing.

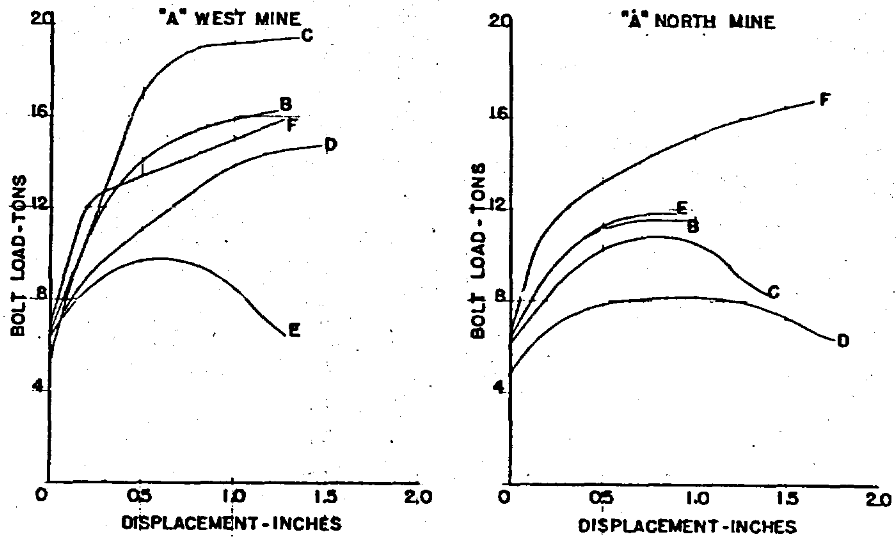

Figure 9.—A comparison of the average curves for the various shells.

Roof Bolting Practices, Dominion Coal Company, Limited, Sydney, N.S.

*By DONALD MacFADGEN**

ABSTRACT

Since 1950, a total of 164 miles of levels and rooms have been roof-bolted in the collieries of the Dominion Coal Company. Roof bolting is an efficient and economic method of support where heavy timber sets or steel booms are necessary for temporary roof support, or where conventional methods of support interfere with the passage of mechanical mining equipment.

This paper discusses present roof bolting practices and special bolting applications.

THE practice of roof bolting in he collieries of the Dominion Coal Company dates from 1950, when preliminary studies and experiments were carried out in a mechanized room and pillar section of Dominion No. 20 Colliery. These studies were made in close liaison with members of the United States Bureau of Mines who were conducting similar studies.

Following the successful application of roof bolting in No. 20 Colliery, this method of roof support was introduced into other collieries of the Company, and, since 1950, a total of 164 miles of roadways and rooms have been roof-bolted.

A paper on the development of roof bolting in the collieries of the Dominion Steel and Coal Corporation was published in 1955, and, for this discussion, the present practices and changes in methods since 1955 will be presented.

The standard bolt used throughout our operations is a slotted mild steel bolt, 4 ft., 1 in. in diameter, having the Canadian Standards Association specification G 40.4. It has a yield strength of 33,000 pounds per square inch, and an ultimate strength of from 66,000 to 72,000 pounds per square inch. The bolts, cast iron wedges and shin plasters are manufactured in our own shops at a cost of $1.26 per completed bolt.

Holman Dryductor Stopers are used for drilling, and have proved very successful due to an eductor which exhausts the cuttings via the hollow drill steel through a passage in the machine, and thence to a vacuum can where the air is filtered and returned to atmosphere. This stoper was adopted after trials of several machines which, in the early days, did not have as fully developed a dust collecting system as the Holman Dryductor. Dust counts show the dust collecting system of the latter to be very efficient.

The split bolt and wedge is anchored with the same stoper, using a dolly on the bolt threads, and the roof-bolt nut is tightened with an impact hammer to a torque of approximately 200 foot-pounds.

In the mechanized room and pillar sections, the immediate roof consists of 2½ feet of hard laminated shale. Above this horizon, the shale bands are fairly soft. When the split bolt and wedge is driven into the roof, the bolt cuts a dovetail in the relatively soft shale. Pull tests on the anchored bolts show that they can withstand a pull of 21,500 pounds with a maximum slippage of ¼ inch.

The standard roof bolting pattern in the mechanized sections consists of 4 to 5 bolts across the width of the opening, with the rows spaced 4 feet apart. The rib bolts are set 6 to 12 inches from the coal rib and angled over the coal at an angle of 30 degrees from the vertical. It is standard practice to follow the roof bolting with warning timber set 4 feet from each rib and 4 feet apart.

Where Joy loaders are used, the roof is bolted to within 3 feet of the face, and roof bolting is completed before the face is undercut. The roof bolting crew consists of two men who roof-bolt the working place immediately after the coal has been loaded out.

This method of roof bolting is not possible where Joy Continuous Miners are installed. With this machine, hydraulic lifting arms are used to place a bar against the roof 8 feet from the face. When the roof is heavy, 85-pound rails are used and, where conditions permit, wood booms or props and cap pieces are used for temporary support. To keep up with the advance of the Continuous Miner, the roof bolting crew consists of three men equipped with two roof bolting machines. These men work as a team, and roof-bolt as close as possible behind the Continuous Miner.

In 1951, roof bolting as a means of support in the advancing longwall levels was tried as an alternative to the use of steel arch supports. The roadways are brushed in the roof of the seam following the total extraction of the coal. These roadways are constructed in two stages. Initially, a roof brushing of from 4 to 5 feet is taken immediately behind the face, and the stone is used to build stone packs on either side of the level. Following the convergence of this initial brushing, a second brushing is taken 250 feet outbye the face, and a 38-pound, "H"-section steel arch is erected. Roof bolting, in place of steel arches, was used to support the roof and ribs at the second brushing. More than a mile of roadway was bolted using 8-foot bolts, but this method of support has since been discontinued due to the crumbling of the roof as a result of its uneven subsidence across the roadway.

In 1959, two experimental retreat longwalls in Dominion No. 20 Colliery were developed and brought into production in order to evaluate the economies of eliminating the

*Dominion Steel and Coal Corporation, Ltd., Sydney, Nova Scotia.

heavy level brushing and maintenance costs inherent in mining by the advancing longwall method. The development levels were driven 3,000 feet to the boundary, and the roof was supported with roof bolts and steel booms. The longwalls were successfully retreated and the roof bolting was effective in maintaining the roadways immediately ahead of the retreating face.

Based on the performance of this retreat panel under a cover of 1,000 feet, experiments were undertaken in Dominion No. 12 Colliery, in collaboration with the Federal Mines Branch, in order to determine if development roadways could be maintained with a depth of cover of 2,500 feet. No. 24 East level was driven as an experimental opening to evaluate various methods of roof support. Mining methods varied from conventional cutting and shooting to mechanized mining, using a Continuous Miner. The supports ranged from steel booms and wood props to steel booms and wood props combined with various roof- and floor-bolt patterns.

A total of forty convergence observation stations were installed on the level. Each station consisted of two 4-foot steel pins, one anchored in the roof and the other anchored in the pavement strata. The maximum rates of closure noted on the observation stations occurred at those places where the supports consisted of steel booms and wood props, and where explosives were not used.

Again, with no explosives, the introduction of roof and floor bolting resulted in a substantial decrease in the rate of closure with a roof-bolt pattern of five bolts per row and rib bolts angled over the rib. This gave the least rate of level deformation.

The data obtained thus far, during the initial 300 feet of development, indicate that the mining method and the type of support installed at the face not only govern the rate of convergence at the face but have an effect on the rate of convergence in the level for a distance in excess of 300 feet outbye the face. When changes in mining, or changes in the type of supports installed at the face, were made, there was a change in the rates of convergence of all the total convergence observation stations located within 300 feet of the level face.

In 1955, a semicircular tunnel was driven downhill, on a gradient of 11 degrees from the surface, to intersect the underground workings at Sydney Mines. The drive went through weak shales and sandstones dipping 4½ degrees in the same direction as the tunnel. The method of temporary support involved roof bolting the roof and sides of the excavation with bolts up to 12 feet in length. The bolts were placed by the drilling crew working off the top of the muck pile.

This method of support eliminated the necessity of timbering, and provided a clear passage for the drilling and mucking equipment. The permanent support, carried 400 to 600 feet behind the face, consisted of 85-pound arches set in concrete. A total of 3,445 feet of tunnel was successfully driven through these weak measures without timber support or serious accident.

Roof bolting is also used as an aid in catching roof coal on the longwall faces when this coal is to be left as a support for a weak shale roof. Bolts are also used to secure the high side coal rib in longwall levels and in rooms, and, for this purpose, a 6-foot roof bolt is used with 4-foot hardwood shin plasters. The bolt is set at an angle in the coal rib and anchored in the roof. This is standard practice in the mechanized room sections, where it had previously been necessary to secure the rib with wood sprags, and has resulted in a safer operation. Where necessary and advisable, roof bolts are also used in conjunction with conventional supports in order to alleviate stone troubles and intrusions.

Summary

A successful roof bolting operation can be maintained where the horizon at which the bolt is anchored is reasonably hard, provided that the strata has not been geologically disturbed and that thin coal seams do not exist immediately above the effective length of the bolt.

Roof bolting is an efficient and economic method of roof support where heavy timber sets or steel booms are necessary to support the roof, or where conventional methods of support interfere with the passage of mechanical mining equipment. However, where wood booms and props suffice as support, roof bolting is a more costly operation.

In some instances, roof bolting along with conventional booming has permitted the successful working of room and pillar sections where wood booming alone would not permit an economic operation.

Reference

Frost, L., *Development of Roof Bolting in the Collieries of the Dominion Steel and Coal Corporation, Limited, in the Sydney Coal Field*, C.I.M. Transactions, Vol. LVIII, 1955, pp. 292-300.

Discussion — Forum on Roof Bolting

Mr. H. P. Boucher, INCO-Thompson

What do you expect from a roof bolt — a complete new method of support, an alternate or replacement for timber, or is it a supplement to timber?

Mr. D. F. Coates

Mr. Coates illustrated two conditions: (1) a flat bed of coal, enclosed by a horizontal roof and floor, worked horizontally; (2) a massive ore deposit being worked vertically in a stope. This led to (1) the strengthened laminated beam theory and (2) the strengthened arch theory. Both types of support are resistant to ground pressures in excess of those safe for timber, but the primary reason for timber, to make a workman's place safe to work, is not necessarily done away with by roof bolting. Roof bolting can replace timber, or supplement it, or it can so speed up mining that the place can be mined out before roof deterioration threatens the safety of the workman.

Three members then discussed automatic roof bolting machines as used at White Pine Copper Mines in Michigan. All agreed that they would be wonderful when perfected.

Mr. T. G. Callcott, Broken Hill Proprietary Company, Australia

Mr. Callcott asked why more split-rod-type bolts were not used instead of the shell type.

Mr. T. S. Cochrane, Mines Branch, Ottawa

Mr. Cochrane replied that the type of anchorage selected to give the best grip depended entirely on the characteristics of the rock layer available for anchorage.

Mr. T. G. Callcott

Mr. Callcott inquired if there were any devices which could be put on roof bolts in order to indicate actual loading.

Mr. D. F. Coates

Mr. Coates replied that, in addition to the hard rubber sandwich-type disc plate, and the belled plates which show loads by curling at edges, both of which are on the market, a cylindrical variety, equipped with strain gauges, was now under development by the Mines Branch.

Mr. G. N. Forrester, Steel Company of Canada

Mr. Forrester said that his company had, for the past 6 years, been actively studying and manufacturing different types of anchorage shells to suit all conditions, and inquired of Mr. Dwarkin if there were any obvious reasons as to why one design might be superior to others in holding power. How much care is taken to ensure the proper size of drilled hole for each type of shell?

Mr. L. Dwarkin, Crow's Nest Pass Coal Company

Mr. Dwarkin said that, for a given rock, one shell might have a larger effective serrated contact area under actual test, but that part of the answer involved the amount of effective pre-stressing that could be developed at installation time.

Question:

Why were 7-foot bolts used instead of 4-foot ones?

Answer:

During the experimental period, a very safe design was required — this meant 7-ft. bolts. It is hoped that, as our knowledge increases, it will be possible to reduce the bolt length.

Question:

Would the use of square steel plates against flat 2-by-12 timber result in much de-stressing due to biting into the wood?

Answer:

During our work, to date, we have never used planks. The bolts were inserted directly against the roof.

Mr. T. S. Cochrane

Mr. Cochrane asked why Mr. Dwarkin selected the maximum load developed to rate the anchorage efficiency of a shell. The graphs showed a large slip of one inch or more in the anchorage at maximum load. Could this be tolerated? Why didn't he choose some lesser point, such as a deformation or slip of 0.15 inch, for a cut-off point and take that load as an index of efficiency, thus ensuring that the maximum anchorage slip would not exceed the allowable roof deflection?

Mr. D. F. Coates

Mr. Coates said that to select a cut-off point arbitrarily and compare results in this way was working to a hypothesis that was not proved.

Mr. Ferguson Grant
(written contribution)

The maximum load should be in the area where the largest section of shell was anchored in the strata. This could be checked by the nut position. Maximum load, with one or more inches of slip, would mean that the expansion shell had been distorted coming up to this load. In other words, the effective holding area of the shell was sharply decreased as the top section of the sleeves would become barrel-shaped and turn in. Unless this shell was pulled into a much harder band of strata, it would be past its point of failure.

At the meeting, there was some discussion on the use of $\frac{5}{8}$-in.-diameter bolts because of failure to attain pre-stressed loads of more than 4 or 5 tons on installation.

D. F. Coates
Head,
Mining Research Laboratories,
Fuels & Mining Practice Div.,
Mines Branch,
Dept. of Energy, Mines and Resources,
Ottawa, Ont.

L. M. Dwarkin
Chief Engineer,
Crows Nest Industries Ltd.,
Fernie, B.C.

Roof Bolting Effectiveness at Michel

Annual General Meeting,
Quebec City, April, 1966

Transactions, Volume LXX, 1967, pp. 32-37

ABSTRACT

Crows Nest Industries Limited adopted roof bolting in 1963 to improve roof control and to extend continuous miner utilization in one mine. Bolting was held back because anchorage tests gave inconsistent results. The time lapse between mining and bolting was suspected as a cause, and so a test entry was driven and sections of it were bolted as part of the work cycle in addition to the normal timbering. This paper presents the results of the instrumentation of test entries and discusses the utilization of the data to improve mining procedure.

Summary

As a result of converting from traditional mining methods to the use of new machinery that mines and loads continuously, it became important to reduce the amount of time required for installing support and to increase the amount of space for manoeuvring the continuous miner.

Owing to the poor roof conditions and to the dispersion of the initial pull test results in areas that had been mined out for some time, additional pull testing was conducted together with two test entries where the effects of early bolting at the face were determined by measuring bed separation in the roof, closure and the variation of bolt loads throughout the entries and with time.

The results of the work showed that bolting, compared to timber support, improved roof conditions. However, in some ground, bolting was not feasible because of inadequate anchorage. The importance of early bolting at the face was indicated by bed separation measurements which showed that normally as much separation occurred in the first 24 hours as in the following 30 days. The importance of installing additional support at prospective cross-sections before cross-cuts are driven was clearly shown by the measurements. Bed separation measurements at the rib showed the effective roof span to be greater than the nominal breadth of the entry.

Although it was not possible to predict from geological or miners' observations the areas that would ultimately deteriorate (taking 2 to 20 days), it was found that a criterion of bed separation could be used. If expansion of the roof rock between the collar of the hole and the anchorage at 7 ft was greater than ¼ inch in the first 24 hours, ultimate poor roof conditions could be expected and, consequently, additional bolts should be immediately installed to prevent deterioration.

Figure 1. — Typical joint system in roof shale, causing a minor fall. Such falls near the face aggravate the installation of bolts.
(photo courtesy of D. K. Norris)

Introduction

In the Crows Nest Pass area of British Columbia, numerous coal seams exist in the Cretaceous Kootenay formation. Active mining is limited to the Michel area, located on the northeast flank of a large syncline. The bulk of the Kootenay formation consists of thick sandstone layers interbedded with shale and bituminous coal (1). Numerous faults have been mapped, some cutting across the bedding planes and others running parallel to them, and distinct joint systems can be seen in the roof rocks of the underground openings, as shown in *Figure 1*.

Mining at Michel had been conducted in the traditional manner by drilling, blasting and loading under wooden roof support. By 1962, a significant portion of production was obtained by means of continuous miners, which cut coal and load in a continuous process. Timbering was now found to consume an undue portion of the mining cycle and to interfere with the mobility and flexibility of machinery. Roof bolting gave promise of alleviating these problems and also

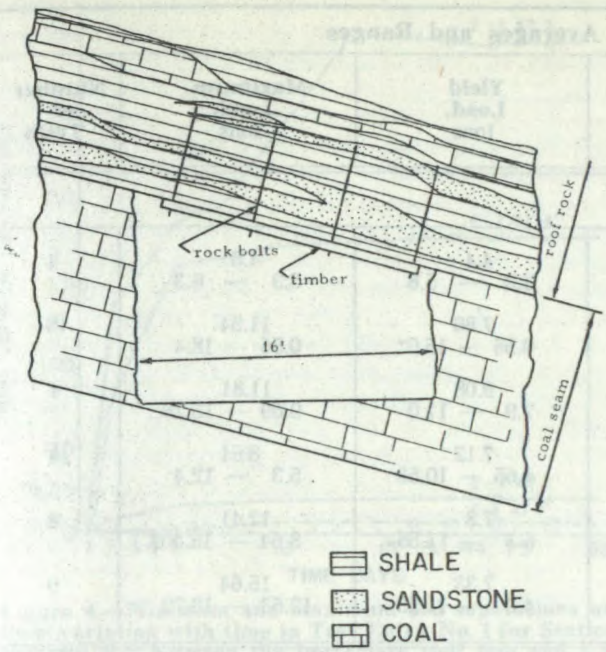

Figure 2.—Typical geometry of rooms and entries with preliminary bolt pattern (7 ft long, ¾-in. diameter, 5-ft spacing).

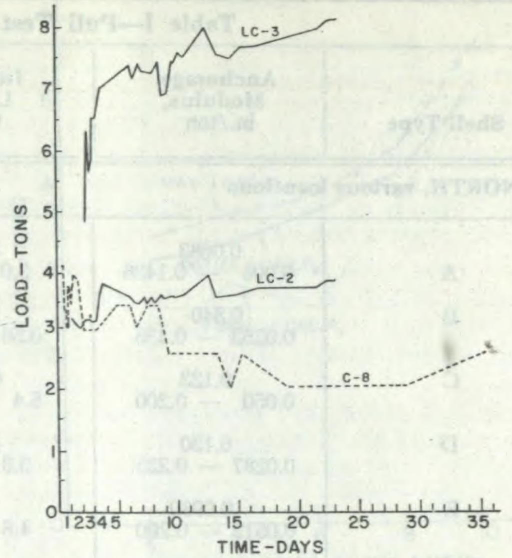

Figure 3.—Variation of some bolt loads with time in Test Entries Nos. 1 and 2.

of improving other conditions, such as material logistics, because of the lesser bulk and weight of bolts compared to timber. The most important factor was the expectation of superior roof control from bolts. Typical entry geometry and roof rock are shown in *Figure 2.*

Testing of Anchorage Capacity

In view of the poor roof conditions in the A-North mine, it was decided to make a special study of roof bolting mechanics in this mine with the assumption that if a satisfactory system could be established here then it would be more than adequate for the other mines.

The first step in the program consisted of a series of pull tests to determine the anchorage capacities in typical roof rock of the six different rock-bolt shells under consideration. The results of the testing have been recorded previously (2), and the data are summarized in Table I, together with the results of some supplementary testing.

The principal conclusions from the pull tests were that the anchorage capacity of the roof rock in A-North mine is inconsistent and that the various shell models differed in their behaviour. As much more investigation was thus required, attention was temporarily shifted to the A-West mine, where the roof was known to be better and quicker results could be expected. This indeed proved to be the case. Anchorage tests there were uniformly good (see Table I), and a striking proof of bolting effectiveness occurred when a timbered portion adjacent to the test area caved. The cave travelled up to but stopped at the first row of bolts. Bolting was then successively instituted in the A-West mine and subsequently in the No. 1 Seam mine, where conditions were similar.

Returning to A-North mine, the dispersion of pull test results indicated that, whereas the average anchorage capacity was satisfactory, low values could be obtained that would not fulfill the support requirements. It was thought that some deterioration of the anchorage strata might have occurred as a result of convergence and bed separation, permitted by the timber supports in the zones where the testing was conducted. Also, it was decided that, even if a roof area could not be bolted and some minor percentage required timber, the savings from the more efficient use of the continuous miner would be worth the complications of having a mixed system of support. However, to determine the cause of poor anchorage capacity, it was decided to conduct a test entry to determine if deterioration of the anchorage strata could be eliminated by bolting close to the face, thus diminishing the bed separation in the roof.

Additional analyses were made to examine the relationship between the initial load set on the bolt, the yield load, the ultimate load and the anchorage modulus before yield (i.e., the deflection of the bolt per ton of load). Rough correlations, with considerable scatter, showed that the anchorage modulus, the yield load and the ultimate load correlated with higher initial loads. The ultimate load also correlated roughly with the anchorage modulus.

Test Entries

(a) ......

Test Entry No. 1 was planned to determine the effectiveness of bolting placed under operating conditions. Five hundred feet of entry, 16 ft in breadth, was divided into ten 50-ft lengths. Sections having timber support only were to alternate with sections both bolted and timbered. This would determine the relative effects of adding roof bolts (7 ft long, ¾-in. diameter, 5-ft spacing). The sections were made as short as possible so that the comparisons would be for the same type of ground; however, they had to be long enough to eliminate the special conditions associated with the ends of each section. It was expected that the length of the sections would correspond roughly with the progress made each shift, so that bolts would be installed on alternate shifts. Besides learning of the influence of bolting close to the face, measurements were to be taken to check some of the design assumptions.

The additional information to be gathered in the test entry included the detailed geology of the roof structure, measurements of the variation of bolt loads

Table I—Pull Test Results, Averages and Ranges					
Shell Type	**Anchorage Modulus,** in./ton	**Initial Load,** tons	**Yield Load,** tons	**Maximum Load,** tons	**Number of Tests**
A - NORTH, various locations					
A	0.0683 0.006 — 0.1426	3.7 3.0 — 4.6	4.4 3.3 — 5.8	4.6 3.3 — 6.3	4
B	0.840 0.0253 — 0.438	5.19 0.56 — 10.0	7.80 0.56 — 15.0*	11.54 0.56 — 18.4	48
C	0.123 0.050 — 0.200	6.30 5.4 — 7.4	9.08 7.9 — 11.0	11.81 9.69 — 15.25	4
D	0.130 0.0287 — 0.225	4.8 3.8 — 6.8	7.12 4.65 — 10.53	8.61 5.3 — 12.4	14
E	0.0930 0.0512 — 0.200	6.3 4.8 — 7.4	7.8 6.3 — 11.55	12.41 8.64 — 18.40*	8
F	0.0546 0.0308 — 0.1121	6.12 4.0 — 8.0	7.33 4.0 — 11.55	15.64 12.63 — 18.72	9
A - NORTH, Test Entry No. 1					
B	0.0513 0.0234 — 0.0715	6.7 5.5 — 7.3	11.9 9.0 — 13.9	12.5 9.0 — 13.9	5
A - WEST					
B	0.0515 0.030 — 0.068	5.42 3.0 — 7.6	13.4 10.0 — 18.5*	18.1 14.0 — 23.8	24
C	0.0504 0.039 — 0.067	6.13 5.2 — 7.0	15.65 14.3 — 16.5*	19.4 18.8 — 19.7	4
D	0.0472 0.048 — 0.136	6.5 4.7 — 8.1	9.2 6.8 — 12.75	12.85 8.16 — 16.9	7
F	0.036 0.032 — 0.042	7.0 5.7 — 8.2	12.5 11.3 — 14.3	19.3 17.0 — 20.8	4
NO. 1 SEAM					
A	0.129 0.116 — 0.143	2.3 0 — 4	3.27 0 — 5.8	3.43 0 — 5.3	4
B	0.050 0.038 — 0.062	6.45 5.39— 3.8	12.64 11.55 — 13.47	16.0 13.15 — 17.88	5

A = single wedge for 1½-in. hole, B = two-leaf bail for 1 ¼-in. hole, C = two-leaf bail for 1 ⅝-in. hole, D = four-leaf bail for 1 ¼-in. hole, E = four-leaf prong for 1 ¼-in. hole, F = four-leaf prong for 1½-in. hole.

*Probably the result of work-hardening due to the re-use of the bolt, and hence the average is not absolutely comparative with other shells.

with time, measurements of bed separation at various elevations in the immediate roof, microseismic monitoring of working in the roof rock and probing in the coal ribs to determine the thickness of relaxed ground.

The detailed geology of the test entry has already been reported (3). In brief, the study showed that the strata had been affected by the slipping associated with folding, which produced numerous small-scale faults and a family of joints (see *Figure 1*). Numerous slickensided surfaces were observed.

The test entry did not proceed as intended, primarily because of the variation in production rate, which resulted in sections of unequal length and instruments that were not installed at planned locations. It is also possible that the quality of installation of the roof bolts was affected, on some of the shifts, by a lack of miners experienced in bolting.

Nevertheless, much useful information was obtained (4). An abstract of this data is as follows. *Figure 3* shows the variation of load with time, as measured on three bolts. *Figure 4* shows the maximum and minimum bed separations measured and their variation with time (three separate rods anchored at 2 ft, 4 ft and 7 ft measured the expansion of the roof rock between the anchorage point and the collars of the holes). The attempts at microseismic monitoring were unsuccessful, as the noise from the mining operation as well as the working of the timber masked the microseisms probably being emitted by the roof rock. The rib probing was unproductive because, unlike previous experience, it was not possible to detect the difference between the resistance to an auger of the

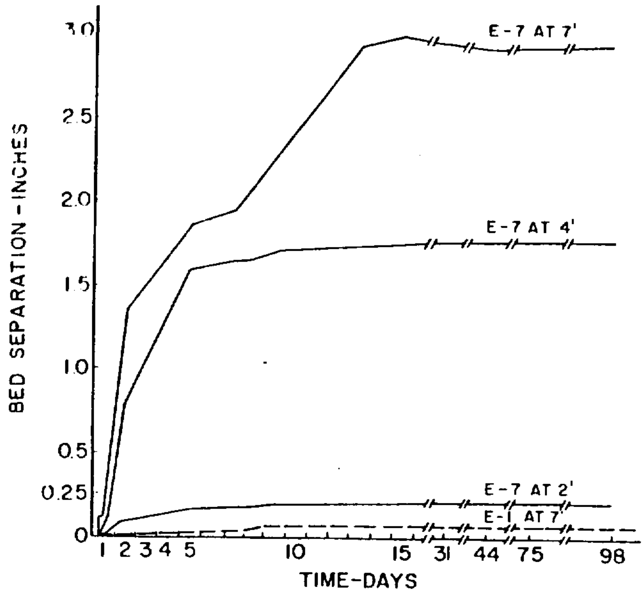

Figure 4.—Minimum and maximum bed separations and their variation with time in Test Entry No. 1 for Stations E-1 and E-7 between the immediate roof line and 2 ft, 4 ft and 7 ft.

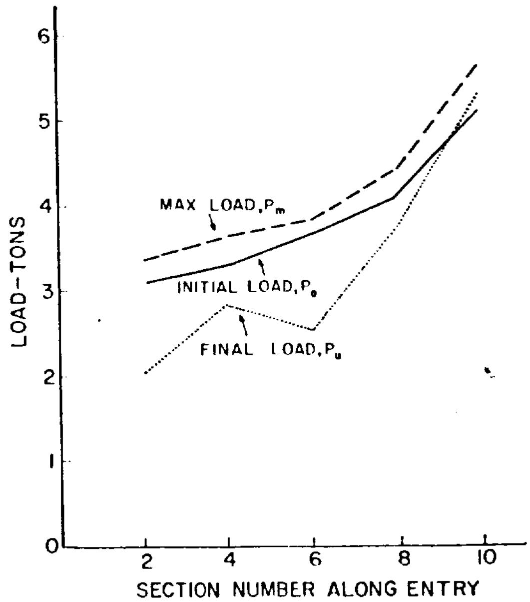

Figure 5.—Variation of average installed loads, P_o, maximum loads, P_m, and ultimate loads, P_u, along Test Entry No. 2. The sections were 50 ft long, so that the spacing of the bolt sections was 100 ft from center to center.

outer relaxed ground and the inner highly stressed ground, although the technique has been successful elsewhere.

(b) ....

As the main questions had not been answered by the initial trial, Test Entry No. 2 was planned. The same layout of alternating 50-ft-long sections was to be used. The supplementary information to be obtained this time included detailed geology of the roof structure, measurements of bolt loads throughout the entry, bed separation measurements and convergence measurements.

Test Entry No. 2 was conducted substantially as planned (5). *Figure 5* shows the variation of the average bolt load throughout the length of the test entry and the relation between initial load, maximum load and final load. *Figure 6* shows the relative magnitudes of bed separation at the center of the entry and at the rib, and the relation between bed separation and convergence — the difference being the compression of the pillar.

Again, the geological studies showed the presence of prominent joint families, two of which were approximately at right angles to each other and approximately normal to the bedding. Many polished and striated bedding planes were observed. In addition, on the basis of cores obtained in the roof, it was found that there were frequent facies changes in the beds, making it impossible to predict the exact nature of the rock at the anchorage point of the roof bolts. This turned out to be particularly critical for Test Entry No. 2, as the predominant materials at this elevation were shales and coal.

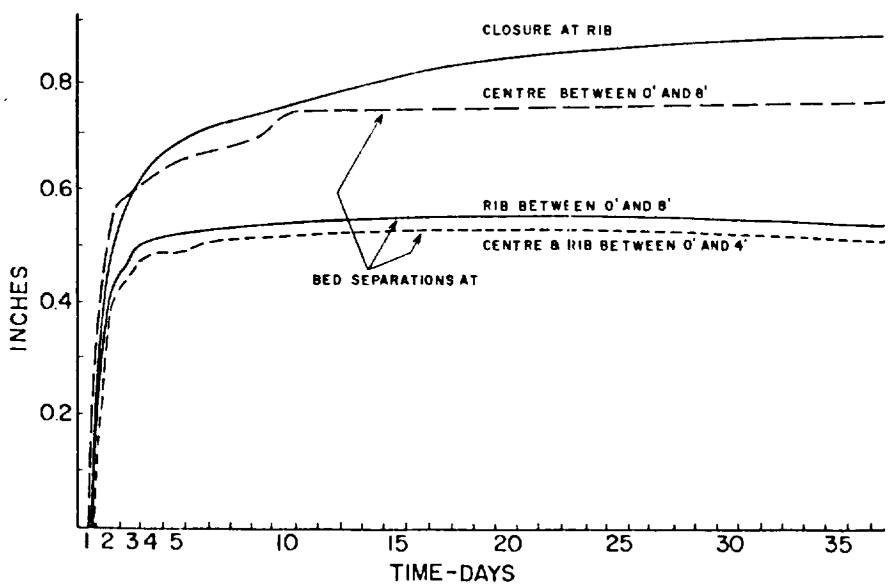

Figure 6.—Comparison between bed separations at the center of the span of the room and at the rib, indicating an effective roof span greater than the nominal opening. Also shown is a comparison between bed separation at the rib and closure at the rib, the difference being approximately equal to the compression of the pillar.

Results

(1) From measurements of bed separation and from observations of general conditions, it was found that bolting improved the roof conditions. In some ground, however, bolting was not feasible because of inadequate anchorage. Consequently, only in the mines with good roof conditions has it been possible to convert to bolting; in A-North, bolting is used where additional support besides timber is required.

(2) A rider seam of coal in the roof was found to be as close as 7 ft to the roof line; consequently, it was decided that 5-ft bolts of ⅝-in. diameter at 4-ft spacing, should be used instead of the original pattern.

(3) It was found that anchorage conditions could deteriorate with the passage of time. Two places were successfully bolted when first mined, but a week later additional bolts could not be installed because of a lack of anchorage.

(4) It was deduced, from the analysis of bail-type shells as given in the Appendix, that for a given bolt load an increase in shell contact area and a blunter wedge both produced less bearing pressure on the walls of the hole and less longitudinal travel of the wedge into the shell. These are desirable features for softer rock; however, a blunter wedge tends to reduce the area of contact. Therefore, the optimum and economic combination can only be determined by pull tests and operating experience.

(5) The bed separation measurements showed quite clearly that, during and after the driving of crosscuts, deflection of the roof increased and the rock deteriorated. It was concluded that additional bolts should be placed between those of the standard pattern before any crosscuts were driven.

(6) Between the collar of the hole and the anchorage at 7 ft, a bed separation of less than ¼ in. during the first 24 hours was found to indicate good ultimate roof conditions; if in operations this criterion was exceeded, it was concluded that additional bolts should be immediately placed to prevent the ultimate deterioration that was found from experience to follow. A warning bolt anchored at a depth of 7 ft and floating freely in a metal collar, with a ¼-in. ring of reflective tape on the bolt, was devised as a monitoring station.

(7) The importance of early bolting, say within half an hour after exposure of the roof, was deduced from the significant amount of bed separation that occurred during the first hour or so at many stations. However, other stations showed that significant bed separation might not start until several hours after exposure of the roof.

(8) In this roof rock, it was found that the installed load on the bolt was very close to both the maximum and ultimate loads that were sustained by the bolt. It is possible that in the weaker strata the installed load was governed by the strength of the rock. Weak strata yield readily and allow excessive travel of the wedge into the shell, so that a small amount of extra load on the bolt (or a small amount of deterioration of the anchorage rock) allows the wedge to pull through and the bolt to fall out.

(9) Because the bed separations measured at the rib line were only somewhat less than those measured at the centerline, the original design assumption that the effective span of the roof would be greater than the nominal 16 ft was confirmed.

Acknowledgments

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APPENDIX

Mechanics of Anchorage

The mode of failure of a rock bolt anchored by a wedge or shell, aside from failure of the steel itself, is a case of bearing failure under an inclined load. The outside surface of a rock-bolt anchor may bear on the wall of the hole at some angle; however, this angle, in most cases, will be small, and in the case of bail-type shell anchors the outside surface of the shell is designed to expand equally and thus be parallel with the side of the hole.

In *Figure 7(a)*, a typical two-leaf bail anchor is shown. The leaves are expanded, with the central wedge drawn down by threads engaging the bolt which is subjected to a torque. Under working conditions, the bolt is under a tension, P, that exerts a downward pull on the anchor. The anchor is supported by frictional forces, F, with maximum values that are dependent on the normal force, Q. The maximum value of Q is, in turn, dependent on the bearing capacity of the rock at that level.

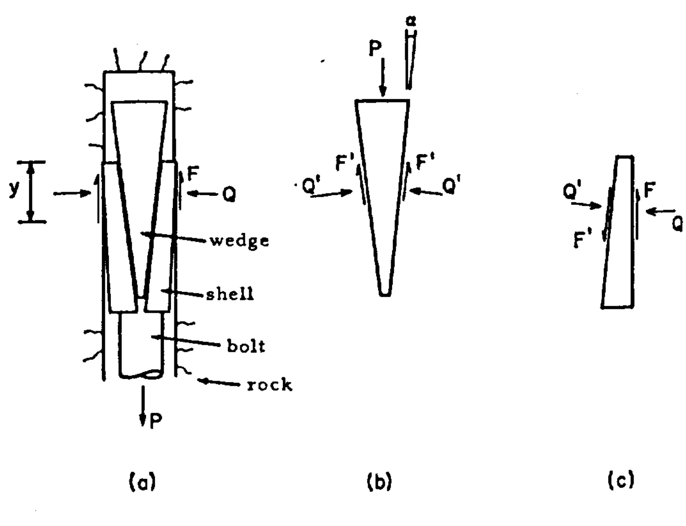

Figure 7.—Mechanics of roof bolt anchorage, showing the forces between a typical wedge and shell, and between the shell and the walls of the hole.

The forces on the wedge, as shown in *Figure 7(b)*, can be analysed. By taking the sum of the vertical forces acting on the wedge, the following relationship can be established:

$$P = 2(Q' \sin \alpha - F' \cos \alpha)$$

where P is the load in bolt, Q^1 and F^1 the normal and tangential reactions on the sides of the wedge, and α the angle to the sides of the hole of the wedge. As slippage will occur between the faces of the wedge and the leaves, the maximum force, F^1, will be related to the normal force, Q^1, by a friction coefficient; hence:

$$F' = Q' \tan \psi$$

where ψ is the angle of friction between the two metal surfaces. Combining these equations, we obtain the following expression:

$$P = 2\,Q' (\sin \alpha + \tan \psi \cos \alpha).$$

By taking the sum of the forces in the horizontal and vertical directions on one of the leaves, as shown in *Figure 7(c)*, the following equation can be established:

$$Q' = Q(\tan (\alpha + \psi) + \cot \alpha) \sin \alpha.$$

By replacing Q^1 with Q in the above equation for P, the following equation is obtained:

$$P = 2Q (\tan (\alpha + \psi) + \cot \alpha) \sin \alpha (\sin \alpha + \tan \psi \cos \alpha).$$

By using the equation for the bearing capacity under an inclined load on a horizontal foundation, but recognizing that the load in this case is applied to a vertical wall, that there is no surcharge on the surface adjacent to the load and that the increase appropriate for a square bearing area would apply to this geometry, the following expression can be obtained that would be applicable to yielding rock (6):

$$P = 2A' cN_c (1 - ((\alpha + \psi)/\phi)^{1/2}) (\tan (\alpha + \psi) + \cot \alpha) \sin \alpha (\sin \alpha + \tan \psi \cos \alpha) \quad \textit{Eq. 1(a)}$$

where A^1 is the area of contact with the rock of one of the two leaves of the shell, c is the cohesion of the rock, ϕ is the angle of internal friction of the rock, N_c is a Terzaghi bearing capacity factor which can be approximated by 7 $\tan^4 (45 + \phi/2)$, α is the wedge angle, and ψ is the friction angle between the wedge and shell. This equation can be abbreviated to the following form:

$$P = A\,c\,N_c\,I\,S \quad \textit{Eq. 1(b)}$$

where A is the total area of all the leaves in the shell, c is the cohesion of the rock, N_c is the bearing capacity factor, I is the reduction factor for an inclined load, which can be taken equal to $1 - [((\alpha + \psi)/\phi^{1/2}]$, and S is the shell factor, being equal to $(\tan (\alpha + \psi) + \cot \alpha) \sin \alpha (\sin \alpha + \tan \psi \cos \alpha)$.

It can be seen that the bolt capacity, P, increases with an increase in wedge angle, α, and, of course, with the area over which the bearing force, Q, acts.

Similarly, from energy relations it follows that:

$$P\delta = 2Qd - \text{Losses}$$
$$\text{or} \quad \delta \propto d/\sin \alpha$$

where δ is the displacement of the force P, and d is the penetration into the rock of the forces Q. Hence, for a critical or maximum penetration, d, the displacement of the force or wedge, δ, decreases with an increase in the wedge angle, α. Therefore, for anchorage rock of either limited bearing capacity or compressibility, these factors suggest that the optimum wedge angle should be larger than for strong rocks.

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