Ground and Standing Support Interaction in Tailgates of Western U.S. Longwall Mines Used in the Development of a Design Methodology Based on the Ground Reaction Curve

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ABSTRACT

Tailgate entries in U.S. longwalls can be subjected to high loads and deformations. Therefore, tailgate entries are generally supported with varying types and amounts of standing support. However, even with standing support, roof falls and blockages can occur in the tailgate when the support system is not properly designed. Safe and efficient operation of a longwall face requires the selection of the appropriate type and amount of standing support. Because of the importance of maintaining a safe and functional tailgate entry, the National Institute for Occupational Safety and Health (NIOSH) is currently developing a standing support design methodology based on the ground reaction curve. Using the ground reaction curve for design requires that the ground and support interaction be quantified. This can be accomplished by the installation and analysis of data from instrumented field sites. Although necessary, the field results cannot develop the complete ground reaction curve. Numerical models, calibrated to the field results, are used to establish the complete ground reaction curve. In the present study, data was obtained and evaluated to establish points for calibration of the ground reaction curve for two western longwall operations.

One test site, Mine A, was located at a depth of 1,550 ft. At this site, the floor consisted of siltstone, while the roof was an interbedded siltstone and sandstone. Because of the depth, a double row of 22-in diameter CANs[1] were installed to support the tailgate. Both the first and second longwall panels have mined past the instrumented site.

The second site in Mine B is at a depth of only 760 ft. The floor at this mine consists of a weak shale/mudstone and the roof consists of thick, top coal overlain by weak shales. At this site, a double row of pumpable cementitious cribs was installed. The second test site has only been subjected to side abutment loading from first panel longwall mining.

The measurements indicate that the support at both mines was heavily loaded when subjected to the side abutment loading from first panel mining. At the deeper site, the support began to yield when the face was more than 300 ft outby during second panel

mining. In comparison to results from previous test sites in the Pittsburgh Seam and Illinois Basin, the amount of support loading and roof-to-floor convergence was an order of magnitude higher at the western sites. This has resulted in a significant difference in the position and shape of the ground reaction curves.

INTRODUCTION

Maintaining a functional tailgate entry is critical to the safe and efficient operation of a modern U.S. longwall. However, tailgate entries are often subjected to high loads and deformations because of abutment loads and caving. Unless adequately and appropriately supported, excessive convergence, ground falls, and blockages can and have occurred. To ensure that the tailgate entries remain functional, various types and levels of standing support are installed. Typically, the type and amount of support is established by trial-and-error or determined by what other longwall mines have done. To improve these approaches, NIOSH is developing a design methodology for standing support based on the ground reaction curve concept. This concept incorporates not only the ground reaction, but also the support load-displacement characteristics. The support characteristics have been or can be measured (Barczak, 2003). Then the support characteristics and support should be selected to match the ground response (Mucho et al., 1999). However, the ground response around the tailgate has typically not been determined and is often ignored in the selection of the support system (Barczak, 2003). The key to this design methodology is to be able to quantify the ground reaction curve. This requires the establishment of instrumented tests.

Several recent studies have been conducted to evaluate the support and ground interaction. In these studies, instrumented test sites have been established to evaluate support and rock mass interaction (Barczak et al., 2008; Dolinar et al., 2009). To develop the ground reaction curves for these sites, the results from the underground tests sites were used in combination with numerical modeling. Based on these studies, ground reaction curves have been developed for tailgate sites in the Pittsburgh Seam and the Herrin #6 Seam in Illinois.

In this study, instrumented test sites were installed in the tailgates of two western longwall operations. The results of the support performance and ground reaction are presented in terms

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1 Mention of any company name or product does not constitute endorsement by the National Institute for Occupational Safety and Health.

of the developed support loads and displacements and the tailgate entry convergence. The points on the ground reaction curve determined from the test site results are presented and discussed. At the two western longwall operations, there was significantly more roof-to-floor convergence than was measured at any of the previous sites. The degree of convergence observed provided a good test of the capabilities of these standing support systems.

DEVELOPING A STANDING SUPPORT DESIGN METHODOLOGY BASED ON THE GROUND REACTION CURVE

The design methodology for standing support is based on the ground reaction curve. The ground reaction curve is the response of the ground to varying levels of support. It was developed for civil tunneling applications but has been utilized in both hard rock and coal mining applications. (Brown, et al., 1983; Hoek and Brown, 1980; Brady and Brown, 1985; Mucho et al., 1999; Barczak, 2003; Medhurst and Reed, 2005; Barczak et al., 2005). The ground reaction will depend on the rock mass properties, the loading conditions and load path, and the support resistance. The support resistance is determined by the load-displacement characteristics of the support. A conceptual ground reaction curve is shown in Figure 1. The ground reaction curve is the support pressure versus the opening convergence. Prior to creating the opening, the support resistance is equal to the forces in the surrounding rock mass and there is no convergence (Point A). When the opening is created and the support level diminished, there is an increase in convergence. The first part of the curve is very steep and near linear. This represents the elastic response of the rock mass. To limit the convergence in this part of the curve requires a substantial amount of support, well beyond what is normally used or can be provided. With a further reduction in the level of the support, the curve becomes non-linear and begins to flatten. This is the result of the rock being fractured and going into yield (Point B). The level of support required to limit the convergence in this region is much less. This is the region where the support could have an impact on the amount of convergence. Once the low point is reached (Point C), the amount of support required begins to increase with additional convergence. This is caused by the downward deflection of the roof allowing more rock to loosen, and the weight of the rocks must then be controlled by the support system.

The support and rock interaction are also shown on Figure 1. Points PQB represent the load-displacement curve for a yielding support. The support is installed after some initial convergence, then develops load and yields before finally intersecting the ground reaction curve (Point B). Equilibrium between the support and the rock mass is then achieved. The minimum level of support necessary to achieve equilibrium is the amount required to reach the nadir (Point C). However, depending on the ground reaction curve, significant rock damage and convergence could occur as this point is approached. Further, there is no margin of safety in this design. The maximum level of support has been termed the support design threshold in a previous study (Barczak et al., 2008). This is the transition point on the curve between linear and non-linear behavior. Levels of support greater than the design threshold would provide little benefit because there is no rock damage and limited convergence.

The ground reaction curve is dependent on the load path taken by the rock mass. Theoretically, in the tailgate entry, because of
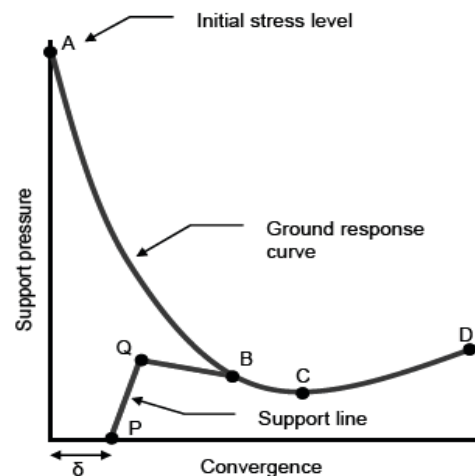

Figure 1.   Conceptual ground reaction curve.

the continually changing loads for a given location as the longwall approaches, there are an infinite number of ground reaction curves. However, for tailgate standing support design and evaluation, there are only four loading conditions that need to be considered. These loading conditions are (A) development loading, (B) side abutment loading from first panel mining, (C) front abutment loading at the tailgate corner adjacent to the face, and (D) full extraction loading inby the face. Figure 2 shows the generalized location of these four points with respect to the panel layout. The ground reaction curves for these four load cases are then determined by numerical modeling.
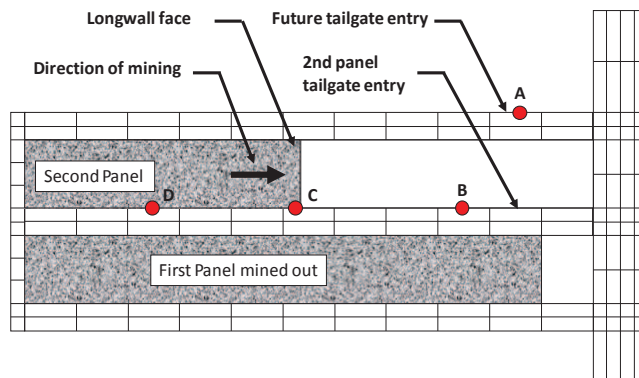

Figure 2.   Generalized longwall panel layout indicating the four locations where the ground reaction curves should be determined.   The points represent the following loading conditions: A-development, B-side abutment, C-front abutment at the face, and D-full extraction (Dolinar et al., 2009).

The support characteristics can be determined through laboratory tests conducted in such facilities as the Mine Roof Simulator (MRS). However, the ground reaction is generally not known or evaluated (Barczak, 2006). The ground reaction can be determined by establishing underground test sites where the support loads and displacements and entry convergence are measured. However, from such sites, only one or two points on a ground reaction curve may be determined.

The full ground reaction curve is developed for all four load cases by using numerical modeling. The finite difference code FLAC is used to generate the curves (Itasca Consulting Group, 2005). The procedure for developing the curves has been detailed in previous studies (Barczak et al., 2008; Dolinar et al., 2009). Site specific loading conditions, mine layout, immediate roof and floor geologies, and physical properties are input into the models to develop the ground reaction curves for each individual site. Support loads are varied in the simulated tailgate entry of the models to produce the ground reaction curve. The field results are used to calibrate the model results and to establish the position of the ground reaction curve.

TEST SITES AND CONDITIONS

Instrumented test sites were installed in the tailgate entries of two western longwall mines to monitor the performance of the tailgate standing support. Each mine had different conditions and each used a different support type. In both mines, support was instrumented near a mid-pillar location and in an intersection.

Support Types

In Mine A, the standing tailgate support consisted of 22-in diameter CANs, manufactured by Burrell Mining Products. The CANs have a yield load of about 100 tons. Mine B used a 30-in diameter pumpable crib. The pumpable cribs have a peak capacity of approximately 160 tons.

Figure 3 shows typical load-displacement curves for each type of support based on tests conducted at the Pittsburgh Research Laboratory in the MRS. For both types of support, the yield or peak load is achieved between 1 to 2 in of displacement and each has a similar stiffness up to yield or peak load. The CAN exhibits an approximate elastic-plastic behavior with the support load still increasing after yield but at a reduced rate. After yield, the CAN will remain stable through 20 in of displacement. On the other hand, the pumpable crib demonstrates a strain softening behavior. The peak load is reached with less than 1 in of support convergence. As a result, in higher convergence environments, the peak load will be exceeded. Therefore, it is the residual strength and not the peak strength that must be considered in such conditions. In this test, the residual strength was above 200 kips. The pumpable crib will generally maintain a fairly high residual strength through about 10 in of displacement. Significant drops in residual strength are associated with the grout fracturing and loss of confinement by the containment bag and wire wrap.

Instrumentation

Instrumentation was installed to measure the support loads, the support, and the roof-to-floor convergence. All the instruments were read remotely using permissible data loggers. The support loads were measured by installing calibrated flat jacks with pressure transducers on the top of the support (Figures 4 and Figure 5). Displacement measurements were made with wire potentiometers. These have a displacement range of about 11 in (Figures 4 and 5). For the CAN displacement, the potentiometers were installed on a piece of plywood that had been placed on top of the support (Figure 4). For the pumpable cribs, the potentiometers were mounted about 6 in from the top of the support (Figure 5).
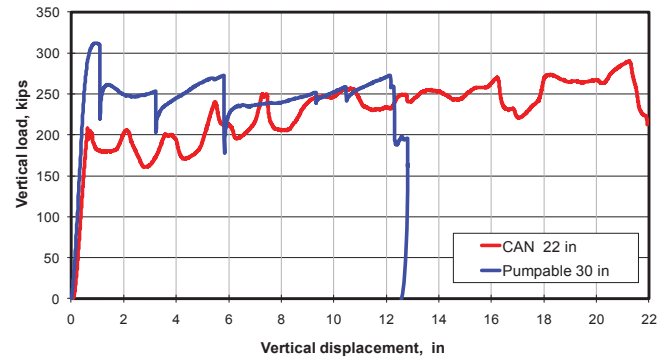
Figure 3.   Load-displacement curves for a 22-in diameter CAN and a 30-in diameter pumpable crib tested in the Mine Roof Simulator. Both supports were 6-ft high.

For both supports, the potentiometers were then connected with a wire to an anchor that was about 12 to 15 in above the bottom of the support.

Figure 4.   Load cell and displacement transducer installed on a CAN.

Figure 5.   Load cell and displacement transducer installed on a pumpable crib.

Wire potentiometers mounted on the roof bolt bearing plates were also used to measure the roof-to-floor convergence. An anchor was installed in a hole drilled into the floor directly below the instrument. For Mine B, roof bolts 9 in long were also grouted into the floor below a roof bolt. The distance between the bolts was then measured manually with a tape measure.

Mine A Test Site

This test site was at a depth of 1,550 ft with a mining height of 8 to 10 ft. The three-entry gate road system had 90-ft and 50-ft wide chain pillars with the wide pillar adjacent to the tailgate entry. The panel width (face length) was 1,050 ft.

The immediate roof consisted of 3 ft of siltstone overlain by 6 ft of sandstone. In the floor, there is 2 to 3 ft or more of siltstone. The average compressive strength for the roof siltstone is 5,900 psi, 12,500 psi for the roof sandstone, and 6,300 psi for the floor siltstone.

Figure 6 shows the layout of the test sites. A total of 8 CANs were instrumented. Initially, a single row of CANs was installed along the panel side of the entry (instrumented CANs 1 to 6). The support spacing in a row was between 8 and 9 ft center-to-center. Installed CAN heights ranged from 8 to 10 ft and were topped with 9 to 20 in of wood. To achieve the required heights, the CANs were made from 2 sections that were welded together with the top section being 5 ft long and the bottom section ranging from 3 to 5 ft long depending on the overall height of the CAN. The instrumentation and support were installed about 300 ft ahead of the first panel face. Because the depth was greater than 1,500 ft, a second row of CANs was installed about 700 ft in front of the face during second panel mining (instrumented CANs 7 and 8). As a result, the second row of CANs was subjected to front abutment loading only and not to the side abutment.

Mine B Test Site

This test site is at a depth of 760 ft with a mining height of approximately 9 ft. A three-entry gateroad system is used with chain pillars 90 ft wide. The panel width is 1,100 ft.

The immediate roof consists of 2 to 3 ft of coal, overlain by 8 ft of a carbonaceous mudstone. In the floor there is 0 to 1 ft of coal, underlain by 3 ft of shale then 13 ft of sandstone. In general the coal measure rocks are weak, and the uniaxial compressive strength average is approximately 2,100 psi for the floor shale, 2,400 psi for the roof shale, 5,200 psi for the roof sandstone, 4,000 psi for the floor sandstone, and 2,000 psi for the coal.

Figure 7 shows the layout of the test sites. Two offset rows of pumpable supports were installed with center-to-center support spacing between 8 and 9 ft. A total of 10 supports were instrumented: 4 in the intersection and 6 at a mid-pillar site. The supports and instrumentation were installed about 250 to 350 ft in front of the first panel mining.
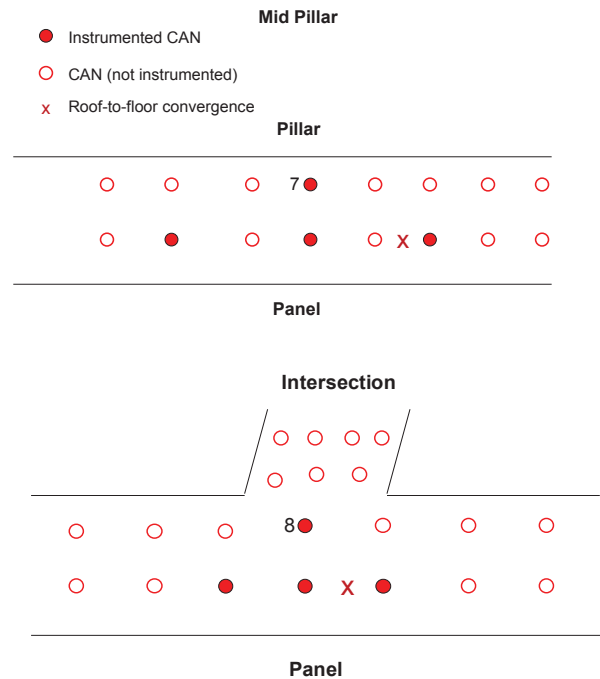

Figure 6.   Layout for instrumented test sites in Mine A.

STANDING SUPPORT PERFORMANCE

Mine A

The support load versus time for first panel mining is shown in Figure 8. The face passed the test sites between February 13 and 15. At the end of the first panel monitoring period, the average support loads developed in the intersection were 66 tons and at the mid pillar site 25 tons. In general the loads increased rapidly after the face passed then continued to increase but at a reduced rate (CAN 4 is the exception). The average support convergence in the intersection was 0.9 in and at the mid pillar site 0.5 in. All instruments used to measure the roof-to-floor convergence were lost, but based on this data in comparison to the support convergence, the estimated entry convergence in the intersection was 8 in and at the mid pillar site 4.5 in. (The instruments measuring the roof to floor convergence were lost within a week after being installed. During this period about 0.6 in of roof to floor convergence was measured.) The loads developed on the support during second panel mining with respect to the distance from the longwall face are shown on Figure 9 and Figure 10. In all cases, the support went into yield. Table 1 gives the yield, maximum, residual, and face loads, as well the distance from the face when the support yielded.

As the CANs went behind the face, all had one or more wrinkles at the top, a couple had over 5, indicating yielding was occurring. CAN 4 at a mid-pillar location buckled (bent over) and split along a weld seam. The instruments used to measure the roof-to-floor convergence were either lost or had reached their maximum range (10 to 11 in). However, it is estimated that there was between 24 and 30 in of floor heave that occurred at the face. Bagging of

Mid-pillar site
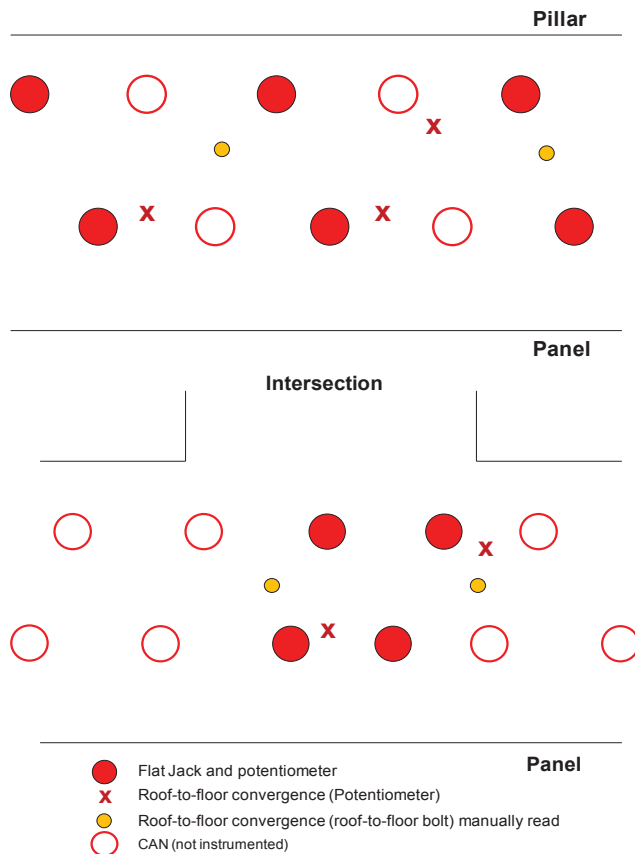
Figure 7. Layout of instrumented test sites in Mine B.

welded wire screen due to loading from the failed immediate roof material between the two rows of support occurred when the face was in the intersection site.

Mine B

Figure 11 shows the support loads with respect to time from the first panel mining. The face passed the sites between November 11 and 14. The average support loads at the completion of first panel mining were 60 tons in the intersection and 37.5 tons at the mid pillar site. There was an initial rapid increase in the load development followed by a leveling off, and in 6 of the 10 supports, load shedding occurred. However, there was no evidence of damage to the pumpable cribs that would have suggested that they were loaded beyond their peak capacity. The average support convergence in the intersection was 0.22 in and at the mid-pillar site was 0.6 in. Measured roof-to-floor convergence in the intersection was 15 in and 17.5 in at the mid-pillar site. The second panel has yet to be mined past these sites.

At both sites, floor heave pushed up around the supports (Figure 12). At the mid-pillar site, the roof started to fail in the middle between the two rows of pumpable supports. This resulted in bagging of the roof screen from loading due to failure of the immediate roof. Additional support in the form of Roc Props was then installed on 6-ft spacing.

ANALYSIS OF THE SUPPORT PERFORMANCE

Significant loads did develop on the support during first panel mining at both mines. For Mine A the loads ranged from 20 to 80% of the expected yield load for the CANs. At Mine B maximum loads were between 30 and 50% of the expected peak capacity for the pumpable support. Most of the support loads developed after the face had passed the test sites. This indicates that the side abutment causes the support loads.

Load shedding occurred on one support at the mid-pillar site in Mine A and on several supports in Mine B during first panel mining. At Mine A, one CAN reached a peak load of about 65 tons, then the load slowly decreased to 33 tons when the face was well past the support. This is only about 70% of the yield capacity measured on the CANs. During this loading phase, the CAN had actually tilted about 8°. During second panel mining, the CAN bent over and split along a fabrication weld joining the lower and upper CAN sections. This load shedding may have been caused by eccentric loading with the floor yielding under the section of the CAN with the highest stress, thus causing the CAN to tilt, then bend and fail at the seam. Increasing the potential for this condition to occur was the 9-ft height of the support and the 18-in height of the wood material on top of the can to establish roof contacts. This resulted in a height-to-width or aspect ratio for the support of 5.7. The stability of the CANs can become an issue with an aspect ratio greater than 5 (STOP, 2004).

In Mine B, load shedding occurred with 4 of the 5 supports that had functioning load cells at the mid-pillar site and 2 out of the 4 at the intersection site. There was no indication that any of the supports had gone beyond their peak capacity, which would be indicated by fracturing of the cementitious grout. Pumpable support No. 6 had about 2 in of convergence and this amount of displacement could indicate that the support was loaded beyond its peak capacity. However, there were no signs that the bag was bulging due to the formation of fractures in the grout material. Further, some of the supports also saw a small rebound or reduction in the convergence. A significant amount of floor heave occurred (over 15 in). The yielding and failure of the floor around the supports may have caused the load shedding.

During first panel mining in Mine A, the intersection support loads were over twice those developed on the support at the mid-pillar site. The distance from the face where the support yielded during second panel mining was also much greater in the intersection than at the mid-pillar site. In general, it would be assumed that larger displacements, and therefore greater loads, would occur in the intersection. However at Mine B, the average peak loads was nearly the same for the intersections (68 tons) and mid-pillar (70 tons), well below the expected support capacity of 160 tons. This may again be related to the floor strength controlling the maximum loads that could develop on the support. The loads from second panel mining may add further clarification on the role of the floor heave in loading the support and the support performance.

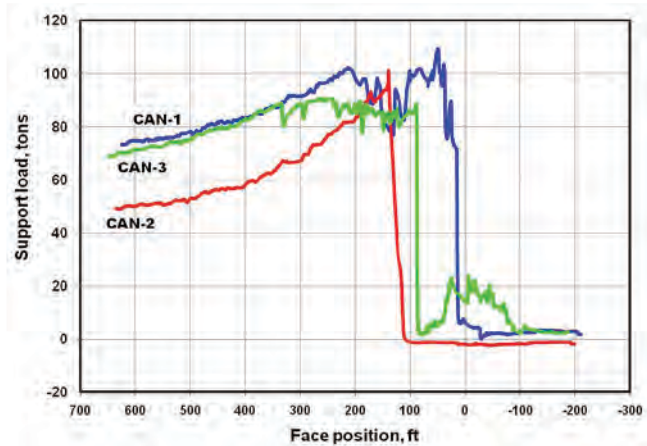**Table 1. Summary of support performance at Mine A, supported by CAN supports, during second panel mining.**

Load cell	Yield load, tons	Yield distance, ft	Maximum load, tons	Load at face, tons	Residual load, tons
Mid Pillar					
CAN--4	27	104	30	3	3
CAN--5	108	33	115	5	15
CAN--6	80	90	93	62	90
Average[1]	94	61.5	104	33.5	52.5
CAN--8	148	22	184	101	61
Intersection					
CAN--1	103	209	109	6	2
CAN--2	92	171	101	0	0
CAN--3	89	334	90	14	2
Average	95	238	100	7	1
CAN--7	-	-	-	-	-

[1] Average does not include CAN-4.
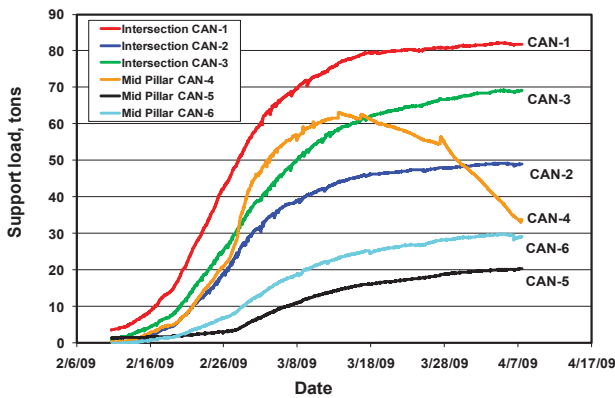
Figure 8. Support load versus time, side abutment loading from first panel mining,

Figure 9. The support load versus the face position for second panel mining for the intersection site at Mine A.

During second panel mining at Mine A, all the CANs went into yield well outby the face. The average yield load for the support along the panel was 95 tons for those in the intersection and 94 tons for the mid-pillar site (excluding CAN 4). The yield loads are close to the 100-ton yield capacity for the support as determined in the laboratory. The average yield distance for the intersection support was 230 ft and 75 ft for the mid-pillar site. Because of the stable yielding characteristics of the CAN, even after yielding outby the face, the CAN has sufficient capacity through the range of displacements that were measured. The yield load was sustained in several CANs over 100 to 300 ft of longwall panel mining (Figure 6). However, significant load shedding had occurred before the face reached the support with 5 of the supports having loads only between 0 and 15 tons when positioned at the face. Only one support along the panel side continued to have a significant post-yield load. For the five load shedding supports, the load shedding occurred very rapidly, but this was not the result of the failure of

the load cells. By this time significant floor heave had occurred (24 to 30 in), and this load shedding could be related to the floor yielding and failing around the supports.

The amount of the support load that resulted from the floor heave is not clearly known, but it must be significant. Much of the floor heave occurred between the supports. However, in general, the standing support system should not be designed to reduce floor heave. If the floor had not failed around the support, floor heave of the magnitude observed at both mines would have seriously damaged the support and little support capacity would have remained to provide support to the roof. This is especially true for the pumpable supports that strain soften after the peak load. To a large extent, floor heave may be considered uncontrolled convergence and a significant amount of support would be required to reduce the floor heave (Barczak, 2005; Barczak et al., 2008).
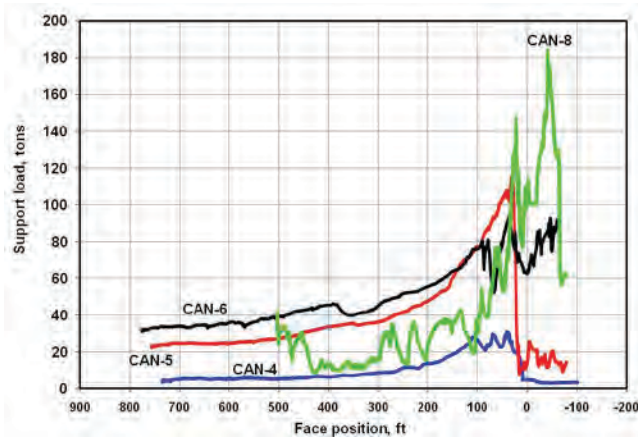
Figure 10. The support load versus the face position for second panel mining for the mid pillar site at Mine A.
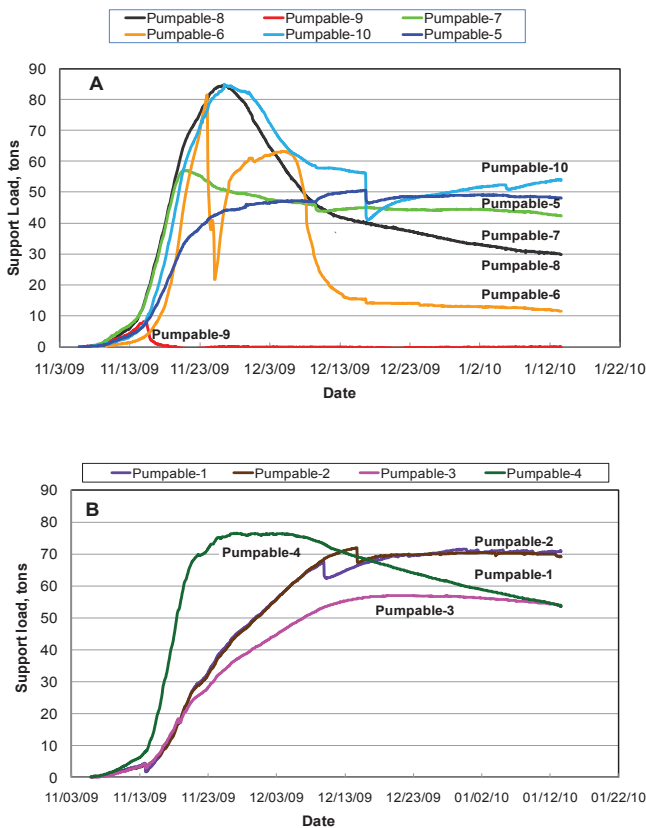

Figure 11. Support load versus time for first panel mining for Mine B—mid pillar (A) and intersection (B).

Figure 12. Floor heave developed during first panel mining from the side abutment at Mine B.

of support applied could be considered to be near the minimum required to maintain stability.

At Mine B, the amount of support convergence was small, but the immediate roof still failed because of the weak roof. From a design viewpoint, the span between the supports may be too large for this roof structure. Under these conditions, standing support was added not so much to limit the ground reaction, but to provide greater distribution of the support for surface coverage over the roof.

Portions of the roof failed between the two rows of support at both mines, indicating the standing support is not fully controlling the unsupported area. Essentially, the standing support is only controlling a limited area. The immediate and intermediate roof that is not controlled by the standing support may still fail. This failed material was in part controlled by the welded wire screen and intrinsic roof support. However, at Mine B, additional standing support was added to help control the areas between the pumpable supports. The area of coverage may have to be increased in such cases.

GROUND RESPONSE IN THE WESTERN MINES WITH A COMPARISON TO GROUND REACTION CURVES FOR MINES IN THE PITTSBURGH SEAM AND ILLINOIS BASIN

The measured ground and support interaction for specific mining locations for the two western mines is given in Table 2. These are actual points on ground reaction curves with each set of numbers representing a point on a different ground reaction curve. Essentially, the field data results will position the ground reaction curve. There are also other points that can be established for the ground reaction curves from the field data. If floor heave between the supports is considered as the response of the ground without support, then further points can be added to ground reaction curves. For Mine A, after second panel mining at the face, the estimated amount of floor heave/convergence was 27 in. For Mine B, after first panel mining, the convergence from the side abutment loading was 17.5 in for the mid-pillar site and 15 in for the intersection site. However, this does not take into account the roof reaction if there

The support at both mines did prevent any large roof fall from developing that could have caused a tailgate blockage. Therefore, the support performance should be considered successful. However, at Mine A, the amount of convergence, though limited by the support, was still sufficient to result in some immediate roof failure. From a ground reaction and design viewpoint, the amount

Table 2. Average support load and convergence for the intersection and mid pillar sites at key locations with respect to long wall face and panel.

Site location	Average support loads, tons	Support rows	Entry support tons/ft	Support convergence, in
First panel—side abutment				
Mine A Intersection	66	1	8	0.9
Mine A Mid pillar	25	2	3	0.5
Mine B Intersection	60	2	15	0.22
Mine B Mid pillar	33	2	8	0.25
Second panel—front abutment/face				
Mine A Intersection	7	2	2	5.5
Mine A Mid pillar	23	2	6	3.4

was actually no standing support; hence it must be considered a minimum level of convergence.

Based on data obtained from similar test sites as established in this study, ground reaction curves were developed for mines in the Pittsburgh Seam and the Herrin #6 Seam in the Illinois Basin (Barczak et al., 2008; Dolinar et al., 2009). Figure 13 shows these ground reaction curves for the side abutment loading and front abutment loading at the face for mid-pillar conditions. In the Pittsburgh Seam case, the depth is 750 ft, and in the Illinois case it is 500 ft.
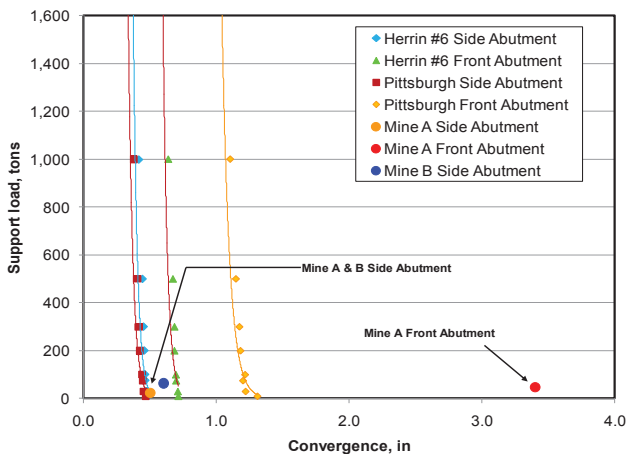
Figure 13. Comparing the ground reaction for the two western mines to that from the Pittsburgh and Herrin #6 seams for the side and front abutment loading conditions.

The ground reaction points for the two western mines are also shown on Figure 13. For the mid-pillar site at Mine A, the support load for the side abutment loading was 3 tons/ft (24 tons) and for the front abutment loading at the face it was 6 tons/ft (48 tons). The support loads from side abutment loading for Mine B at the mid-pillar site were 8 tons/ft (64 tons). For the side abutment case from first panel mining, when only the support convergence is considered, the ground reaction is similar for Mine A and the Pittsburgh and Herrin #6 Seam sites despite the difference in depth. Also, there is only slightly greater convergence in Mine B than

in the Pittsburgh and Herrin Seams. Essentially, the position of the curves for the side abutment loading is similar to the Eastern mines. A significant difference does develop with front abutment loading. In this case, the position of the ground reaction curve has shifted significantly to the right for Mine A. This implies more convergence for the same level of support.

When the support loads drop to near zero in the eastern mines there is only a small increase in the convergence. However, for the two western mines, the convergence between the supports is at least 15 in for the side abutment loading in Mine B and at least 24 in from front abutment loading at the face with second panel mining at Mine A. Therefore, as the support level is diminished toward zero for the two western mines, there would be a large amount of convergence, which this would create a long tail on the ground reaction curves. In comparison, there is only a small tail for the ground reaction curves for the eastern and mid-western mines indicating some non-linear behavior at low support levels. Essentially, there can be a significant difference in the position and shape of the ground reaction curves. For Mine A, the depth may be contributing to this development. However, Mine B has a similar depth as the Pittsburgh Seam and similar gate road layouts. Therefore, for Mine B, the difference in ground reaction must be caused by the weak roof and floor. Further, the roof-to-floor convergence is, at most, 25% greater than the support convergence for the eastern mines (Barczak et al., 2008; Dolinar et al., 2009). This can largely be ignored when evaluating support and ground interaction. This is not the case with the two western mines, where the roof-to-floor convergence is several times greater than the support convergence. The large amount of convergence, primarily floor heave between the supports, may affect the operational function of the tailgate entry and also the structural integrity of the rock where the roof and ribs could destabilize.

Minimizing or preventing floor heave across the opening, not just under the supports, would require a substantial amount of support. For Mine A, considering that the peak loads developed were about 95 tons per support, and based on the support area, the support pressure required to minimize the floor heave is 36 tons/ft^2. If this pressure were applied across the opening, the required support resistance would be 720 tons/ft of entry. The capacity of the CAN support system at yield in this application was only 24 tons/ft of entry. For Mine B, based on the average peak support load of 70 tons, the support pressure required to minimize the

floor heave is 14 tons/ft. The necessary support pressure across the opening would be 280 tons/ft compared to the achieved support pressure at peak of 17.5 tons/ft.

CONCLUSIONS

Instrumented tests sites were established in the tailgates of two western longwall mines to measure the ground and standing support interaction. At Mine A, the supports have been subjected to both first and second panel mining, while at Mine B only the first panel mining has been completed. Several conclusions can be reached from the results and analysis developed from these test sites.

- Significant loads can develop on the support from first panel mining. This is the result of side abutment loading.
- The standing support in intersections can see much higher loads during first panel mining and yield at greater distances from the face during second panel mining than those located in the middle of the pillar. This justifies the need for increased levels of support in the intersection.
- Significant amounts of roof-to-floor convergence did occur at both mines. Much of this convergence is due to floor heave. The standing support did not control the floor heave except underneath the support footprint. Essentially, the floor broke and heaved up around the supports. However, the standing support should not be used to control the floor heave even though the floor heave can impact the functional requirements of the tailgate. The primary function of the standing support should be to control the roof. Substantial amounts of standing support would be required to prevent this floor heave.
- Load shedding occurred with the CAN supports prior to reaching their yield capacity. This characteristic is not seen in the laboratory tests. This may be caused by the floor heave, whereby the floor is yielding under the supports. With the pumpable support, load shedding also occurred that may also be related to floor heave and the floor yielding and not due to the failure of the support.
- Support stability can become an issue as the height-to-width (aspect ratio) ratio goes above 5 with the CANs. This can result in a large reduction in support capacity.
- The position of the ground reaction curves is similar for the two western mines and those from the Illinois Basin and the Pittsburgh Seam for the side abutment loading from first panel mining. However, the shape of the curves is different. The two western mines have long tails on the curves as a result of the large amount of roof-to-floor convergence between the supports. For the front abutment loading at the face, the ground reaction curve position has shifted significantly to the right for the one western site subjected to second panel mining. This is the result of greater support convergence at the western mine. Again, the curve would have a long tail indicating a large degree of non-linear behavior.

DISCLAIMER

The findings and conclusions in this paper have not been formally disseminated by the National Institute for Occupational Safety and Health and should not be construed to represent any agency determination or policy.

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