CABLE SUPPORT IN LONGWALL GATE ROADS

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ABSTRACT

Cable bolt technology used by the U.S. coal industry was developed to a large extent in the 1990s. Today, these cable systems include both cable bolts and cable trusses to provide supplemental and secondary support in gate roads. This cable technology is significantly different than the cable systems in use in either U.S. hard-rock mines or Australian coal mines. Development of this technology was initiated and spurred by research efforts of the U.S. Bureau of Mines, and has continued under the health and safety programs of the National Institute for Occupational Safety and Health (NIOSH). It was also followed up by work of roof support manufacturers to create an essentially new support system. In this paper, the important support characteristics of both cable bolts and cable trusses are discussed. The design of cable systems and the basis for that design for tailgate and headgate situations are reviewed and explained. Case histories are presented on the application of these support systems based on experience gained at a number of in situ test sites.

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INTRODUCTION

Cable bolting was introduced into the U.S. coal industry in 1992 through the research efforts of the U.S. Bureau of Mines. This original work was conducted at a longwall operation in western Colorado for the purpose of finding alternative secondary tailgate support [Tadolini and Koch 1993]. For the initial test, cable design and installation were based on cable technology developed for the hard-rock mine industry [Goris et al. 1993; Goris 1990; Goris 1991; Goris et al. 1994]. This involved the installation of the cables by hand and using a fully grouted cable with a pumpable grout system. This technology was certainly adequate for the hard-rock industry, but not for a high-production longwall. Initially, a crew could install 8 to 12 cables per shift, but with experience this number increased to 30 cables per shift.

From this beginning, the cable bolt system used today quickly evolved until an essentially new product was created. The result was a headed cable bolt installed by machine and anchored with a resin grout cartridge and a partial grout column [McDonnell et al. 1995]. Furthermore, resin manufacturers developed special resins for cable bolt anchors. With these new cable bolt systems, it is now possible to install up to 70 or more cables per shift with a double-boom bolter.

A driving force behind the development and use of cable bolts as secondary support in the tailgate was to replace wood cribs. Wood cribs, especially in the West, were becoming expensive because of a shortage of timber and a lessening of timber quality. A typical longwall gate road in the West supported by wood cribs requires about 248 acres of timber [Tadolini and Koch 1993]. Cutting this amount of timber has an environmental impact. The installation of wood cribs is labor intensive and materials handling is a significant problem, not only from a logistic consideration, but also from an injury standpoint. Especially in the West, where seam are high, a large number of injuries result from crib installation. The high density of the wood cribs necessary in a gate road restricts ventilation and impedes the use of the tailgate as an escapeway [Kadnuck 1994]. All of these considerations can put constraints on the development of super longwall systems.

From a ground control standpoint, wood cribs are far from ideal. Four-point wood cribs are regarded as a soft support system. Improperly built cribs can result in a wide variation in the performance of individual cribs, thus exacerbating ground conditions. In many western mines, yield pillars are used, and by design the tailgate will be subjected to large amounts of deformation, much of which is the result of roof-to-floor convergence. Yet crib systems will resist this deformation, taking up a large portion of crib capacity. Wood cribs also have problems handling the large lateral movements associated with yield pillar designs.

Today, cable bolts are competing against other newly developed types of standing support developed to replace the wood cribs [Barczak et al. 1996; Mucho et al. 1999]. These new systems are definite improvements over wood cribs. Still, from a ground control standpoint these standing support systems use a significant portion of their capacity to resist main roof-to-floor convergence and limit full access by equipment to the face.

Cable bolts are used not only as tailgate support but also for support in bleeders [Tadolini et al. 1993; McDonnell et al. 1995]. They are used as supplemental support, not only in longwall operations, but also in room-and-pillar mines. There are also efforts to adapt cable bolts as primary support because they can be installed so they are much longer than the height of the opening, a factor that could improve roof support in thin-seam mines. Some consideration is also being given to using a single-pass system in gate roads, where the cable bolts will be used both as the primary support for development and secondary support when the panel is mined.

The Australians had used cable bolts in their coal mines as both supplemental and secondary support [Gale 1987; Gale et al. 1987]. The cables were normally 10 m (33 ft) long, fully grouted, and installed with a pumpable grout system. Because the Australians used a two-entry system with large abutment pillars and 60 to 120 m (200 to 400 ft) between entries, the design and use of their cable system offered little that could be adapted to use in a U.S. longwall tailgate.

The development of cable bolts also spurred the development and use of cable trusses. Cable slings had been used on a limited basis since the 1970s [Mangelsdorf 1982; Scott 1989]. However, anchoring the cables was a problem until a system was devised that used anchorage systems based on a resin cartridge inserted into the drill hole. These truss systems can be installed with the assistance of a roof bolting machine, though installation can still be accomplished in the traditional manner with small drills. Cable trusses are now used as supplemental support in headgate entries, especially where horizontal stress causes damage. Cable trusses have been used in open entry recovery rooms and to a limited extent, as secondary support in tailgates.

This paper will discuss cable bolts and trusses and the design of cable systems for the support of longwall gate road entries.

CABLE BOLT

Cable bolts are made from a high-strength steel cable. The most common cable used is seven strands 1.52 to 1.59 cm (0.6 to 0.625 in) in diameter (Goris et al. 1994; McDonnell et al. 1995). The cable consists of six outer strands wrapped around a middle or king wire strand (figure 1). The cross-sectional area of the steel for the cable is 0.55 cm^2 (0.217 in^2). Cable bolts can be of any length, but typically range from 2.4 to 6.1 m (8 to 20 ft) for use in a coal mine. The cables bolts are anchored in the roof with resin grout cartridges using only a partial grout column. This leaves a free cable length in the lower portion of the hole. Cable diameters range from 1.27 to 2.29 cm (0.5 to 0.9 in), but only the properties of the 1.52-cm (0.6-in) in diameter cable bolts will be discussed in the following sections.

Figure 2 shows the components of a typical cable bolt. A cable bolt consists of a cable head that ties the cable strands together and allows the bolt to be installed and rotated with a roof bolter. For ground control, the head is necessary for the ungrouted portion of the cable to take load and resist rock movement. The head also permits the installation of bearing plates and other surface control devices. A barrel-and-wedge system is used to attach the head on the cable.

A stiffener is necessary to install the cable bolt and insert it through the resin cartridge with a roof bolter. Without the stiffener, the cable is too flexible to be pushed through the resin cartridge and will bend outside the hole. If possible, the stiffener should be long enough to be in the hole before the cartridge is punctured by the cable and yet short enough to be installed at a given mining height. A stiffener that is as long as the resin cartridge will allow this to occur. Another function of the stiffener is to prevent the cable from being nicked by the bearing plate during installation, which reduces the potential for corrosion of the cable.

To assist in anchoring the cables and mixing the resin, anchor buttons, "birdcages," nut cases, or bulbs are used in the upper or anchor portion of the cable. These systems are designed for specific hole diameters, usually for holes 2.5 to 3.5 cm (1 to 1-3/8-in) in diameter. An end button holds the cable end together and assists in inserting the cable through the resin. A resin keeper or dam keeps the resin confined along the section of cable to be anchored and also compresses it. The necessity of the resin keeper will depend on resin viscosity, hole diameter, and cable bolt design.

Other designs allow the cables to be tensioned. This is usually done at the head of the bolt through the wedge-and-barrel-head or by a threaded bar attached to the end of the cable. Tensioning the head and wedge is done by hand while the threaded rebar can be tensioned by the roof bolter. Cables with yieldable heads are available where large roof deformation is expected, and loads will exceed cable strength [Tadolini and McDonnell 1998; Vandekraats et al. 1996].

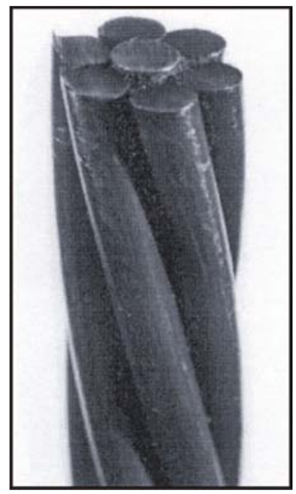

Figure 1.–Seven strand steel cable used to make cable bolts.

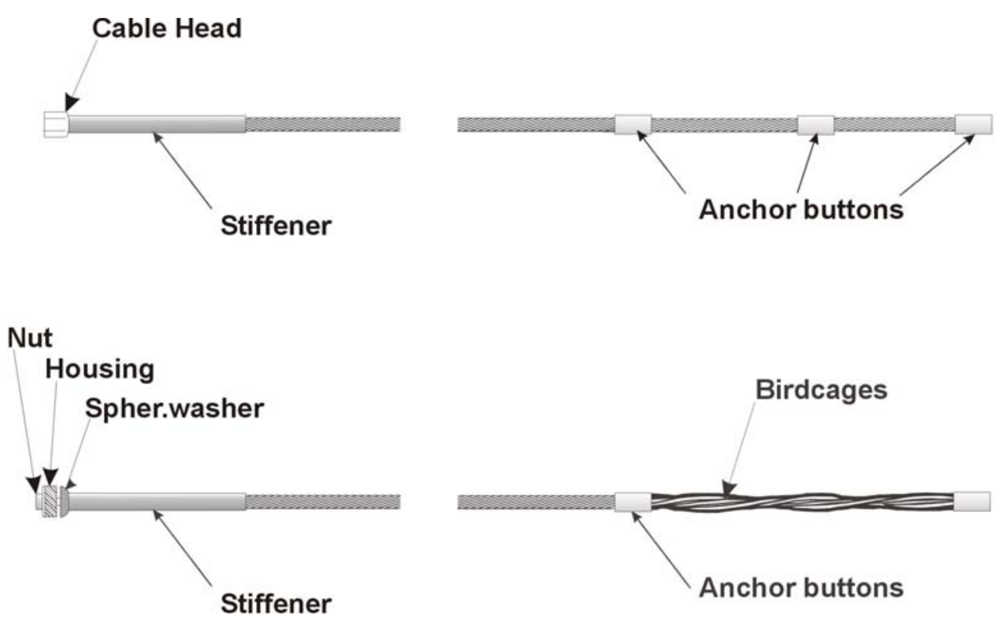

Figure 2.–Cable bolt and cable bolt components.

CABLE SYSTEM CHARACTERISTICS

The cable system consists of a cable bolt with a ungrouted or free length of cable and a resin grout anchor. The bearing plate and other surface control devices held in place by the cable bolt are also part of the system. The performance of the cable support will depend on how well these components act together as a system. Figure 3 shows a cable system installed in a tailgate. In general, the cable should be the weakest part of the system where the other components should be designed to reach the ultimate strength of the cable. This includes the head, anchor, and bearing plate. An exception is the specially designed cable head that allow for controlled yield below cable capacity.

Figure 3.–Cable bolt system installed in a tailgate.

CABLE BOLT CAPACITY

Cable ultimate strength will usually be between 244.7 and 266.6 kN (55,000 and 60,000 lbf) and will normally exceed 260 kN (58,600 lbf), while elongation of the cable at failure can range from 3.5% to 8%. Cables will begin to yield at about 1% strain. Figure 4 shows a typical test for a cable conducted in the laboratory where ultimate strength exceeded 260 kN (58,600 lbf). The load deformation curve from an underground pull test of a resin-anchored cable bolt is shown in figure 5. In this pull test, the cable length was 0.3 m (10 ft) and the grout anchor 0.9 m (3 ft). A maximum load of 268.7 kN (30.2 tons) was achieved with the cable failing during the test, indicating that the anchorage exceeded cable strength. In this case, two strands were broken on the cable, which is typical failure for a cable. When a cable breaks, there is a sudden, drop in load, often to near zero. This is then followed by some load recovery, but this is limited by the strength of the remaining strands and can be highly variable. The load will drop again when the remaining intact strands begin to break. Essentially, the final residual strength of the cable will be zero although the cable will have some intermediate residual strength. From pull tests, the elongation at failure is usually less than 4% strain [Barczak et al. 1996].

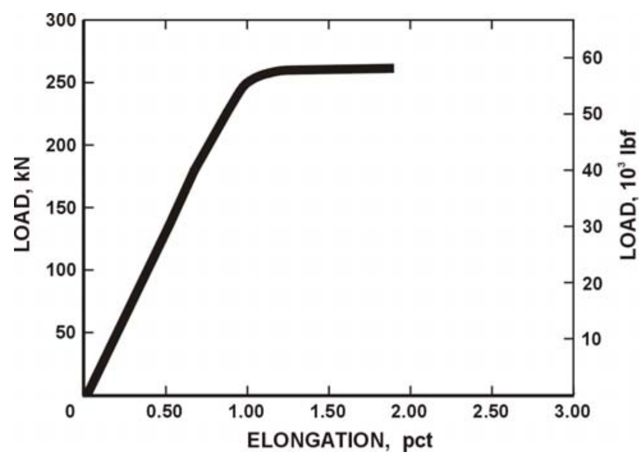

Figure 4.–Load-deformation curve from a laboratory tensile test of a cable.

CABLE SYSTEM STIFFNESS

The stiffness of a cable bolt will be determined by the free cable length in the hole and the elongation properties of the cable. However, elongation in the resin-anchored section of the cable will influence stiffness and must also be considered. The deformation properties of a cable consist of three components-a construction, elastic, and rotational elongation. Construction stiffness is permanent but is usually small. The rotational component is due to the rotation of the cable about the axis during a test or as the cable is loaded. The elastic component is dependent in part on the elastic modulus of the steel composing the cable. The elastic modulus of the steel is 203.4 GPa (29.5 million lbf/in^2). However, the elastic modulus of the cable is

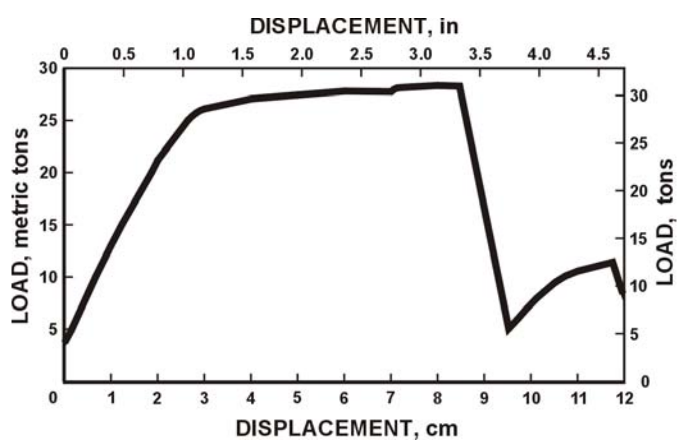

Figure 5.–Load-deformation curve from a pull test of a cable bolt underground.

also dependent on the construction of the cable, which involves the lay length. The lay length is the distance one strand takes to make a complete revolution around the cable [Maryland and American Iron 1985]. The stiffness can be calculated from the following equations.

$$K = E \times A /(L) \tag{1}$$

where K = stiffness, kN/cm (lbf /in),

A = area of cable, cm^2 (in^2),

L = free cable length, cm (in),

and E = elastic modulus of the cable, GPa (lbf/in^2).

Knowing the elastic modulus, length, and area, the cable stiffness can be calculated for a given load. Cable stiffness has been measured underground using a pull test on cables installed in a limestone roof [Zelanko et al. 1995]. For a 3-m-(10-ft-long) cable bolt with a 1.5-m (5-ft) resin anchor and therefore a 1.5-m (5-ft) free length of cable, the initial cable stiffness below the system yield was 106 kN/cm (30.4 tons/in) for cables installed in a 2.5-cm (1-in) in diameter hole, and 98 kN/cm (28 tons/in) for a cable installed in 3.5-cm (1-3/8-in) in diameter hole.

Based on these stiffness values, the deformation modulus of the cables can be calculated from the stiffness equations. However, a correction must be applied to the free length of cable to allow for elongation of the cable in the anchor. From the load transfer characteristics and distances determined experimentally for grouted rebar, the elongation of the cable in the anchor can be approximated by an additional 20 cm (8 in) of free cable length [Serbousek et al. 1987]. Although the anchors will affect the load transfer, any error in determining the additional free cable length from cable stretch in the anchor portion of the cable will have only a small effect on stiffness calculations. From the above test results, for the 106-kN/cm (30.4-tons/in) stiffness, a free cable length 1.72 m (5 ft 8 in) , and an area of 0.55 cm^2 (0.217 in^2), the calculated cable modulus is 132 GPa (19.1 million lbf/in^2).

Using this calculated elastic modulus and the stiffness equation, the stiffness of cables bolts with different free lengths can be determined. For a 4.3-m (14-ft) cable with a 1.2-m (4-ft) anchor and 3.0 m (10 ft) of free cable length, cable stiffness would be 56.4 kN/cm (16.1 ton/in). The assumption is made that the anchor has sufficient length where the anchor will not slip and a portion of the anchor will have little or no load below the yield of the system.

The stiffness of the support will determine how quickly the support will develop resistance and load as the roof deforms. The cable bolt stiffness can be compared to the stiffness of other support systems. For a 1.5-m (5-ft), long No. 6, fully grouted rebar bolt, the stiffness is 700 kN/cm (200 tons/in), and for a 1.8-m (6-ft) long, 1.9-cm (3/4-in) in diameter point-anchor system , the stiffness is 175 kN/cm (50 tons/in) [Karabin and Hoch 1979].

Cable bolts have much less stiffness than most primary support systems. Although the cables are more flexible, the lower stiffness indicates that they will not resist movement as much as other primary support for a given load. For secondary support, a four-point poplar wood crib is 1.8 m (6 ft) high will have a stiffness of 75.3 kN/cm (21.5 tons/in) [Mucho et al. 1999]. This is equivalent to a cable bolt with a 2.3 m (7.5 ft) free length. However, in a tailgate support system, at least two or three cable bolts would be used in place of a single crib. In this case, the cable system would be two or three times stiffer than a crib.

SHEAR CHARACTERISTICS

Resistance to shear and lateral movement can be developed along the free length of the cable and result in cable loading. To determine the shear characteristics of the cables, a series of laboratory tests were conducted where both ungrouted and grouted cables were installed across the block boundary in pairs of concrete blocks [Goris et al. 1995; Goris et al. 1996]. The blocks were sheared parallel to the contact surface and perpendicular to the installed cable bolt. The results showed that the initial peak shear strength was not changed, but that the residual shear strength at 3.8 cm (1.5 in) of displacement was doubled (figure 6). The cables were not immediately activated, but required about 1.0 cm (0.4 in) of displacement before resisting the shear for a 3.5-cm (1-3/8 in) in diameter hole and about 1.52 cm (0.6 in) before significant resistance occurred. Essentially, shear is resisted only when sufficient movement has occurred and the roof has already been mobilized.

In this series of tests, the maximum lateral displacement on the cables was about 3.8 cm (1.5 in). None of the cables failed as a result of this level of displacement. At this point, the cables were loaded to about 60 kN (13,500 lbf), still well below the ultimate strength. However, in a field study, where about 0.3 to 0.46 m (1 to 1.5 ft) of lateral movement occurred across the entry, several cable bolts had failed. It is estimated that the cable bolts failed at between 5 to 10 cm (2 to 4 in) of lateral movement [Dolinar et al. 1996]. The failures occurred from a combination of shear and tension.

RESIN ANCHOR

Several factors influence the effectiveness of the resin anchor, including anchor length, type of resin, and hole diameter [Zelanko et al. 1995]. Figure 7 shows a cross section of a cable installed in resin. The cables will transfer the load through the anchor to the rock with all the load transfer taking place within the anchor length. However, if the anchor is too short, the anchor could slip (rock-grout interface failure), especially in weak rock. With a longer anchor, the anchor

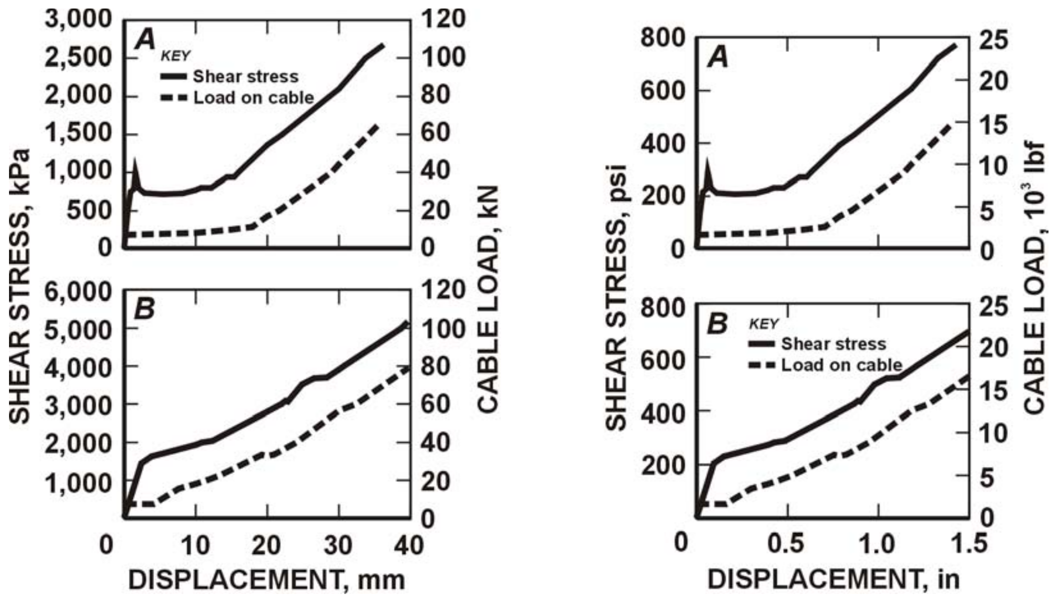

Figure 6.–Typical cable load and shear stress versus shear displacement A, smooth joint with cable bolt reinforcement and B, rough joint with cable bolt reinforcement.

could still fail along the grout-cable interface as the cable is loaded and yields where the anchorage will not exceed the strength of the cable. The anchor failure mechanisms are discussed in more detail elsewhere [Goris 1990; Goris 1991].

An important aspect to the development of an adequate anchorage is the addition of buttons, nut cases, garford bulbs, or birdcages to the anchor portion of the cable. During cable installation, these anchor components assist in mixing the resin, which should provide for an improved quality and consistency of the resin anchor, especially in the larger-diameter holes (3.5 cm [1-3/8 in]). Further, laboratory pull tests on short (76.2 cm [30 in and less) column grout anchors have shown that anchor components embedded in the grout significantly increase anchorage capacity over that of a cable without embedded anchors [Goris 1990, 1991]. With an increase in the resin column length to 0.9 m (3 ft), the conventional cable without anchors can achieve the ultimate strength of the cable, although test results are somewhat inconsistent. In laboratory pull tests, only 60% of the tested cables reached the ultimate cable strength while 40% of the cable anchors failed at significantly lower loads [Martin et al. 1996a]. Without the addition of anchor components embedded in the resin, there is a high probability that the cable anchor will be significantly weaker than the cable.

Both laboratory and field investigations using pull tests have shown that 1.2 and 1.5 m (4 and 5 ft) of resin anchor will achieve the ultimate capacity of the cable if properly grouted [Martin et al. 1996a; Zelanko et al. 1995]. Although laboratory tests have shown that a length of 0.9 m (3 ft) or less of anchor can result in the cable reaching ultimate strength, this was not achieved on a consistent basis. In the elastic range, below the yield of the cable, most of the load is transferred within the first

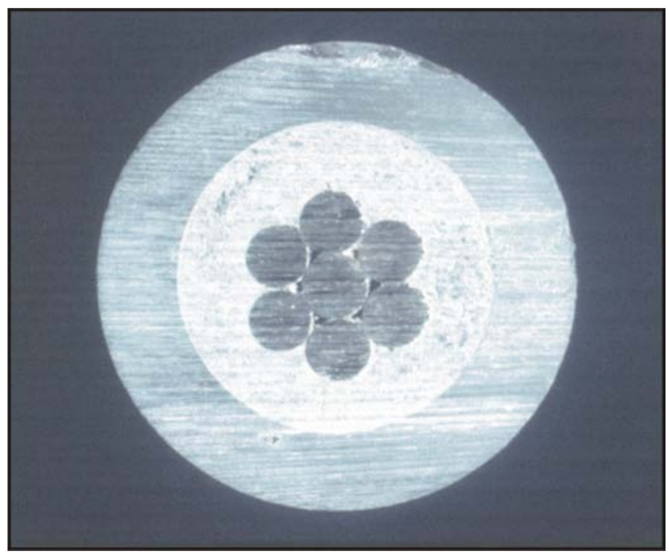

Figure 7.–Cross section of 1.52 cm (0.6 in) cable set in resin, installed in a 2.5 cm (1 in) diameter hole.

0.6 m (2 ft) of the anchor [Goris 1990; Goris 1991; Serbousek et al. 1987]. Essentially, the cable load in the anchor decays exponentially with distance along the anchor. Beyond about 0.6 m (2 ft) there will be little load in the anchor. However, once the cable yields, the lower portion of the resin anchor may begin to fail or the cable may debond from the grout, resulting in the loads being transferred further up the anchor. Essentially, the anchor becomes shorter as the cable is loaded beyond yield. Therefore, a minimum of 1.2 or 1.5 m (4 or 5 ft) of resin will provide a margin of extra length in allowing the cable to reach the ultimate strength. Also, this margin of extra length gives some degree of safety for improper grout installation although this is no guarantee that even a 1.2- or 1.5-m (4- or-5 ft) long anchor not properly installed will result in the cable reaching ultimate strength. Cable systems using longer resin anchor columns of 2.1 to 2.7 m (7 to 9 ft) have also been installed. However, depending on the resin used and the thrust capacity of the bolter, problems may be encountered with inserting the cable through that length of resin column.

An investigation was conducted underground using pull tests on 3-m (10-ft) cable bolts with 1.5-m (5-ft) resin anchors to evaluate parameters other than length that could affect the anchor performance [Zelanko et al. 1995]. Parameters varied in this study included hole diameter, resin type, and use of a resin keeper. The 0.6-in cable is generally installed in either a 2.5 or 3.5-cm (1-or 1-3/8-in) in diameter hole. Overall, the capacity and stiffness of the installed cable bolts were lower in the larger-diameter holes (3.5 cm [1-3/8 in] as compared to a 2.5 cm [1 in]. Although an adequate anchor can be achieved in holes of either diameter, there is less consistency in performance with the larger holes.

Higher-viscosity resins gave a better anchor performance than lower-viscosity resins. Cable resins are less viscous than the standard resins and allow for easier installation with the roof bolter. However, besides decreased performance, resin loss could occur more easily with these types of resin, thus requiring longer column lengths. More viscous bolt resins can be used, but with longer resin lengths, older bolting machines may not be able to thrust the cable through the cartridge. Furthermore, with the standard resins, another installation problem that can develop is loosening of bearing plates or plates not being in contact with the roof. In such cases, the higher- viscosity resins prevent the end of the cable from being pushed up completely through the cartridge even though the roof bolter was able the push the plate against the roof. When this occurs, the ungrouted section of cable bends and with the release of pressure, the cable will spring back, resulting in a loose plate. Resin keepers were also found to be important for the larger-diameter holes, but not a factor in 2.54-cm (1-in) in diameter holes. However, the need for a resin keeper will depend on the overall design of the anchor section of the cable bolt.

From this study, the system with the thinner annulus, the 1.52-cm (0.6-in) in diameter cable in a 2.5-cm (1-in) hole, performed better with more consistent behavior than the system with the larger annulus. In this regard, the button, birdcage, or nut case diameters must be matched with the proper hole size. This study highlights not only the necessity for a properly installed cable anchor, but also a properly designed anchor system as well.

CORROSION

Corrosion is a issue, although the extent of the problem is not completely known. Cables are more susceptible to corrosion and failure than other types of support. A nick in a cable strand that corrodes has a much greater impact than corrosion in a roof bolt. Some observations suggest that bright or black cables have about a 10% decrease in area of the strands six months after installation [Martin et al. 1996b]. Both galvanized strand and epoxy coated cable can be used to minimize the potential for corrosion [Goris et al. 1994]. Manufacturing techniques are now available that do not adversely effect either the strength or flexibility of galvanized cable [Tadolini et al. 1994].

Figure 8.–Surface control in the form of a "Monster Mat" installed with cable bolts.

SURFACE CONTROL

Bearing plates are necessary for the functioning of a cable bolt with a free length and allow the cable to load and resist rock movement while transferring this load thorough the anchor to rock deeper into the roof. Therefore, the bearing plate must be designed for the cable to reach ultimate strength, or a minimum of 260 kN (58,600 lbf). "Monster Mats" and T 5 channel are often installed with the cables to provide additional support and surface control across the row of cables (figure 8). A Monster Mat is a steel pan 0.48 cm (3/16 in) thick and 33 cm (13 in) wide, while a T 5 channel is 0.5 cm (0.2 in) thick and has a 10-cm (4- in) wide bearing surface. Both systems can add significantly to surface control and also provide some structural support. These systems are installed in conjunction with the high-capacity bearing plates.

DESIGN CONCEPTS FOR TAILGATE SUPPORT

In the tailgate, the primary support is designed to withstand development mining, but may not be able withstand the longwall environment and control the lower roof. Therefore, cable bolts can be installed as secondary support to maintain the entry. The cable bolts must keep the roof from falling and the entry open during panel mining. As the lower roof moves and deforms, the cables will distribute the forces that develop below a given failure horizon deeper into the roof through the cable and anchor support. Although there is primary support, it is not normally taken into account when designing the cable support system or, for that matter, another type of secondary support system.

The basic design concept in using partially grouted cable bolts to support the roof is suspension. Essentially, the cable bolt system must maintain and control the dead weight load of rock or rock movement below a potential failure horizon in the mine roof. This in part determines the spacing of the cable bolts. Furthermore, an adequate cable anchorage length must be obtained above a given failure horizon and, combined with the location of the failure horizon, determines cable length. Experience based on test sites in tailgates have further refined and established a basic design for cable spacing and row spacing. Although the cable systems are designed for the full dead weight of the rock, this is seldom seen and is somewhat an oversimplification of conditions, but it provides a starting point for design and designing to a worst-case scenario. Also, lateral roof movement, as well as vertical expansion from lateral roof movement, can cause significant loads to develop on the cables even beyond the weight of the rock.

CABLE LENGTH

The selection of cable length is the probably most crucial aspect of the design of a cable system. Depending on geologic conditions, selecting a length may be simple and straightforward, while in other cases, it may require an iterative process using a range of information. The key is to identify the location of potential failure horizons in the roof that may develop when the panel is mined.

Once the deepest potential failure horizon is identified, the cable length will be the depth of this failure horizon plus the length of an adequate anchor. Typical cable lengths in gate roads are between 3.7 and 4.9 m (12 and 16 ft). However, a minimum length in general should be for the cable bolt to be long enough to be anchored above the primary support. In this case the primary support zone is being suspended by the cables. However, there may be failure planes that develop above the primary support and require a longer length of cable bolt. This potential failure zone may be a flat or arched surface, depending on how the roof may fail. In a gate road situation, much deeper movements may occur that are not relevant to the stability of the immediate roof or the opening.

The initial step in designing an adequate support system requires gathering detailed information on ground conditions and the underground mining environment. To determine a potential failure horizon will require examining the roof and roof geology or evaluating roof performance to determine an adequate cable length. Such information may include a general estimate of rock mass strength or rating, geologic structure, and strengths of the immediate and main roof members. This information can be obtained from roof core samples and supplemented by observations from a borescope or camera to evaluate test holes in the roof. If the rock overlying the immediate roof is stronger or more competent, this may be an obvious place to locate the anchorage and is the easiest situation for determining cable length. However the geology may not be that clear-cut or the depth of the stronger unit may be too deep to be of practical use for supporting the immediate roof. Actual mining experience, test sites, and examination of roof falls can provide more data to help in the design of the cable system. Tests sites with instruments such as multi point extensometers can also be used to locate and evaluate these potential failure horizons. Such instrumented test sites can be used to confirm the adequacy of cable's length and design.

DESIGN FOR SUSPENSION

For cables, to consider that the rock is being supported through suspension may be an oversimplification, but does provide a basis for establishing the initial design of the system. Designing for suspension requires that the cables carry the weight of the rock under the potential failure zone, which, in many situations, is the worst case scenario [McDonnell et al. 1995]. In some situations, there will be loads that actually exceed rock load because of geology, horizontal stress, lateral rock movement, and mining-induced loads.

For suspension, the simplest approach is to identify a parting plane or a flat-lying, potential failure plane above the bolted roof horizon where the roof will shear at the pillar edge of the opening and the entire weight of the rock must be supported as a detached block (figure 9). The weight of the material can be determined from the following equations.

$$F_w = W_e H_p \gamma \, , \qquad (2)$$

where F_w = weight of rock per linear length, kN/m (lbf/ft),

W_e = effective width of opening, m (ft),

H_p = distance from coal roof to parting plane, m (ft) ,

and γ = rock density, kN/ m^3 (lb/ft^3).

If an arched roof failure is formed with the pillars carrying some of the weight, the cables need only support the weight of the rock under the arch (figure 10). The height of the arch will be determined by a combination of the geology, as well as by the vertical and horizontal stresses acting on the roof and the induced mining stresses. Obviously, the length and the number of cables will depend on the height of the arch, and therefore this requires the identification of the failure surface. The weight of the material within the arch can be estimated from the following equation.

$$F_a = \frac{\pi}{4} w_e H_a \gamma \, , \qquad (3)$$

where F_a = weight of rock under pressure arch per linear foot, kN/m (lbf/ft),

and H_a = height of pressure arch, m (ft).

The behavior of the pillar under different loading conditions will affect the width of the opening and therefore the weight of the rock that must be supported (figure 11). The depth of the yield zone can be determined from equations developed by Wilson and depend on the strength of the coal pillar [Wilson 1972]. The following equations can be used to estimate the depth of the yield zone. w = pillar width in meters (feet).

(1) Rigid floor conditions–

$$w = 2 \frac{m}{F} \ln\left(\frac{q}{p + p'} \right) \qquad (4)$$

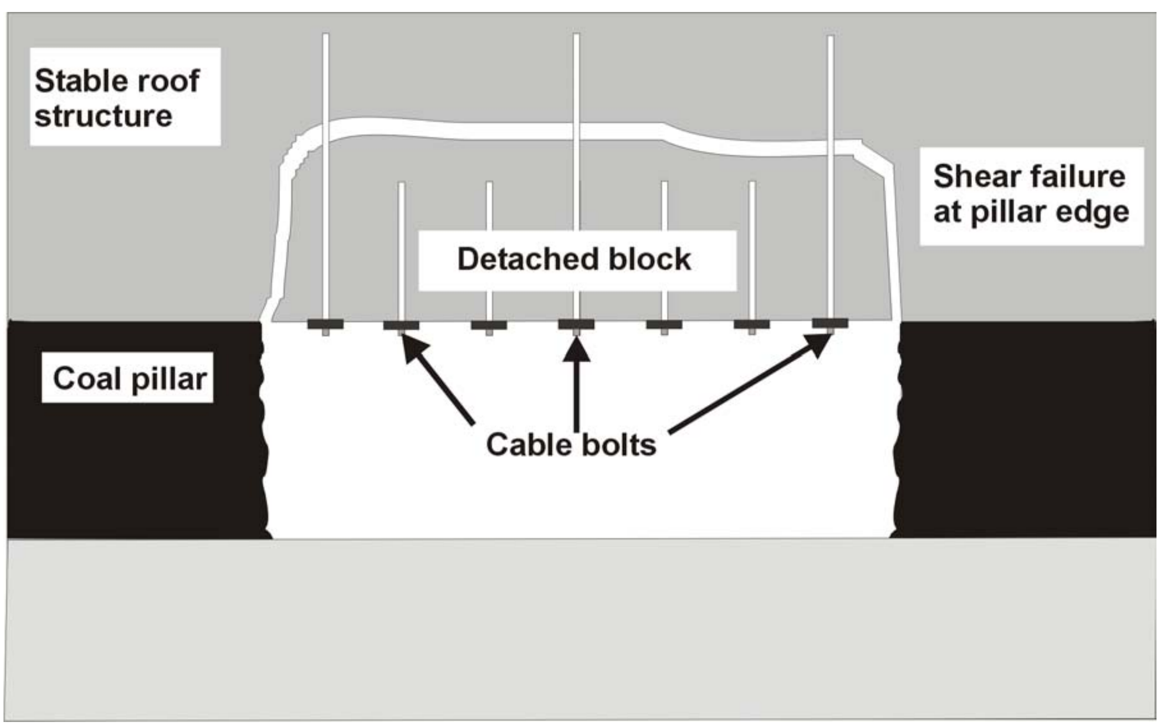

Figure 9.–Detached block of failed roof supported by cables.

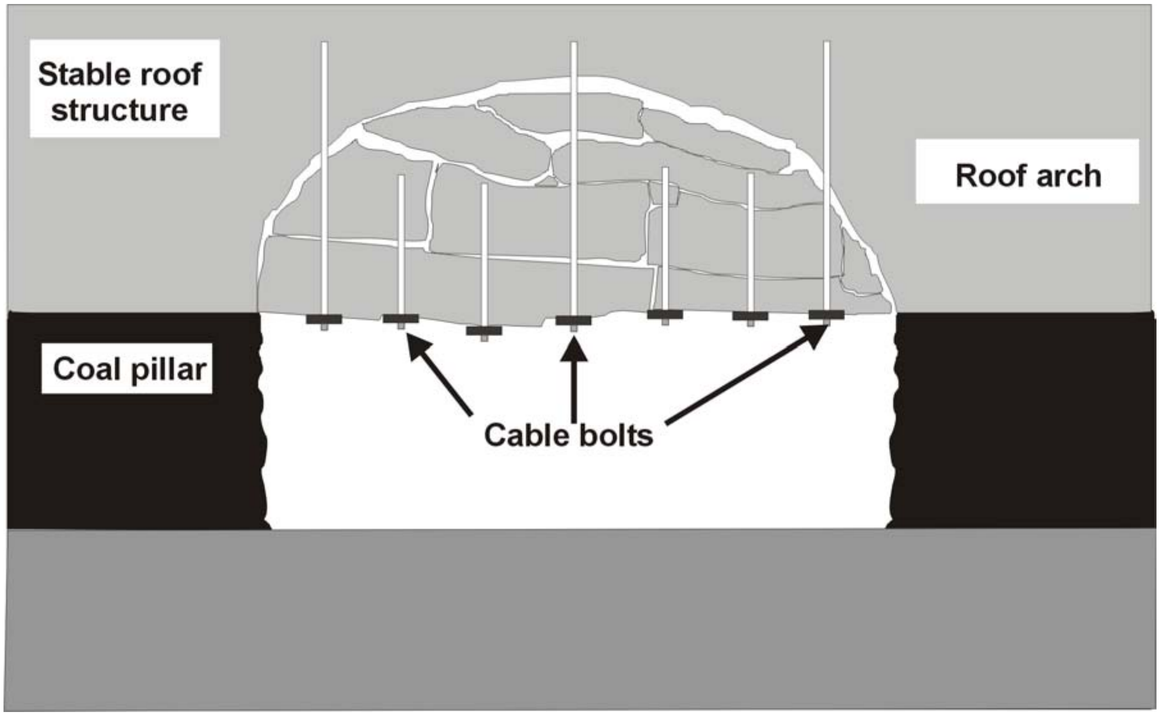

Figure 10.–Formation of pressure arch of failed mine roof material.

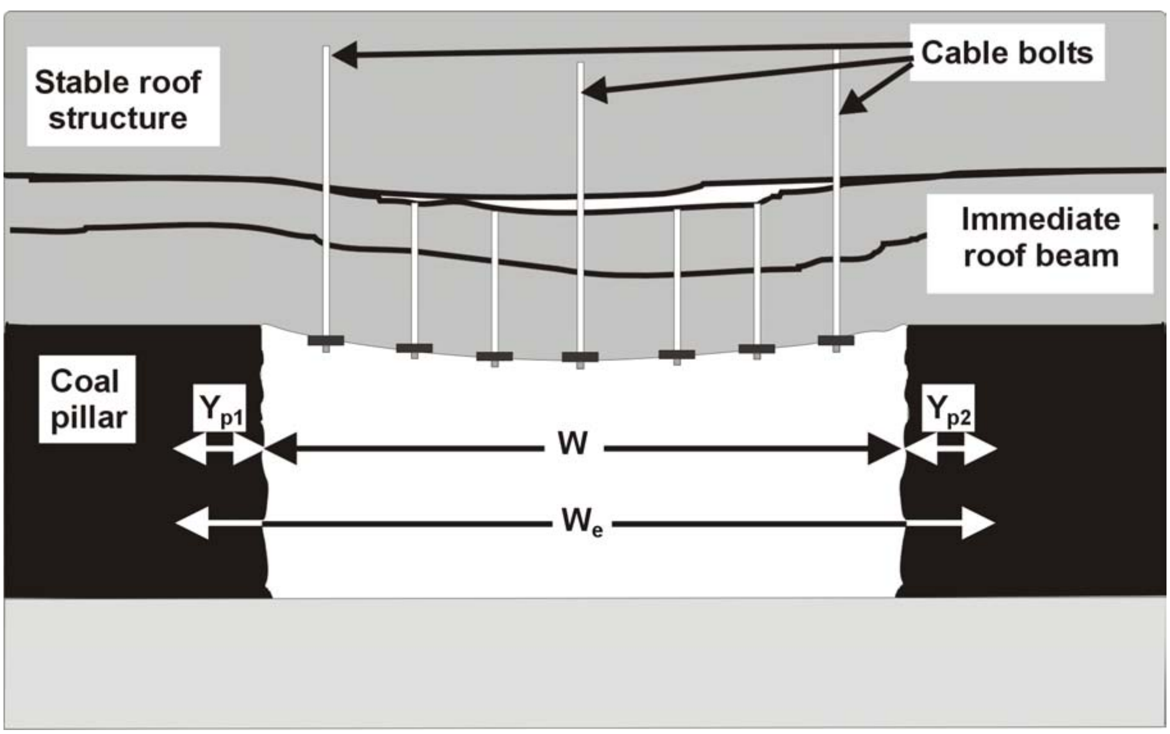

Figure 11.–Formation of yield zone in coal pillar, (W_e = effective width of opening; yp_1 = yield zone of pillar 1; yp_2 = yield

(2) Yielding roof-floor conditions–

$$w = m\left[\left(\frac{q}{p + p'}\right)^{\frac{1}{k} - 1} - 1\right] \qquad (5)$$

$$F = \frac{k - 1}{\sqrt{k}} + \frac{(k - 1)^2}{k} \tan^{-1}\sqrt{k}\ , \qquad (6)$$

where $\tan^{-1}\sqrt{k}$ is expressed in radians,

m = seam height, m (ft),

q = overburden load, t/m^2 (st/ft^2),

p = artificial edge restraint, 0 t/m^2 (st/ft^2),

p' = uniaxial strength of fractured coal, 1/m^2 (st/ft^2),

and k = triaxial factor = $\frac{1 + \sin\varphi}{1 - \sin\varphi}$, where ϕ = angle of interval friction, deg.

Figure 12 shows charts developed from these equations to calculate the depth of the yield zone. The charts were created using a angle of internal friction of 35°. The effective opening or roof width can then be determined from the following equation:

$$W_e = W + Y_{p1} + Y_{p2}, \qquad (7)$$

where W = mined width of opening, m (ft),

Y_{p1} = yield zone for pillar 1, m (ft),

and Y_{p2} = yield zone for pillar 2, m (ft).

Based on the weight of material that must be supported, the spacing of cable bolts across the opening, as well as row spacing, can be calculated. Using a cable with a capacity of 260 kN (58,600 lbf) and varying the number of cables across the opening and row spacing, a design can be determined for different thicknesses of rock that must be supported. Figure 13 shows this design chart for an effective width of 7.6 m (25 ft) and a rock density of 2,403 kg/m^3 (150 lb/ft^3). This chart is based on a flat failure surface developing at the given horizon with the additional weight of material for the yield zone. A separation at 2.4 m (8 ft) would require four cables per row with 2.4-m (8-ft) row spacing. However, there are no safety factors calculated into these charts.

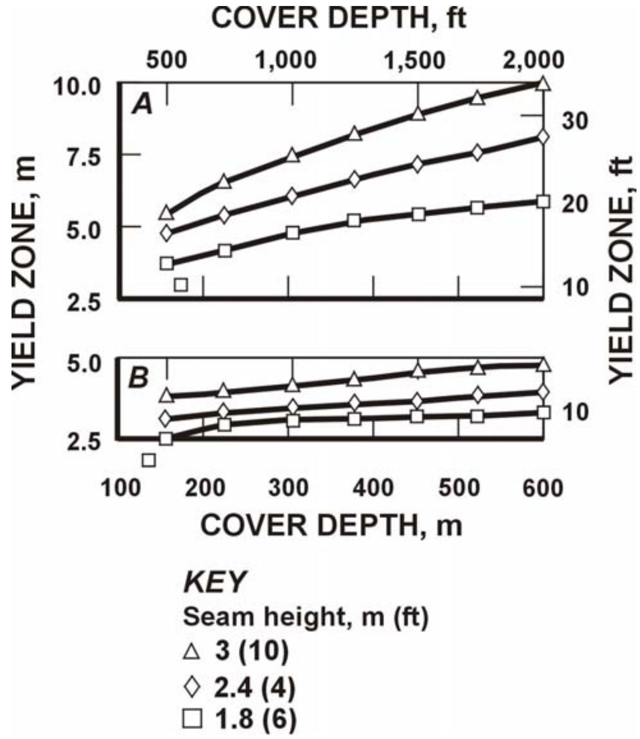

Figure 12. –Design chart to determine yield zone width in coal pillars. A, yielding roof; B, rigid floor.

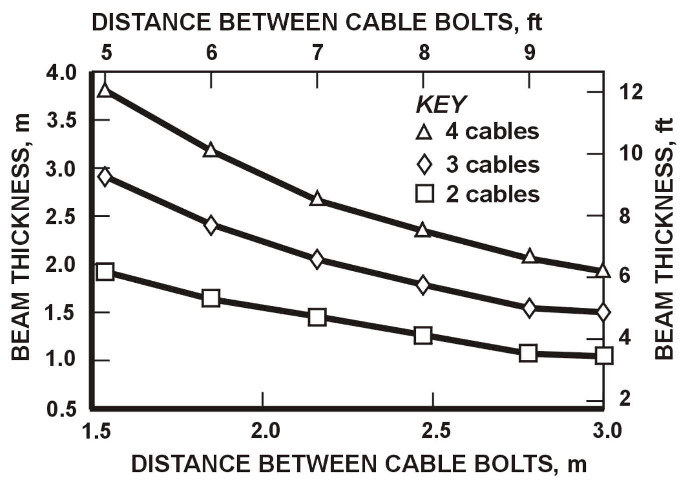

Figure 13.–Cable support design chart, where the effective width of opening (W_e) = 7.6 m (25 ft) and rock density (γ) = 2,403 kg/m^3 150 lb/ft^3 [McDonnell et al. 1995].

PERFORMANCE OF CABLE BOLTS WITH RESPECT TO TAILGATE INTERACTION ZONES

When using cable bolts for secondary support in the tailgates, there are three zones that must be considered in evaluating the design and performance of the cable systems. These zones are the outby abutment zone for both vertical and horizontal stress, the shield zone from the face to the back of the shield, and the cave zone. Each zone has different performance requirements, and therefore, the cable system much be designed to meet these requirements. If problems do occur in the tailgate that results in the shutdown of the face, the cost to the operation in both downtime and clean-up can be high.

In the forward abutment zone, the cable support must maintain an open tailgate entry and prevent any major roof falls that impede the use of the tailgate as a secondary escapeway and for ventilation (figure 14). In the abutment zone, the cable loads will depend in part on the geology, depth, and pillar design. This zone receives the most support from the pillars and the panel and may be up to 45 to 60 m (150 to 200 ft) wide. The depth will control the pillar yield zone that develops along the tailgate entry and at the face, where an increase in the entry or intersection span will, in general, result in more roof separation and movement. This yield zone will obviously increase near the face. With pillar design, abutment pillars will offer the most support to the tailgate entry. In many situations, little load or roof movement will be seen. With a yield pillar adjacent to the tailgate, significant roof movements and cable loads can develop when the pillar yields. Often this will include lateral movement that the cables must withstand.

Geologic structures such as joints, faults, and sand channels, can cause locally high loads to develop on the cable support and can result in cable failures and even small roof falls. These roof control problems will usually begin with a sudden increase in the rate of vertical loading from the abutment. In such cases, some additional support may be required locally if the cables fail. Although horizontal stress is not typically a problem in the tailgates because of the adjacent caved panels, horizontal stress damage to the roof may have been caused by a previously mined panel and the damage may have been transmitted through the crosscuts to the tailgate entry. Lateral roof movement may occur just outby the face and result in additional cable loads or even cable failure in shear. Furthermore, damage done to the roof in this zone and subsequent loads on the cables will impact performance in the other zones. In general, in the abutment zone, the highest loads and roof movements will be seen in the intersections although with yield pillars, this may occur at mid-pillar.

In the zone from the face to the back of the shields, performance requirements are very similar to those for the abutment zone —the area must remain open as an escapeway and for ventilation (figure 15). However, support of the panel has been removed and replaced by the shields, and this creates an opportunity for the roof to move because of the loss of support. Therefore, higher cable loads will develop here than in the abutment zone. This is the situation for which the cable system should be designed. The degree of roof movement and separation will depend to a large extent on the geology and any previous damage done to the roof. Cables often begin to load in this zone when there was little movement in the abutment zone, especially when abutment pillars are adjacent to the tailgate. Maximum loading and roof movement are seen just as the cables go behind the shields.

In many operations, there is no need to maintain the tailgate behind the shields and the performance of the cable bolts in this

Figure 14.–Tailgate abutment zone outby the face supported with cable bolts.

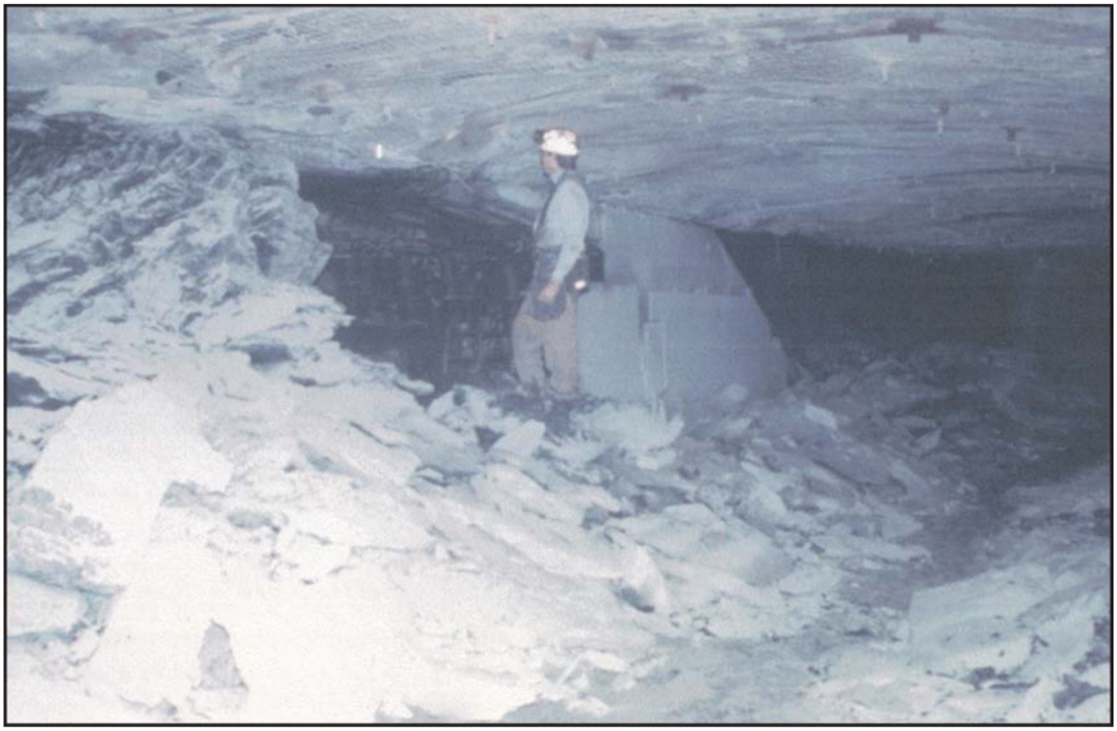

Figure 15.–Tailgate shield zone supported by cable bolts.

area is not a factor. However, in some mines, ventilation requirements necessitate that the gateroad be kept at least partially open to the nearest crosscut behind the face, a distance of usually 30 to 45 m (100 to 150 ft). The maintenance of this section of the tailgate by the cables is dependent to a large degree on the geology and the cave and only to a limited degree on cable system design. In the tailgate adjacent to the cave, the roof develops into a cantilever that must be supported. If the roof is not strong enough and the cave goes above the cables then the cantilever could fail and close most of the entry (figure 16). If the roof is strong enough to maintain the cantilever, then the cables will help to maintain any lower weaker roof. The critical factors are whether the cave develops above the cables and if the zone is strong enough to maintain the cantilever. The cables probably add little overall strength to the cantilever. However, there are cases where the entry has stayed open more than 45 m (100 ft) behind the face [Koehler et al. 1996; Martin et al. 1996; Mucho et al. 1996] (figure 17).

Geologic structures such as joints can cause periodic failure of even a competent roof behind the face. Essentially, there is no guarantee with cables that this zone can be maintained to the next crosscut. If the tailgate must be kept open, then other types of support should be considered. However, even if the roof fails, a portion of the tailgate alongside the pillar will usually remain open, although this is a restricted area [Molinda et al. 1997].

DESIGN BASED ON TEST SITES

Test sites have been used to establish, evaluate, and confirm cable system designs. Besides being a good practice, test sites may be required by MSHA when cable systems are used for the first time at a mine. Test sites can also be used to modify existing cable system designs. Although observation can be used to judge the successes of the design, instruments that monitor both roof movement and separation and cable loads to quantify the results and confirm the design are preferred. Monitoring of roof movement is especially useful when evaluating cable length. Final cable system designs should be based on evaluation of test sites.

The design most used in tailgates has been one in which there are four cable bolts per row. Although three bolts can provide adequate support with the same safety factors, four bolts per row have certain advantages. This number provides good coverage across the entry, thus maintaining an effective support front, especially as the cable row goes behind the face. Also, in a given row, the failure of a single cable represents a loss of support of only 25% with four cables per row and 33% for three cables per row. Although the cable support is designed on the basis of an area of support, as the support goes behind the face, the performance of a single row or the line of support becomes important. Finally, with the use of double-boom bolters, it is usually more efficient to install four bolts than three bolts per row. Row spacing has varied from 1.2 to 1.8 m (4 to 6 ft). With row spacings wider than 1.8 m (6 ft), interaction between rows can be lost and the effectiveness of the reinforcement as a system reduced.

Additional support to the crosscuts must also be considered when using cable bolts because of the increased spans in the intersections and any damage in the crosscuts from previous panels. Generally, this support can consist of one or two rows of cables installed in the crosscuts. Instead of (or in conjunction with) the cables, cribs can also be set in the crosscuts. Another modification to the design is to angle the outside cable bolts toward the pillars and panel. This angle is usually about 10° from vertical and will allow the anchorage to be in a more stable roof zone.

DESIGNS FOR LATERAL MOVEMENT

Cable bolts will offer resistance to lateral movement, although shear is resisted to a large degree only after the peak rock strength has been exceeded. Essentially, the rock has failed and is now mobilized where the cables will offer significant post-failure resistance by significantly increasing the residual shear of the rock [Goris et al. 1995, 1996]. However, in some cases, because of large lateral deformations, the cables may not be able to stop or limit this displacement prior to failing. At a mine in western Colorado, a tailgate supported with cable bolts was subjected to large lateral deformation. This occurred as the adjacent panel was being mined, with the horizontal stress abutment in the headgate causing roof damage not only to the headgate, but also to the tailgate of the next panel through the crosscuts [Dolinar et al. 1996]. This panel was supported with cable bolts and rigid trusses. About 0.3 to 0.45 m (1 to 1.5 ft) of lateral movement occurred in places along this entry. All the rigid truss cross bars had been thrown from the anchor bolts while about 20% of the cable bolts failed. It is estimated that the cables withstood about 5 to 10 cm (2 to 4 in) of lateral movement before failure. These are very tough ground conditions where few support systems could be expected to prevent movement of this magnitude. With shear or lateral movement, the flexibility of the cable bolts is not fully utilized.

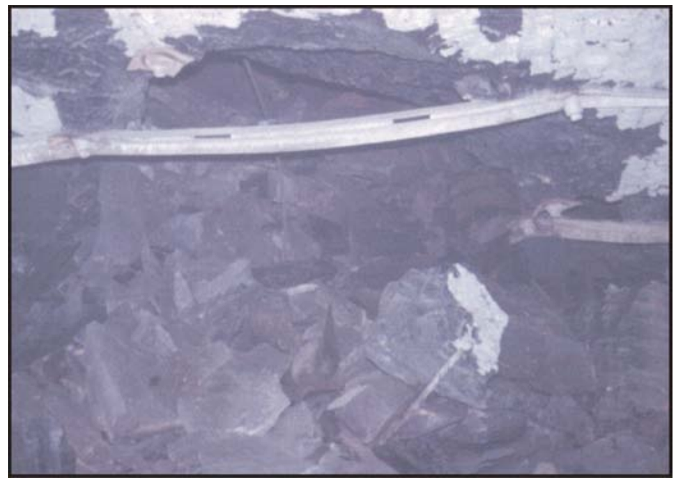

Figure 16.–Tailgate behind the shields has caved though supported with cable bolts.

Figure 17.–Tailgate behind the shields kept open with cable bolts.

Obviously, it may be difficult or impossible to stop such large movements with support, and other approaches may need to be considered to prevent support failure. If the support does not fail, then it can support the damaged roof by suspension. However, there are some alternative approaches that can be used to minimize the impact of large lateral movements on cable supports. One approach is to keep the cable bolts out of the highest zones of shear or differential lateral movements that occur near the edge of the pillar. To do this, cables can be positioned 0.6 to 0.9 m (2 to 3 ft) from the rib. Another successful approach is to use cable bolts with a yielding head. These heads will allow the cable system to yield in a controlled manner at loads below the ultimate capacity of the cables [Tadolini and McDonnell 1998; Vandekraats and Watson 1996]. Some of these heads will allow up to 50 cm (20 in) of controlled movement, thus letting the cable deform with the roof. With nonyielding cable heads, the head will lock in the bolt, and stretch in the system must take place as the bolt goes into a yield condition.

CASE HISTORIES OF CABLE BOLTS AS SECONDARY SUPPORT IN TAILGATES

The following section gives case histories for tailgates supported with cable bolts either as the main or only secondary support system. In each of these cases, a 1.52-cm (0.6-in) in diameter cable bolt with an ultimate capacity of 260 kN (58,600 lbf) was used. At these test sites, cable loads were monitored usually with hydraulic U-cells and pressure pads, while differential roof sag measurements were made within and above the cable horizon. Roof-to-floor convergence measurements were also obtained at some sites. Usually, several intersections as well as midpillar locations were monitored.

CASE HISTORY 1

This mine is located in western Colorado and used a yield-abutment pillar configuration. Three different cable bolt system designs were tested in a 274-m-(900- ft-long) section of the tailgate. They included a passive system, a stiff passive system (increased grout anchorage length), and a tensionable system [McDonnel et al. 1995; Tadolini and Koch 1993; Tadolini and Koch 1994]. The roof geology consisted of 1.2 m (4 ft) of coal overlain by 0.6 m (2 ft) of silty shale and 1.2 m (4 ft) of interbedded shale, silty shale, and sandstone. After evaluating the geologic data on the roof, it was thought that roof separation would most likely occur in the silty shale although separation might also develop higher in the interbedded shale and sandstone. Above the immediate roof was a 4.9-m-(16- ft-) thick massive sandstone. This sandstone provided a good anchorage from which to suspend the lower roof. The entry width was 5.8 m (19 ft), but with pillar yield, the effective width for design was assumed to be 7.9 m (26 ft).

Figure 18 shows a tailgate entry cross section with the cable configuration where four cables per row were installed on 1.5-m (5-ft) row spacings. The cable bolts were 4.9 m (16 ft) long. With this configuration, the cables would have just enough capacity to hold up 3 m (10 ft) of rock if the separation occurred at this level and the full weight had to be supported by the cables. For surface control, bearing plates, monster mats, and wire mesh were used. For the passive site, the cables were installed with a resin grout length of 1.7 m (5.7 ft), which assured adequate anchorage in the sandstone and resulted in a free cable length of 3.1 m (10.3 ft). For the stiff passive system, the resin anchor length was 3.7 m (12 ft), leaving only 1.2 m (4 ft) of free cable length. In the tensional section, the resin length was again 1.7 m (5.7 ft) with a free cable length of 3.1 m (10.3 ft). These cables were tensioned to 35 kN (8,000 lbf). Because of the thrust from the roof bolter, the cables in the passive sections were installed with 6.7 to 22.2 kN (1,500 to 5,000 lbf) of load.

With panel mining in the passive area, the maximum total roof separation was about 0.6 cm (0.25 in) in an intersection. In the stiff and tensional areas, the maximum total separation was between 3.2 and 3.8 cm (1.25 and 1.5 in) in both sections. The movement and separation took place within 30 m (100 ft) of the face and did not affect functioning of the tailgate or load the support beyond the cable's strength. Cable loads in the passive section ranged from 0 to 107 kN (0 to 24,000 lbf) and averaged 21.3 kN (4,800 lbf). In the stiff section, cable loads alongside the shields ranged from 71 to 116 kN (16,000 to 26,000 lbf). For the tensioned cable site, the loads ranged from 18.2 to 151 kN (4,100 to 34,000 lbf). However, in the tensioned test site area, several geologic features, including coal spars and a clay dike, were observed in the roof. In one small area, the cables were loaded to over 133 kN (30,000 lbf), while the roof was broken and fractured. In this area, some cables appeared to have failed or the cable heads had slipped. Nine wood posts were set to provide additional support, although the section through the area was mined without incident. In the passive area, the roof remained open 30 to 45 m (100 to 150 ft) behind the face, while for the stiff system, the entry remained open about 30 m (100 ft) behind the face.

All three systems worked extremely well and were able to keep the tailgate open through the abutment zone, alongside the shields, and even for a distance behind the shields. However, from these test sites, it could not be determined if there were any difference in performance among the systems. In the tensional area, localized geologic structure in the immediate roof did induce higher cable loads and possibly cable failures, but no significant problems were apparent in controlling the roof.

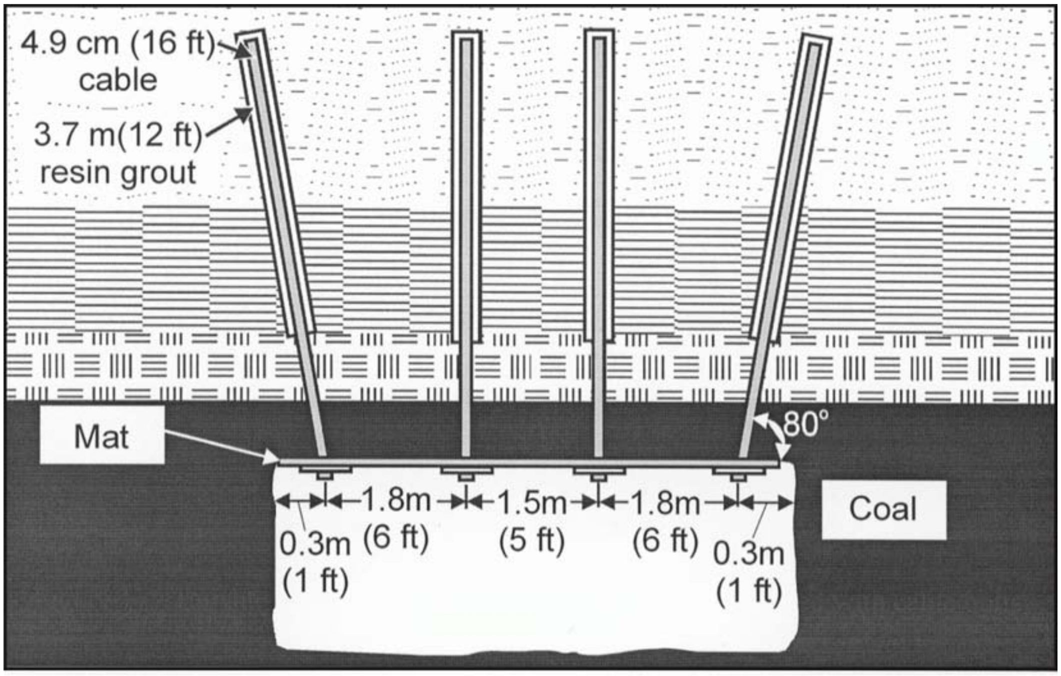

Figure 18.–Cross section of tailgate entry showing cable bolt design for case history 1.

CASE HISTORY 2

Case 2 was a mine located in Utah with a double entry yield pillar configuration in the tailgate [Tadolini and Trackemas 1995]. The depth of cover at the mine averaged about 460 m (1,500 ft). Because of the yield pillar, the effective opening width was estimated at 9.7 m (32 ft). The geology of the immediate roof consisted of thinly bedded siltstones, sandstone, and mudstones along with carbonaceous material to a depth 1.2 to 1.8 m (4 to 6) ft. This was overlain by a sandstone containing bands of carbonaceous material. Sand channels cut into the immediate roof, but not into the coal, and affected roof quality locally.

Figure 19 shows the geology as well as a cross section of the entry with the cable system design. The cable bolts were 4.9 m (16 ft) long with four bolts per row and a row spacing of 1.5 m (5 ft). The resin anchor length was 1.5 m (5 ft), resulting in a free cable length of 3.4 m (11 ft). This free cable length allowed for greater cable elongation in the high-stress and deformation environment caused by crushing of the yield pillar. This cable system design would support a roof thickness of up to 3.3 m (10.9 ft) based on a dead weight load.

The installed cable loads averaged 15.1 kN (3,400 lbf). With panel mining, cable loads ranged from 0 to 178 kN (0 to 40,000 lbf) during the life of the test site. Loading and unloading of cables occurred in the same row, while shearing in the roof was observed at different depths. This shearing action resulted in differential lateral movement between roof layers and could explain the loading and unloading of the cables. In the area of the sand channels, several cables failed because of this differential movement, and some standing support was added. Separations were observed in the mine roof, but never above the anchor horizon. Up to 10 cm (4 in) of overall roof separation was seen, most of which occurred between the mudstone-sandstone layers within the lower 1.2 to 1.8 m (4 to 6 ft) of the roof. This was within the elongation capacity of the cables. However, cable loads up to 275 kN (40,000 lbf) and cable failures indicated that this level of elongation was approaching the limit of the cable system especially as it was developed by shear. Despite this high-deformation environment, the tailgate was kept open and functional with the cable support even under the sandstone channels (figure 20).

In a series of initial experiments with cable support, the mine installed a double row of wood cribs with spacing that was increased from 1.8 to 6.1 m (6 ft to 20 ft) through the tailgate test area. Finally, a section with no cribs and only cable support was tested. The results of these trials indicated that the best roof conditions were when there few or no cribs. The hypothesis was that the standing support damaged the roof as it resisted the main roof-to-floor convergence. The roof damage and subsequent hazardous conditions resulted as the cribs were compressed against the roof with such force that it caused the immediate roof to break.

CASE HISTORY 3

Case 3 is a mine in western Colorado using a three-entry system with two abutment pillars [Dolinar et al. 1996]. The roof generally consists of a thinly bedded siltstone (stack rock) and massive, fine-grained sideritic siltstones that grade laterally

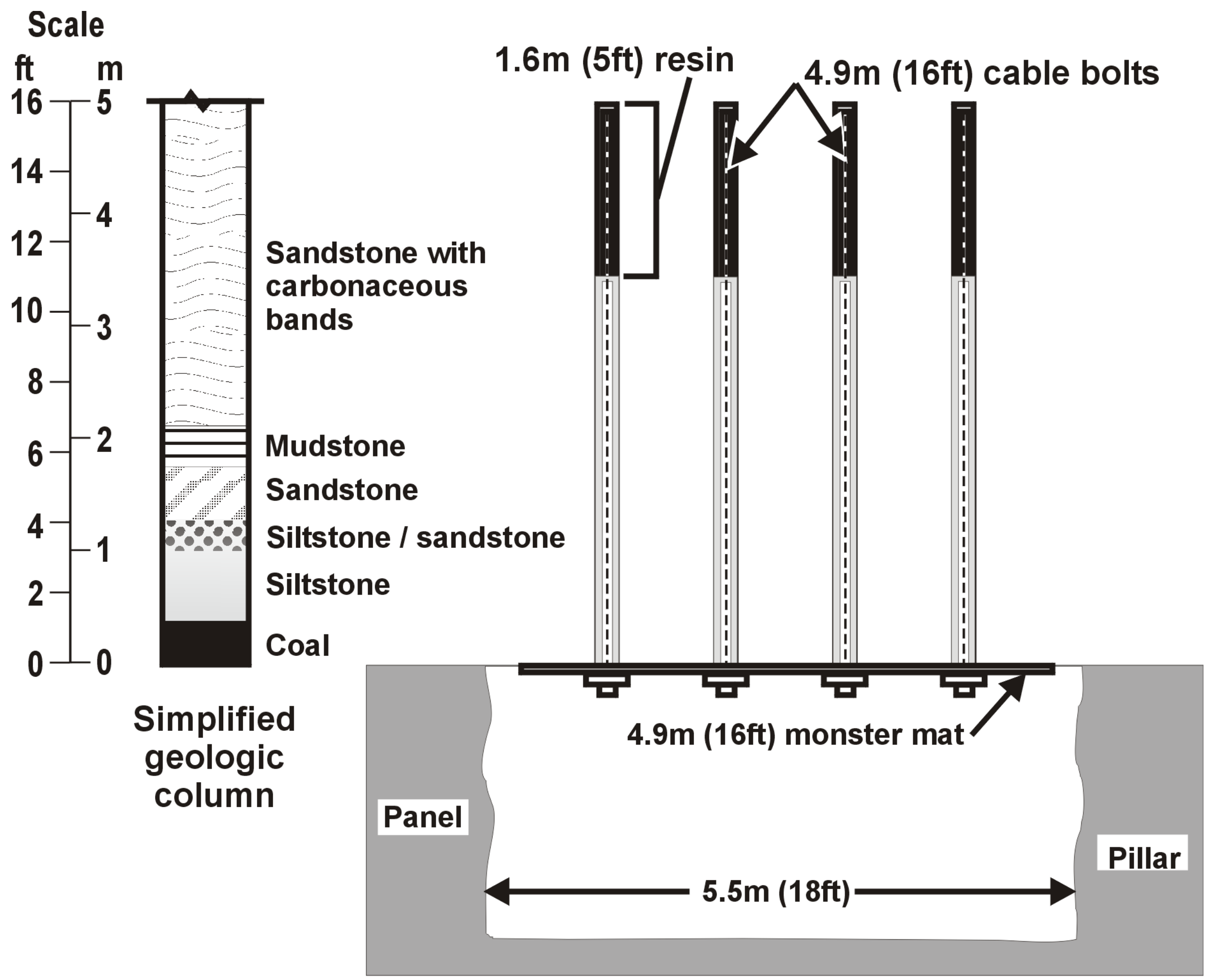

Figure 19.–Cross section of tailgate entry showing cable bolt design for case history 2.

to a dark gray limestone and to sandstone. Another seam overlies the mined seam at distances ranging from 0.9 to 5.5 m (3 to 18 ft). The thickness of the interburden is important to the roof control problems that develop at the mine. The mine is also subjected to high horizontal stresses with a ratio of maximum horizontal to vertical stresses of 1.7.

Test sites were established in tailgates of two adjacent panels. Figure 21 shows the geologic column and an entry cross section with the cable support design. The cables were 4.9 m (16 ft) long with four cables per row with a row spacing of 1.5 m (5 ft). Anchorage length was 1.5 m (5 ft), leaving a free cable length of 3.4 m (11 ft). In addition, high-capacity dome-bearing plates as well as monster mats were installed for surface control. At the initial test site, the interburden was 1.8 m (6 ft). Since this was the first use of cables at this operation, a double row of cribs was installed as additional support. Even with the crib support, cable loads averaged 98 kN (22,000 lbf), while a total of nine cables failed in the 122-m (400-ft) test zone. The failure was due to the large lateral movements that occurred in the interburden. Roof separation ranged from 2.0 to 5.3 cm (0.8 to 2.1 in) and occurred between 1.2 to 1.8 m (4 to 6 ft) into the roof. Cribs in the test site were highly deformed by the lateral roof movement.

At the second site in the adjacent panel, interburden thickness was 5.5 m (18 ft). The maximum increase in cable loads was 56.9 kN (12,800 lbf) with an average increase of only 2.2 kN (500 lbf). The different sag stations showed less than 1.8 cm (0.7 in) of movement. Roof conditions remained excellent, and no roof control problems were encountered in the entire cable section. Often the roof would remain standing one or more crosscuts behind the face, a distance of about 45 to 90 m (150 to 300 ft).

The difference between the two sites was the interburden. The thinner interburden consisted of weaker layers rock (stack rock) subject to horizontal stress damage and lateral movement. With the extensive lateral movement at the first site, a combination of cribs and cables did maintain the tailgate. However, the cribs did little to stop the lateral movement, and the cables may have been able to maintain the gate road without the cribs. With less lateral movement, the gate road probably could have been easily maintained with the cable support. In a third tailgate with a thin interburden and supported only by

Figure 20.–Two entry yield pillar tailgate showing the abutment zone outby the face, supported with cable bolts

cables and rigid trusses, large lateral movements were also encountered. Several cable bolts did fail along with all the rigid trusses. This occurred as the adjacent panel was mined. In this tailgate, lateral movement of between 0.3 to 0.46 m (1 to 1.5 ft) occurred and when the panel was mined, a roof fall did occur in the tailgate that resulted in some delays of the longwall. Under these very severe ground control conditions, additional support may be required although these are tough conditions for most support systems to control.

CASE HISTORY 4

Case 4 is a mine located in Utah with a yield-abutment pillar configuration [Koehler et al. 1996]. The geology of the immediate roof consists of 0.3 to 0.6 m (1 to 2 ft) of coal and 0.3 to 0.6 m (1 to 2 ft) of mudstone overlain by a 0.3 to 0.6 m (1 to 2 ft) layer of gray sandstone. Above this was a white sandstone with occasional shale bands to a depth of at least 6.1 m (20 ft). The cable support design consisted of 4.3-m-(14- ft-) long cables with four cables per row on 1.5-m (5-ft) row spacings. The resin anchor was 1.2 m (4 ft) long, leaving a 3 m (10 ft) length of free cable. T5 channel was used for surface control.

The installation loads on the cables averaged 12.9 kN (2,900 lbf). During mining of the panel, load increases on the cables ranged from 0 to 118 kN (0 to 26,500 lbf). In the intersections, the cable loads increased an average of 7.1 kN (1,600 lbf). However, the highest cable loads were associated with a near-vertical joint located near a mid-pillar instrument site. Maximum cable load increase was 118 kN (26,500 lbf) while the average increase was 66.7 kN (15,000 lbf). Higher cable loads were measured along the pillar side that may be attributable to the yield pillar and the roof breaking adjacent to the pillar in reaction to pillar yielding. This may also explain why the largest loads were seen at the midpillar locations. The maximum roof movement measured in the intersection was only 1.0 cm (0.4 in). Through the test section, there was 10 to 15 cm (4 to 6 in) of roof-to-floor convergence because of the yield pillar. Behind the shields, the tailgate would remain open for 15 to 41 m (50 to 135 ft). Then the entry would cave to just behind the shields. This distance was controlled by a near-vertical joint set subparallel to the face. This was the same joint set that resulted in the highest cable loads at the test site. Outby the cave, the tailgate remained open with no ground control or roof problems.

CASE HISTORY 5

Case 5 is a mine in southern West Virginia with a three-entry abutment pillar configuration. The immediate roof at the mine makes a transition from sandstone to shale [Mucho et al. 1996]. In some areas, the sandstone appears to be massive, while in others it appears to be highly laminated, fossilized, and interspersed with coal streaks. The horizontal stress is high enough to cause damage at some locations, especially with a thinly laminated roof. The cable system design consisted of 3.7-m (12- ft-) long cables with four cables per row on a 1.8-m (6-ft) row spacing. These bolts were tensioned by the use of a threaded rebar head at the bottom end of the cable. The resin anchor was 1.5 m (5 ft) long.

When the panel was mined, less than 0.25 cm (0.1 in) of roof separation was recorded and cable loads increased on average only 8.9 kN (2,000 lbf). Maximum loads and roof separations occurred as the instruments went behind the shields. The tailgate roof area was extremely stable outby the cave. Behind the face, the roof stayed up for a distance of 23 m (75 ft) before the roof caved to just behind the shields. This cyclical caving of the tailgate roof occurred throughout the test area. In addition, when the adjacent panel was mined, there was almost no floor heave in the cable section (tenths of inches) as compared to the crib section where several inches occurred.

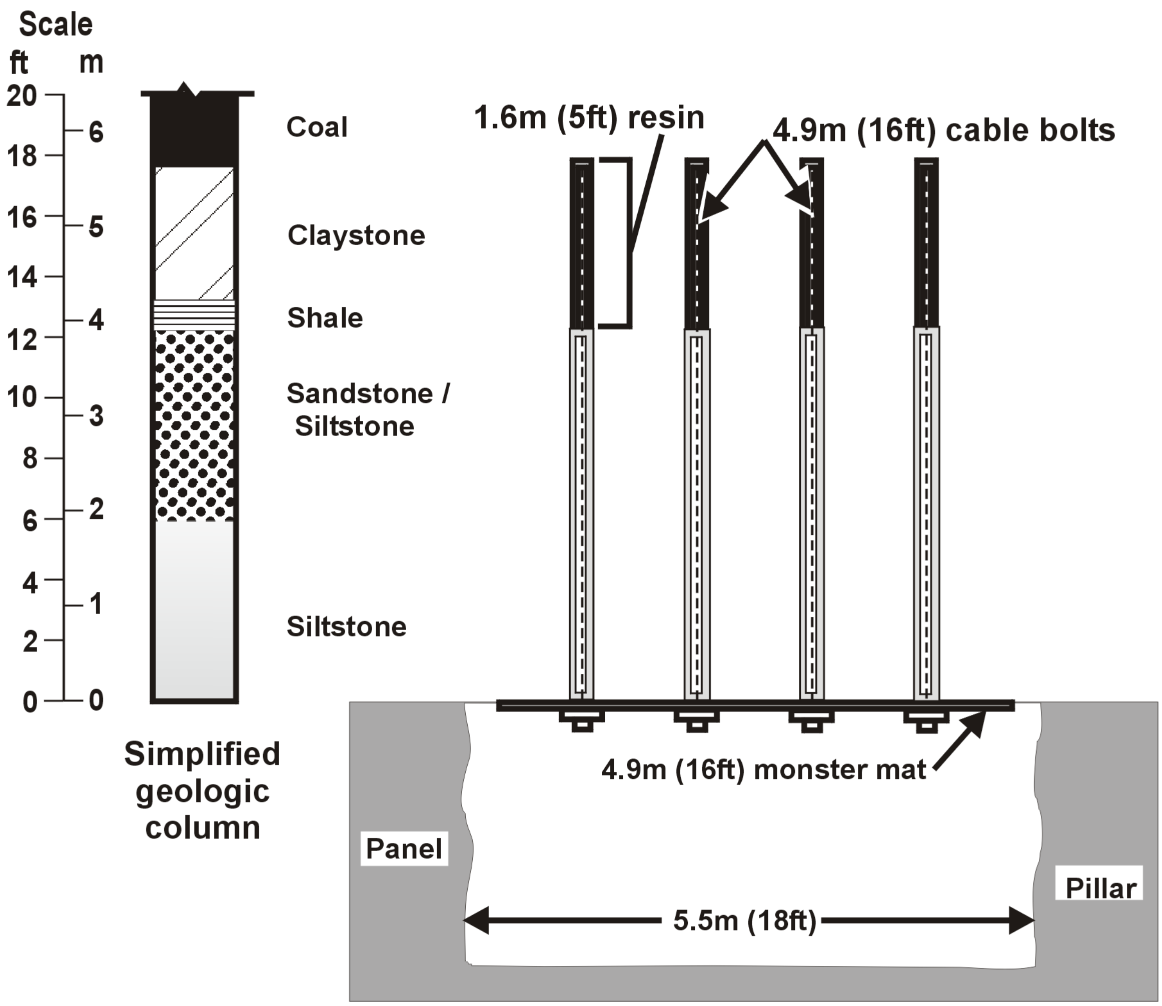

Figure 21.–Cross section of tailgate entry showing cable bolt design for case history 3.

CABLE TRUSSES

Cables trusses are anchored over the pillars and panels outside the potential failure envelope and provide resistance to roof movement along the roof line. Because cable trusses have a high strength and flexibility and a low stiffness, they can survive in a high-deformation and stress environment where other supports would fail. Essentially, cable trusses move and deform with the rock with the truss providing only limited resistance to vertical movement [Scott 1994]. Some systems can be pretensioned, but tensioning is probably not significant to improved ground control, but that assures the truss is tight when installed and therefore can respond immediately to roof movement. Cable trusses have been used in mines since at least the 1970s, but on a limited basis [Scott 1989; Mangelsdorf 1982]. However, in the 1990's with the advent of cable bolting in U.S. coal mines, newly designed cable truss systems that can be installed with roof bolters and anchored with resin grout cartridges are now being used much more extensively as supplemental support, especially in headgate entries.

DESCRIPTION

Cable trusses are constructed from a seven strand cable usually having a diameter of 1.52 cm (0.6 in) and an ultimate strength of 260 kN (58,600 lbf). However, cables with a diameter of 1.27 cm (0.5 in) are also used. Cable trusses are normally installed in a hole 3.5 cm (1-3/8 in) in diameter, although the system can be installed in a 2.5-cm (1-in) hole. The drill holes are typically up 2.4 m (8 ft) deep and drilled 0.6 m (2 ft) from the rib at an angle of about 45° over the coal rib. Domed and grooved bearing plates usually 15 by 40 cm (6 by 16 in) in size are used as bearing surfaces for the rock and cable. This allows for a two-point contact along the roof at installation. At the drill hole-roof interface, the cable will also be in contact with the rock, and a crushed zone may develop as the cable loads. Cable trusses may be composed of either single or multiple pieces, which affect how the systems are installed but not their function.

A one-piece truss consists of a single, continuous cable with anchorage buttons and a resin mixer on each end of the cable in the anchor zone (figure 22) [Dolinar et al. 1996]. The truss uses a no-spin system to mix the resin while a push button on the cable and a special bolter wrench allow for the insertion of the cable into the hole and through the resin with the roof bolter. The procedure is that one end is installed, and the resin is allowed to cure. Then the other end of the cable is placed into the hole on the opposite side of the entry and thrust through the resin with the roof bolter.

With this system, installed cable truss loads ranging from 15.1 to 51.6 kN (3,400 to 8,200 lbf) have been measured. The goal is not to develop large loads in the roof but to simply tighten the truss so that it will provide some immediate resistance to rock movement.

The three-piece cable truss consists of two angle cable bolts and a horizontal cable member (figure 23) [Oldsen et al. 1995]. The angle bolts can be constructed with nuts or birdcages for anchorage as well as resin keepers. The cable bolts are pushed and rotated into the hole and through the resin using a special wrench and a roof bolter. A splice tube assembly is attached to the angle and the horizontal cables, which allows the pieces to be connected and the system to be tensioned. The housing and wedge assembly that form the cable heads are installed in the field and allow the cables to be tensioned against the splice tube. A tensioner powered by the hydraulics of the bolter is used to tension the system and at up to 71.2 kN (16,000 lbf) of preload.

ANALYSIS OF CABLE TRUSS LOADING

The loads developed in a truss can be evaluated by simple statics. Figure 24 shows a simple free-body diagram of the loads for a half of a truss. The following equations can be used to describe the relationship between the reaction force R broken into horizontal and vertical components and the cord tensions.

$$Y_r = T \sin \propto \tag{8}$$

$$X_r = H - T \cos \propto \tag{9}$$

where T = tension in the diagonal member,

H = tension in the horizontal member,

Y_r = vertical reaction force,

X_r = horizontal reaction force,

and $\propto$ = angle of inclination of the cable.

The reaction force R may be a compilation of several forces, especially in the case where the inclined cable bears against the roof at the drill hole. However, these equations are still valid for describing the vertical force Y_r applied to the rock by the truss or to the truss by the rock [Mangelsdorf 1979]. With a cable truss, the tension transfer between the horizontal cord and the diagonal cords becomes more complex. Figure 25 shows a free-body diagram for the more complex loading conditions for a cable truss. Essentially, tension load transfer will take place by slippage of the cable over the bearing block or plate, the bearing block or plate over the rock, or the cable over the rock at the edge of the borehole. These load transfers are dependent on overcoming these frictional forces. Because of these complex loading conditions, the tension in all three legs of the truss must be measured along with roof sag to evaluate field performance.

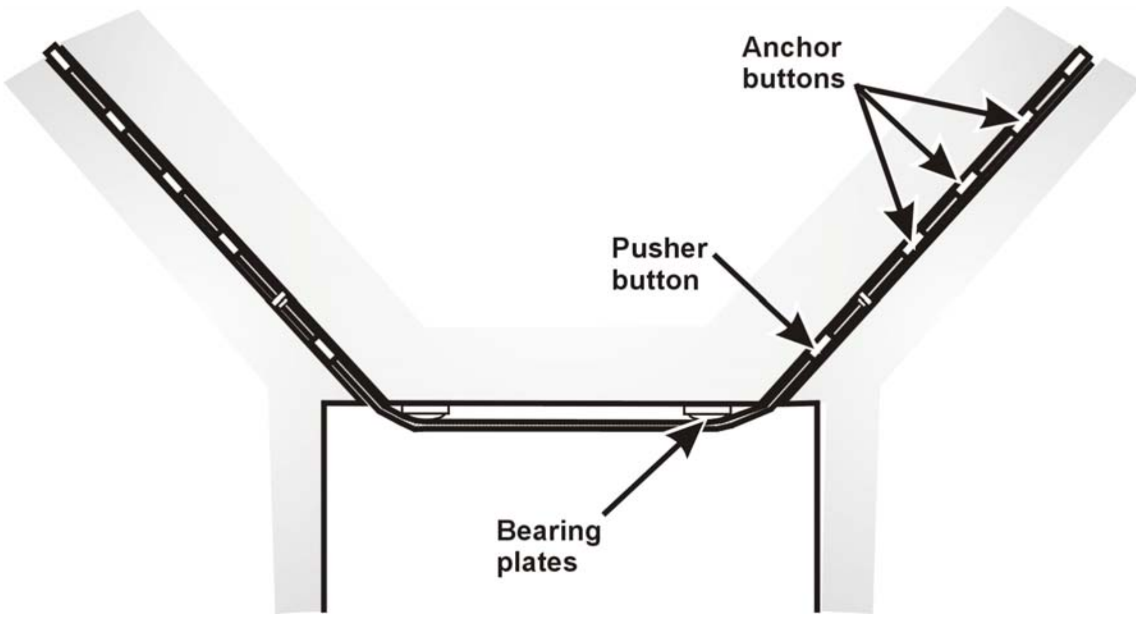

Figure 22.– Single piece cable truss.

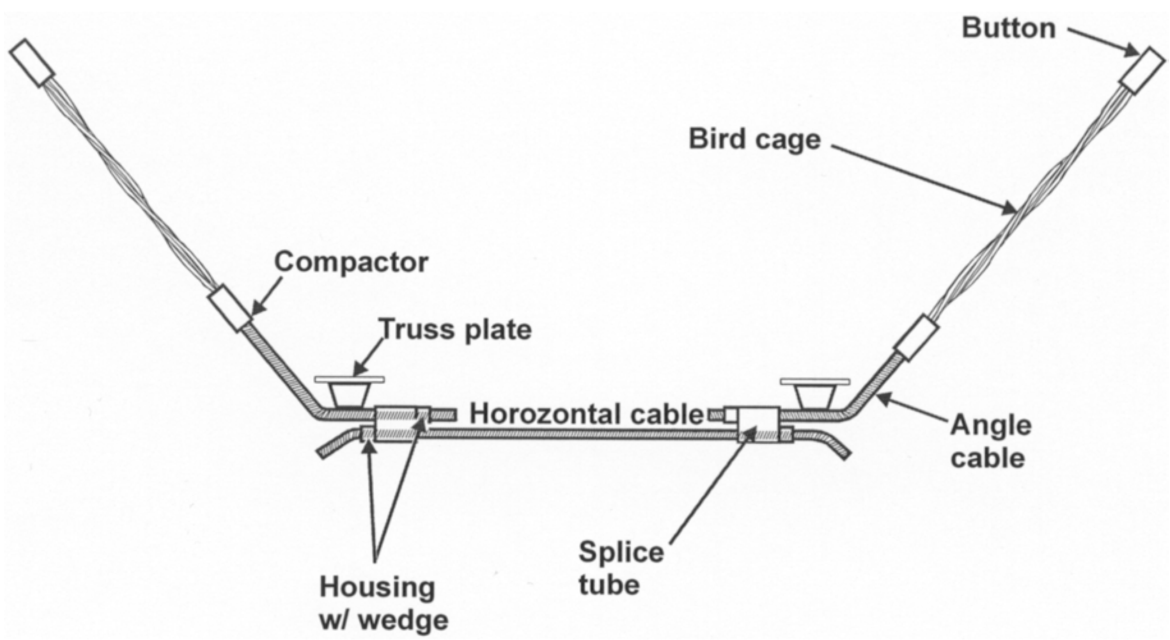

Figure 23.–Three piece cable truss.

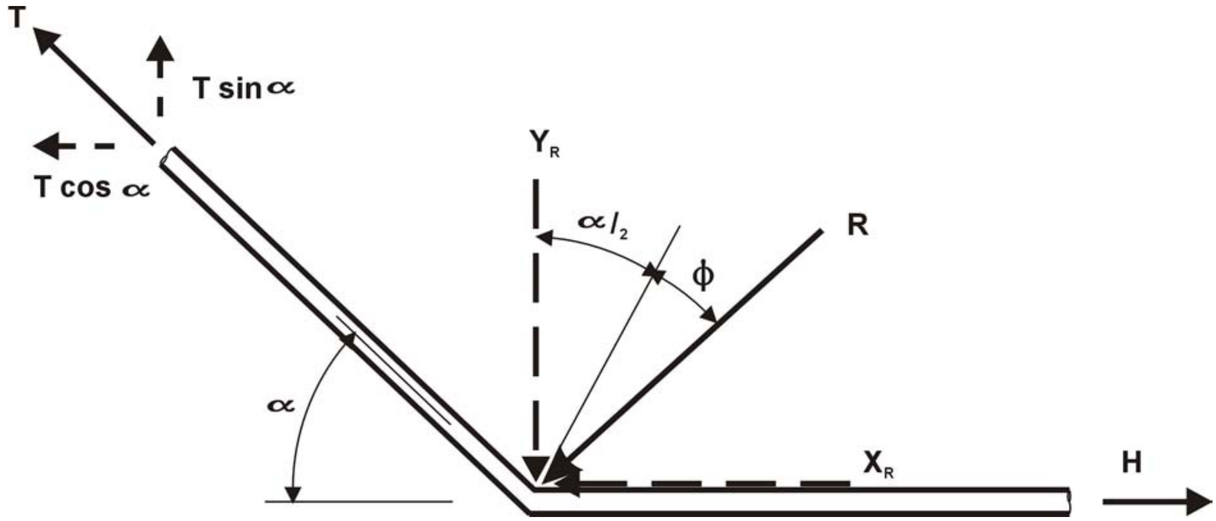

Figure 24.–Free body diagram of half truss showing the truss loads.

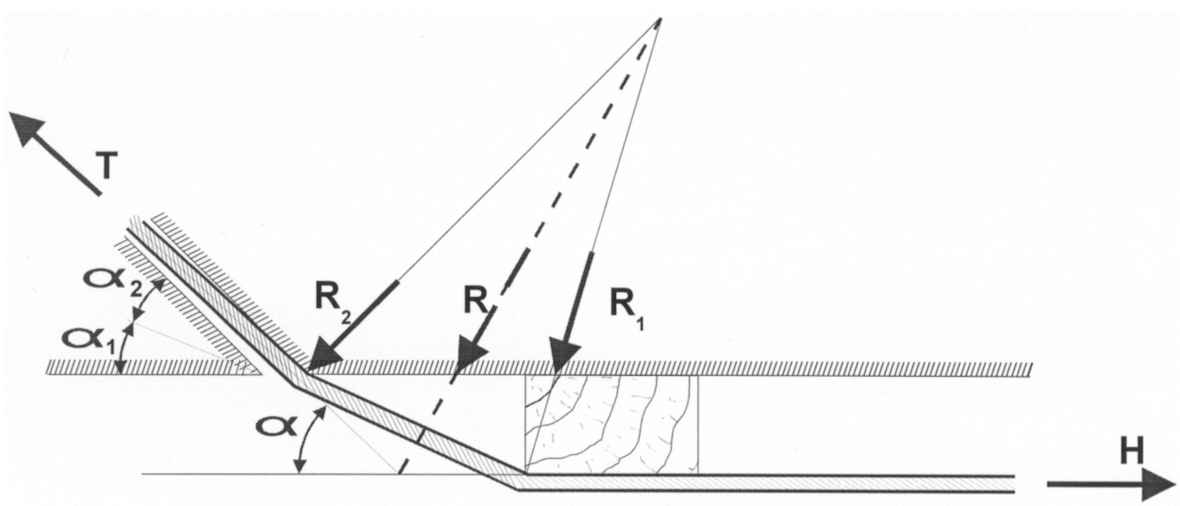

Figure 25.–Free body diagram of half cable truss, showing static loads that develop.

LABORATORY TESTS TO EVALUATE CABLE TRUSS PERFORMANCE

Laboratory investigations have been conducted to evaluate the loading characteristics of cable trusses where special load frames have been constructed to approximate the field conditions. The results from a series of tests conducted at the University of Pittsburgh indicate that only about 80% of the load is transferred from the angle member to the horizontal member as a result of friction across the contact blocks or bearing plates [Mangelsdorf 1979]. In these tests, the angle member was at a 45° angle to the horizontal. When tested to failure, the load in the diagonal cord member was 88.9 kN (20,000 lbf) where the ultimate strength of the 1.27-cm (0.5-in) in diameter cable was 102.3 kN (23,000 lbf). The angle member achieved only 87% of the ultimate load of the cable, with failure resulting from bending of the cable over the contact block, which caused a point of stress concentration and reduced the range of inelastic deformation of the truss. The truss had reached the yield point at the same approximate level as cable yield. At failure, the vertical load calculated from the load measured in the cable diagonal was 62.3 kN (14,000 lbf). The measured vertical stiffness for a half truss was 8.8 kN/cm (2.5 tons/in) and represents a stiffness of 17.5 kN/cm (5 tons/in) for the full truss.

Investigators at the University of West Virginia have also conducted laboratory tests with cable trusses in a specially designed truss frame [Oldsen et al. 1995]. The results of these tests were similar to those in the University of Pittsburgh study. This study did however, provide some further insight into frictional losses and load transfer between sections of the cable and applied loads. In these tests, a 1.52-cm (0.6-in) in diameter cable truss with an ultimate strength of 260 kN (58,600 lbf) was used, although the cables were not taken to failure. Again, the angle between the angle and horizontal members was 45°.

However, there are differences of interpretation of the data regarding the vertical load capabilities of the truss. In the University of West Virginia report, it is stated that the load on the diagonal is 222 kN (50,000 lbf) when the total applied vertical load or plate loads is 400 kN (90,000 lbf). There are two problems with this interpretation. First, the data are being extrapolated beyond the actual test data. Second, the plate loads are assumed to be the vertical loads. Extrapolation beyond the test data can at times be questionable, while using loads applied at the plates as vertical stress involves uncertainties about the frictional conditions within both the jacks used to load the trusses and the test frame, as well as the angle of the applied load between the test frame and the cable. Essentially, the only reliable measurement of vertical load should be that calculated from the diagonal member. By using the diagonal load, the result is that the vertical truss load is only 311 kN (70,000 lbf). Assuming this is near cable failure, the ratio of a vertical load of 311 kN (70,000 lbf) to an ultimate cable load of 260 kN (58,600 lbf) is 120%. The ratio of the vertical load to the ultimate cable strength from the University of Pittsburgh tests was 124.6:102.3 kN (28,000:23,000 lb) or 122%. These calculations were based on symmetrical loading at the plates.

FIELD EVALUATION OF CABLE TRUSS PERFORMANCE

Headgate

Trusses are now being used extensively to provide supplemental support to the headgate entry where the damage to the headgate entry is often the result of high horizontal stresses [Mark et al. 1998; Oldsen et al. 1995]. The ability of cable trusses to handle headgate conditions is illustrated by the following case.

A mine located in western Colorado had roof damage in the headgate ahead of the face as the panel was mined [Dolinar et al. 1996]. This was the result of horizontal stress concentration ahead of the face and geologic features susceptible to stress damage. (See case 1 for a more detailed description of the geology.) A single-piece cable truss with a diameter of 1.52 cm (0.6 in) was installed on 1.2-m (4-ft) centers in the headgate entry (figure 26). To evaluate loading during installation and as the panel was mined, special cable strain gages were installed on the horizontal section of some of the trusses. The installed load on the trusses ranged from 15.1 to 36.9 kN (3,400 to 8,200 lbf). From mining, the maximum load was 74.7 kN (16,800 lbf) for a truss just inby the face, an increase of 55.2 kN (12,400 lbf). This shows that the cable trusses were loading and resisting the roof movement. The cable trusses were able to control the roof conditions that developed in the headgate successfully despite the lateral movement and roof damage (figure 27).

Other investigators have measured 17.8 to 26.7 kN (4,000 to 6,000 lbf) of increase resulting from horizontal stress damage and cutters in headgate situations [Oldsen et al. 1997]. In these cases, the cable trusses also successfully controlled the roof. No cables trusses failed while the ridged trusses had. However, if failure progresses a sufficient depth into the roof, the dead weight load of the rock could exceed truss capacity. This occurred at a mine in western Kentucky where cable trusses were installed on 1.2-m (4-ft) centers [Miller 1996]. The roof failed to a rider seam when the distance to the rider was under 3 m (10 ft) and resulted in truss failure when the weight of the rock exceeded truss capacity.

Tailgate Support

Rigid trusses have been successfully tested as the only secondary support in a tailgate at a test area established in a mine in southwest Pennsylvania [Stankus et al. 1994]. In the test, a section of tailgate entry 112.7 kN (370 ft) long was supported by trusses on 1.2-m (4-ft) centers. Loads on the horizontal members increased by 44.5 kN (10,000 lbf) in the abutment zone. Behind the shields, the roof did stay up for a distance of 7.3 to 9.1 m (24 to 30 ft).

Cable trusses in combination with cable bolts have also been tested as the main secondary support in a section of tailgate at another mine in southwestern Pennsylvania [Molinda et al. 1997]. In this case, the cable trusses were installed on 2.4-m (8-ft) centers and supplemented with one row of 3.7-m- (12- ft) long cable bolts placed along the pillar side of the entry on 1.8-m (6-ft) centers. The tailgate outby the face and along the shields stayed open with only minor damage to the roof being noted. The maximum roof separation measured was just under 2.5 cm (1 in). Behind the shields, the trusses failed almost immediately because of the cave and only about 25% of the entry remained open alongside the pillar for ventilation. This small section was kept open by the cable bolts. Even this small airway appears to have been closed off about three-fourths of the way to the crosscut behind the face.

Design of Cable Truss Systems

Because cable trusses have a low vertical stiffness and are very flexible, they can deform to the shape of the roof. This, in combination with the high strength of the cable, makes the truss an excellent support where especially large lateral deformation occurs. From the laboratory tests, measured vertical truss stiffness of 17.5 kN/cm (5 tons/in) is significantly lower than the stiffness of a cable bolt or a cable bolt system, where up to four cable bolts would be used in place of the cable truss.

The loading of a truss is complex; however, based on laboratory work, the total amount of vertical load or dead weight the cable truss can sustain appears to be about 120% of the ultimate strength of the cable or about 311 kN (70,000 lbf) for a 1.52-cm (0.6-in) in diameter cable with symmetrical loading at the bearing plates. Failure of the cable truss will usually occur between the anchor hole and the bearing plate when cable tension load in the angle section is around 87% of the ultimate strength of the cable (figure 28) [Tadolini et al. 1998]. Furthermore, a cable truss in situ can be subject to asymmetrical loading, resulting in an even lower load capacity. Therefore, to determine the performance of a cable truss in the field, it would be necessary to measure the strains or loads on all three sections of the cable as well as measuring roof sag [Mangelsdorf 1979]. For cable trusses in general, strain measurements are usually determined only on the horizontal member. Thus, a complete picture on the performance of cable trusses in situ has not been obtained.

Generally, trusses are installed between the existing rows of primary roof support, so spacings will be on 1.2-or 1.5-m (4- or 5-ft) centers. However, if the roof failure is deep enough, the dead weight load of material can exceed truss capacity. Based on laboratory tests, for a 1.52- cm-(0.6-in) long cable with an ultimate strength of 260 kN (58,600 lbf) and the truss carrying a load 120% of cable strength, the dead weight load capacity is 313 kN (70,500 lbf). With 5.5-m (18-ft) wide opening, a 1.2-m (4-ft) truss spacing, and a rock density of 2,307 kg/m^3 (144 lbf/ft^3), a failure depth of about 2.1 m (7 ft) would exceed this capacity. In such cases, either tighter truss spacing, higher capacity trusses, or additional supplemental support must be used in conjunction with the truss system.

For the cable trusses, anchorage requirements are the same as for a cable bolt where a minimum of 1.2 to 1.5 m (4 to 5 ft) of anchorage length should be used. The trusses are anchored in angle holes that are usually drilled at a 45° angle over the coal rib. Other angles can be used, but this will affect loading and load distribution in the truss. These angle holes allow the truss anchorage to be outside the potential failure zone. Once the anchorage is undercut so that when the trusses are behind the face, this is no longer the situation and the truss fails because of the loss of the anchor. Similar conditions may also develop in intersections or the shield zone where the angle member is not anchored above a coal rib, but in a potential failure zone.

Bearing plates are used and installed up to around 15 to 30 cm (6 to 12 in) from the anchor hole. These bearing plates allow two points of contact on the roof, lessens the cable bend, and allows for more efficient load transfer along the cable.

Cable trusses have been used successfully as supplemental support in the headgate to control damage from high horizontal stresses that can develop near the longwall face. The strength, low stiffness, and flexibility of the cable trusses are important characteristics that allow the support to survive and maintain control of a damaged and highly deformed roof. Other types of support, especially rigid trusses, have failed under conditions where large lateral movements occur. As the main secondary support in the tailgate, cable trusses have been relatively successful in a few test cases. However, the trusses do have trouble maintaining the tailgate open behind the shields because of loss of anchorage. Also, there are no data to indicate whether there is any loss of anchorage and therefore support as the cables are undermined between the face and the back of the shield. When mining through an intersection, either side of the truss could fail as a result of loss of anchorage because the truss is anchored in a potential failure zone.

Figure 26.–Cable trusses installed in the headgate entry.

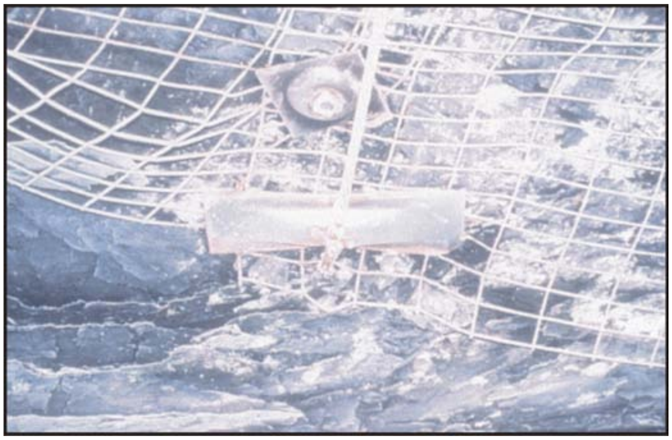

Figure 27.–Roof damage from horizontal stress around cable truss in headgate just outby the face.

Figure 28.–Shear failure of cable truss near bearing plate.

SUMMARY AND CONCLUSIONS

Cable technology as used in the longwall gate roads in U.S. coal mines was developed in the 1990's. This technology includes cable bolts and cable trusses. Cable bolts consist of a headed cable utilizing a partial grout column anchor formed from a resin cartridge and installed with a roof bolting machine.

When evaluating cable bolts, there are several characteristics and components of the cable system that are important to bolt performance. This includes cable strength, elongation, stiffness, and shear resistance, and system anchor capacity. In general, the cable should be the weakest part of the system. Therefore, to exceed the ultimate capacity of the cable, the anchorage length should be a minimum of 1.2 to 1.5 m (4 to 5 ft) long. The stiffness of the cable system, the ability to resist loading, is determined by the free length and elastic properties of the cable. As determined from in situ pull tests, the elastic modulus of the cable was found to be about 131.7 GPa (19.1 million lbf/in^2). This value can be used to calculate cable bolt stiffness. For improved long-term performance, galvanized wire strands or epoxy-coated cable should be used to resist cable corrosion and limit the potential for any strength reduction of the cable. Furthermore, high- capacity bearing plates and heavy-duty mats or channel provide added protection with surface control and an element of structural support for the immediate roof.

Design of cable bolt systems as secondary support in tailgate entries is based to a large extent on suspension, although this is somewhat of an oversimplification of the conditions that can develop. Cable lengths are determined by the depth in the roof of a potential failure horizon over the entry plus an adequate anchorage length. The number or density of cables will then be determined by the dead weight load of rock below that failure horizon. Lateral roof movement may also cause significant loads to develop in the cables where the cables resist the movement and increase the residual shear strength of the rock However, in some cases, the cables may not be able to stop or limit lateral movement and, as a result, can fail. It has been estimated that the cables can handle up to 5 to 10 cm (2 to 4 in) of lateral movement. There are measures that can be taken to reduce the potential for cable failure, including locating the bolts outside the highest lateral deformation zones or using yielding bolt heads.

For supporting tailgates, there are three zones that must be considered when evaluating the design and performance of the cable system. These zones include the outby abutment zone for both vertical and horizontal stress, the shield zone, and the cave zone. In situ tests have been used to further define and confirm cable bolt designs and performance in each of these zones. In these test cases, the number of cables used per row was four with 1.2- to 1.8-m (4-to 6-ft) row spacings. The cable lengths at the site varied from 3.7 to 4.9 m (12 to 16 ft). These cable bolt systems were very successful in supporting longwall tailgate entries with few resulting ground control problems.

From a ground control standpoint, cable bolts have an advantage over standing support where they do not resist main roof-to-floor convergence. This is especially important with a yield pillar system because much of the capacity of the standing support will be taken up by this convergence. Although cable bolts have maintained the tailgate entry behind the shield for long distances, the cave and roof geology are the main factors that determine this distance and not the cable system design. Therefore, cable bolts cannot guarantee that the tailgate can be kept open to the first crosscut behind the face for ventilation.

Cable trusses were greatly improved in conjunction with cable bolt development, and as a result, are now used more extensively than previously, especially as supplemental support in headgate entries. In the headgate, the cable truss has been used to control damage caused by horizontal stress. High strength and flexibility and low stiffness are reasons why trusses can survive in a high-stress and high-deformation environment and still function to maintain a highly deformed roof.

Furthermore, the cable truss anchorage is outside the potential roof failure zone. Therefore, cable trusses have been successful in providing supplemental support in critical headgate entries.

However, based on laboratory tests, the capacity of trusses to carry dead weight loads appears to be only about 120% of the ultimate strength of the cable. More tests, including in situ studies, are required to determine if this capacity could be used for design or must be modified.

To evaluate the performance and capacity of a truss in situ though requires monitoring loads on all three cable legs along with roof sag. Although trusses have been instrumented, to date this has not been done to the level required for a complete evaluation of their performance. As secondary support in the tailgate entry, cable trusses have been tested or used only on a limited basis. Behind the shields, trusses can only keep the tailgate open for a very limited distance. Beyond such distances, there are questions on how well heavily loaded trusses will perform when the anchors are undercut by mining or in intersections where anchors are not supported by a coal rib.

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