DESIGN OF ROOF BOLT SYSTEMS

By Christopher Mark, Ph.D.[1]

ABSTRACT

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

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INTRODUCTION

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

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

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

REINFORCEMENT MECHANISMS OF ROOF BOLTS

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

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

- *Simple Skin Control:* Strong, massive roof subjected to low stress levels can be essentially "self-supporting," meaning that a major roof collapse is unlikely to occur. However, cracks, joints, crossbeds, or slickensides can create occasional hazardous loose rock at the skin of the opening (figure 1*A*). Pattern bolting is therefore required to prevent local loose rock from falling, but the bolts may be relatively short and light. Skin control is also an important secondary function of roof bolts with the other three support mechanisms.
- *Suspension:* In many mines, a thin layer of weak, immediate roof can be suspended from an overlying thick, strong unit that is largely "self-supporting" (figure 1*B*). Experience has shown that roof bolts are extremely efficient in the suspension mode [Conway 1948; Damberger et al. 1980; Mark et al. 1994b], although suspension becomes more difficult if the weak layer is more than 1 m (3 ft) thick. The Coal Mine Roof Rating (CMRR) somewhat quantifies this effect through the Strong Bed Adjustment [Molinda and Mark 1994].
- *Beam Building:* Where no "self-supporting" bed is within reach, the bolts must tie the roof together to create a "beam" (figure 1*C*). The bolts reinforce the rock by maintaining friction on bedding planes, keying together blocks of fractured rock, and controlling the dilation of failed roof layers [Peng 1998; Gale et al. 1992]. In general, it is much more difficult for roof bolts to build a beam than it is to suspend weak rock from one.
- *Supplemental Support Required:* Where the roof is extremely weak or the stress extremely high, roof bolts alone may not be sufficient to prevent roof failure from progressing beyond a reasonable anchorage horizon (figure 1*D*). In these cases, cable bolts, cable trusses, or standing support may be necessary to carry the dead-weight load of the broken roof, and the roof bolts act primarily to prevent unraveling of the immediate roof [Scott 1992].

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

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

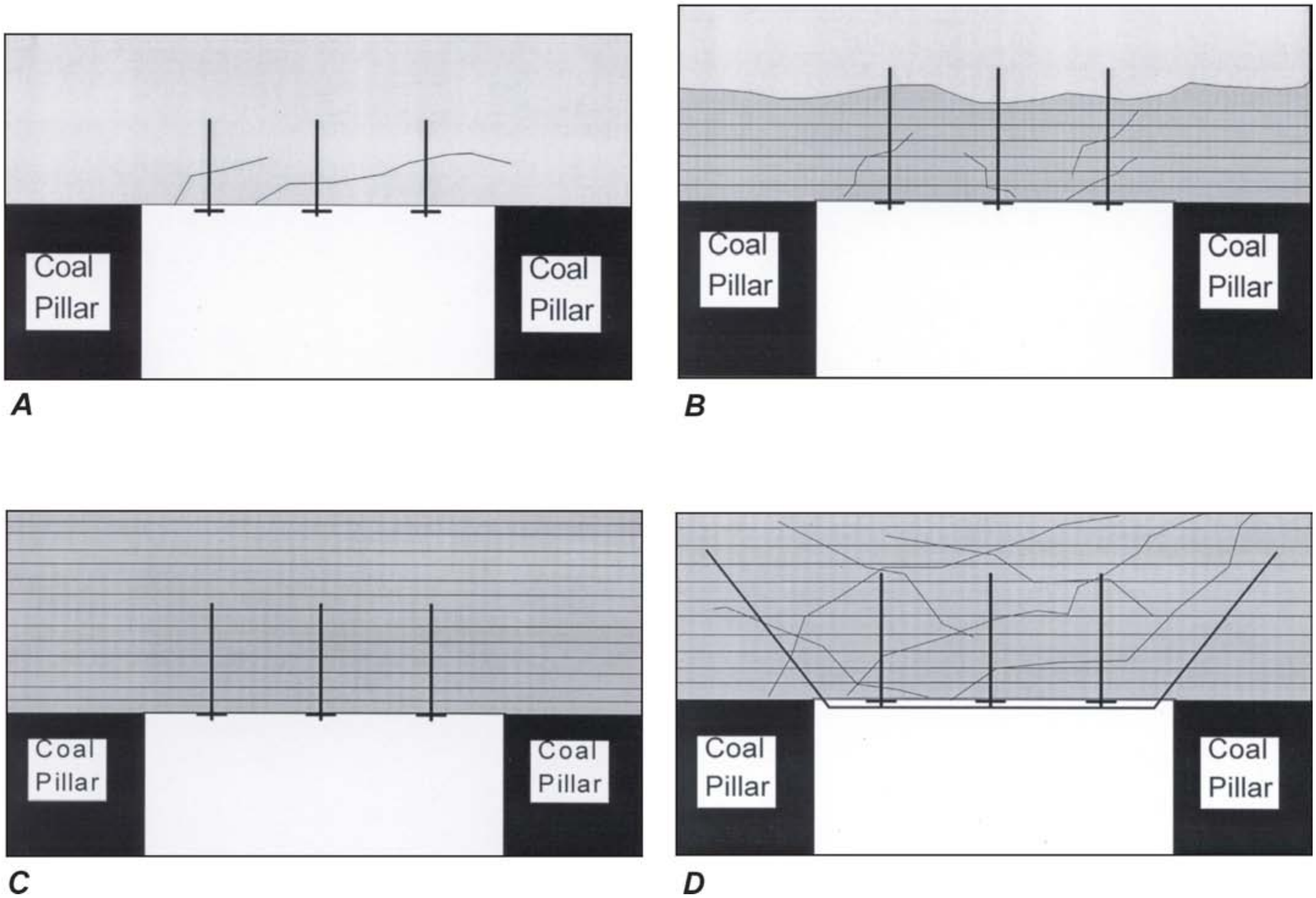

Figure 1.—Roof support mechanisms. *A*, simple skin support; *B,* suspension; *C*, beam building; *D*, supplemental support in failing ground.

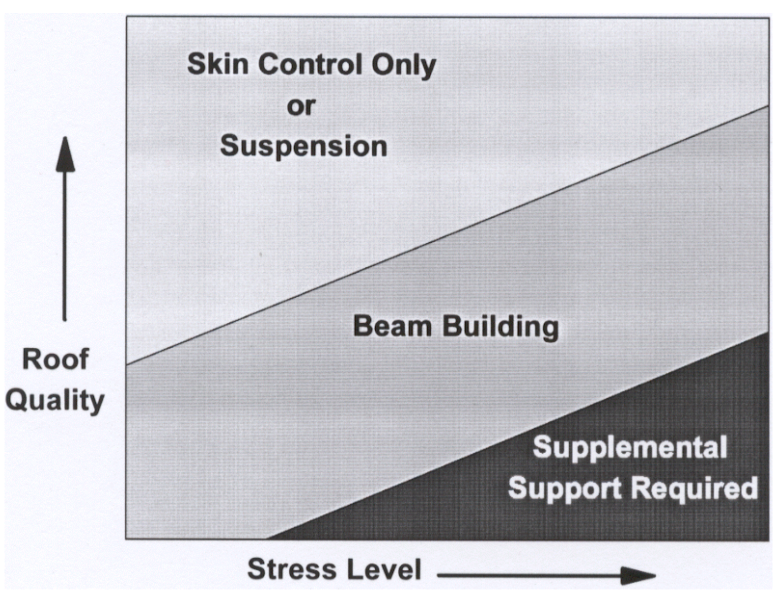

Figure 2.—Roof support mechanisms determined by stress level and roof quality.

CHARACTERISTICS OF ROOF BOLTS

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

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

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

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

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

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

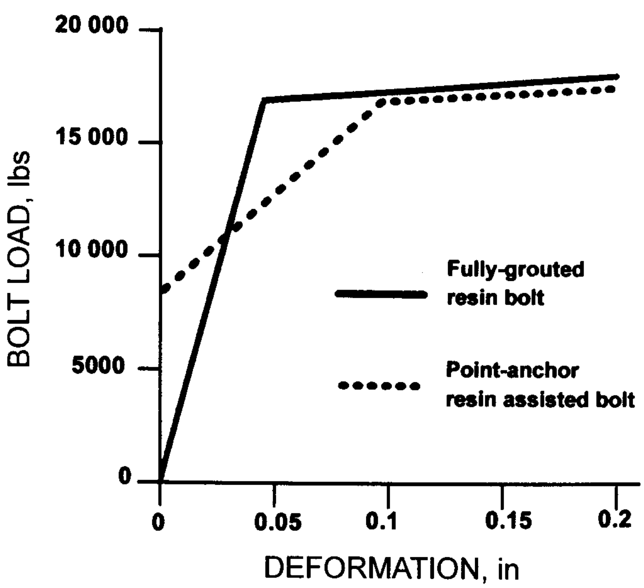

Figure 3.—Stiffness of fully grouted and resin-assisted point-anchor bolts compared (using data from Karabin and Hoch [1980]).

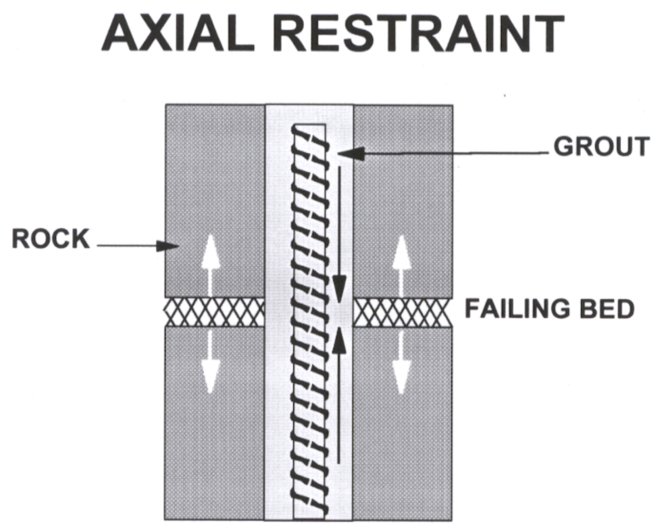

Figure 4.—Tension in a fully grouted bolt caused by dilation of a failed roof bed.

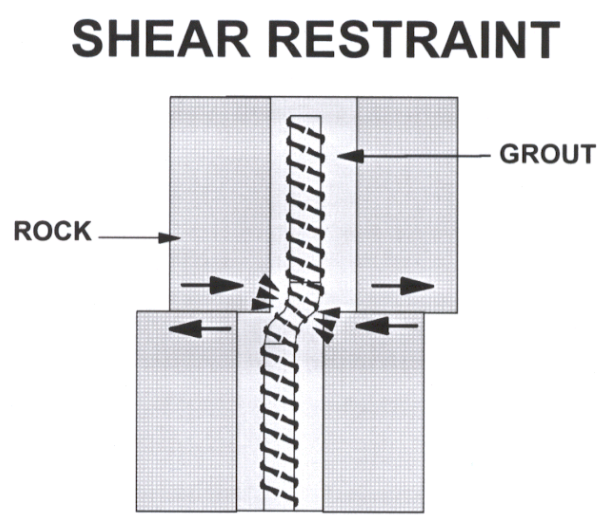

Figure 5.—Tension and bending in a fully grouted bolt caused by slip on a bedding plane.

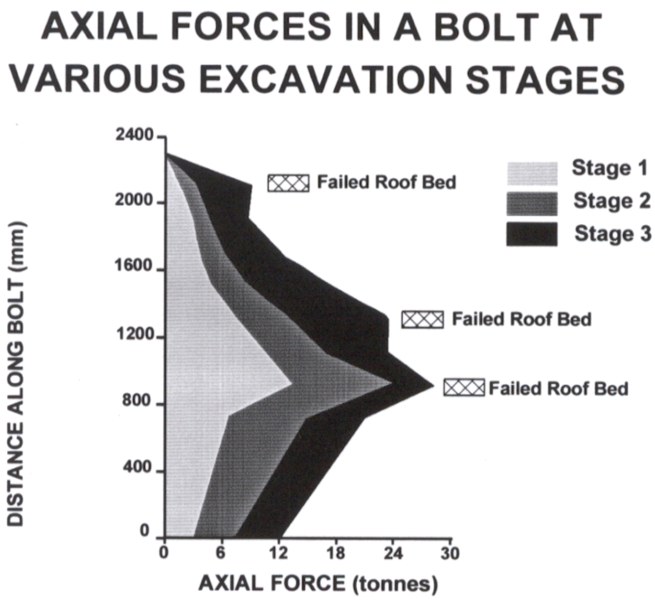

Figure 6.—Typical load distributions measured in a fully grouted bolt at three time during its service life (after Gale [1991]).

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

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

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

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

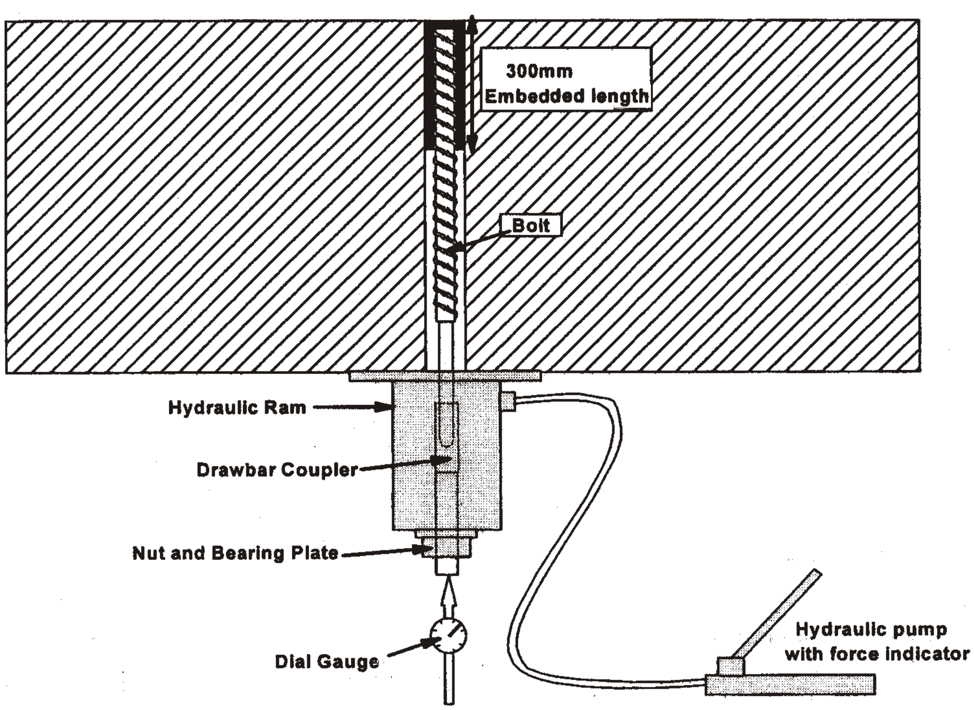

Figure 7.—A short encapsulation pull test.

Table 1.—Anchorage factors for fully grouted resin bolts

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

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

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

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

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

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

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

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

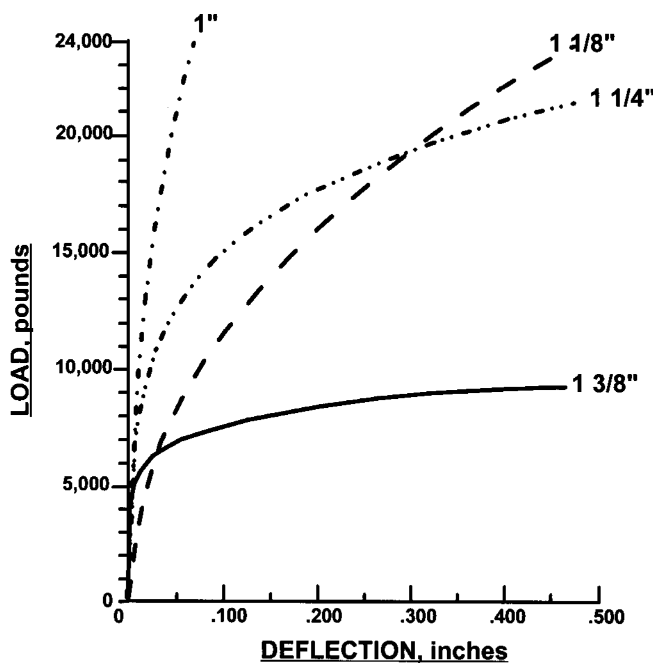

Figure 8.—Effect of hole annulus on grouted bolt performance. Results were obtained from short encapsulation pull tests on 19-mm (0.75-in) diameter rods (after Karabin and Debevic [1976]).

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

INSTALLED TENSION

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

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

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

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

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

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

BOLT CAPACITY

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

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

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

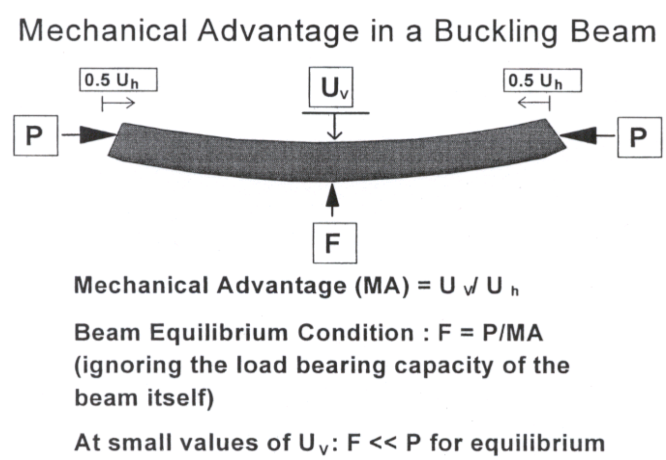

Figure 9.—The Euler buckling beam concept (after Frith [1988]).

Table 2.—Markings on the heads of roof bolts

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

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

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

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

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

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

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

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

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

BOLT LENGTH

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

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

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

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

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

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

where B_L = bolt length;

S = span; and

RMR = rock mass rating [Bieniawski 1987].

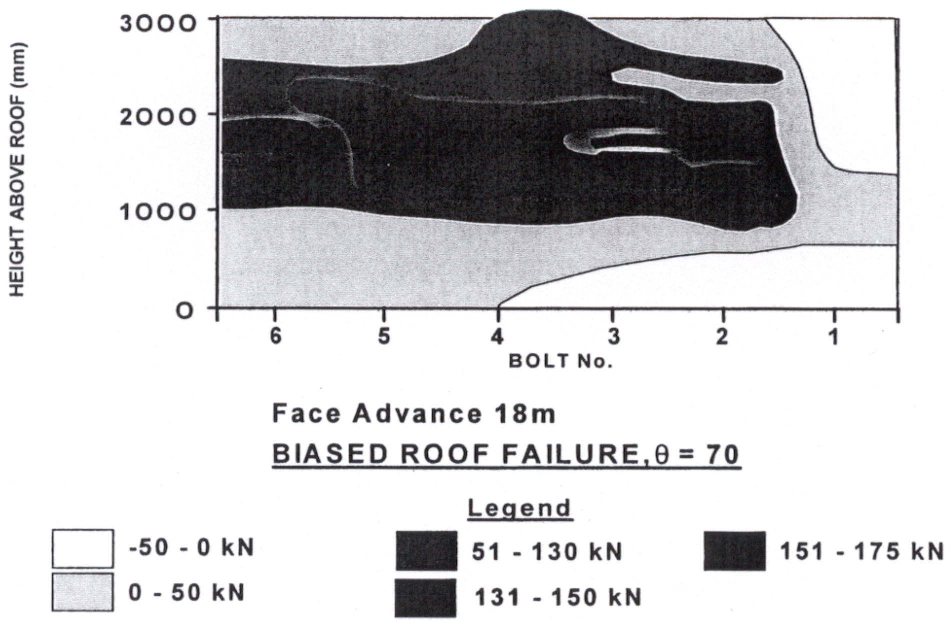

Figure 10.—Nonuniform bolt loading measured in an entry developed at an angle to the maximum horizontal stress [Gale 1991].

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

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

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

ROOF BOLT PATTERN

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

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

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

TIMING OF BOLT INSTALLATION

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

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

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

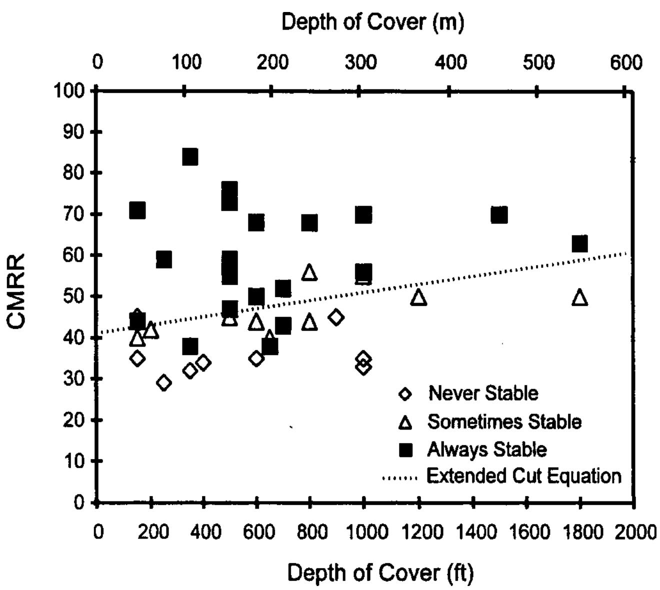

Figure 11.—Effect of the depth of cover on the stability of unsupported roof.

SKIN SUPPORT

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

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

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

INSTALLATION QUALITY

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

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

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

With fully grouted bolts, potential installation problems include—

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

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

ROOF BOLT FAILURE MECHANICS

Roof bolts can fail in one of several ways:

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

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

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

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

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

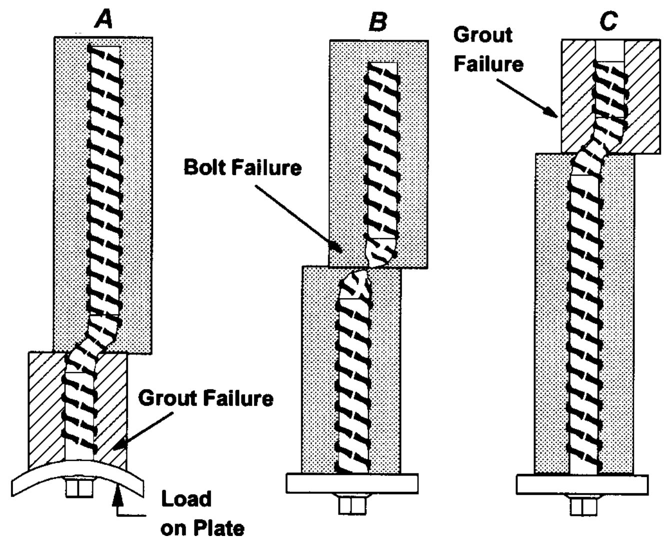

Figure 12.—Failure mechanisms of a fully grouted bolt [after Serbousek and Signer 1987]. *A*, roof movement near head; *B*, roof movement in central portion; *C*, roof movement in anchorage zone.

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

APPROACHES TO THE DESIGN OF ROOF BOLT SYSTEMS

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

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

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

where P = required bolt capacity;

U = unit weight of the rock;

t = thickness of suspended rock;

n = number of bolts per row;

W_e = entry width;

R = row spacing; and

SF = safety factor.

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

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

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

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

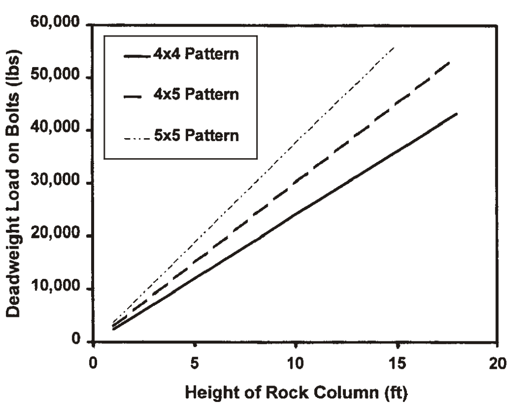

Figure 13.—Dead-weight loads on roof bolts.

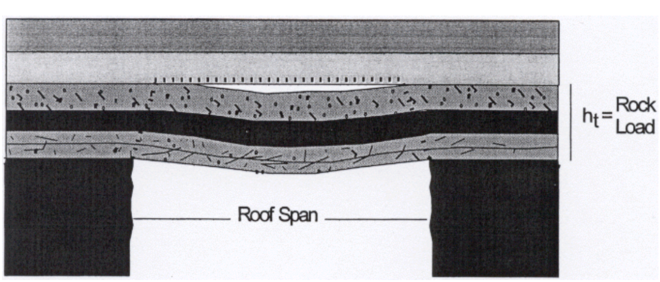

Figure 14.—The rock load height concept (after Unal [1984]).

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

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

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

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

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

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

where K = 1 for massive strata; K = 2 for cohesive, medium bedded strata; and K = 3 for finely laminated, noncohesive strata (figure 15).

In Maleki's approach, tensioned, fully grouted bolts are recommended for the most difficult conditions.

Design by Measurement (pattern/length): This design approach was developed in Australia [Gale 1991; Gale and Fabjanczyk 1993] and was largely adopted by the U.K. Code of Practice [Bigby 1997]. The basic concept is that as individual roof beds become overstressed and fail, they force stresses higher into the roof, which can in turn fail more beds (figure 16). Reinforcement aims to mobilize the frictional strength of failed roof beds in order to restrict the height and severity of failure in the roof. It involves measuring the loads developed in roof bolts during mining, together with a definition of the height and severity of roof deformation obtained from multipoint extensometers. Based on the measurements, optimization of the bolting design might include—

- *Adjusting the bolt length* so that adequate anchorage is achieved above the highest level in the roof where failure is occurring;
- *Adjusting the bolt density and placement* to maximize reinforcement where the roof needs it most;
- *Improving load transfer* by reducing hole size, optimizing bit type, or flushing the hole.

The results are considered valid for environments that are similar to the one studied. Significant changes in the geology or stress field requires additional monitoring.

Optimum Beaming Effect (tension/length): Stankus and Peng [1996] proposed the Optimum Beaming Effect, which is defined as the roof beam that has no separation within or above the bolted range and uses the shortest bolt possible. Its basic tenet is that high installed tensions can be substituted for bolt length. They also argue that longer bolts elongate more in response to load, therefore allowing more roof deformation. The method has been implemented in a finite-element model (see section on "Numerical Modeling" below). Unfortunately, there does not seem to be sufficient justification for this theory. Molinda et al. [2000] found that shorter, tensioned bolts had higher roof fall rates than longer, nontensioned ones in three of four cases where both bolts were used in the same mine.

Structural Engineering Model (tension): In Australia, Frith [1998] proposed a model that divides mine roof into two classes:

- *Static roof* that is essentially self-supporting and requires minimum reinforcement; and
- *Buckling roof* that is thinly bedded and tends to fail layer-by-layer due to horizontal stress.

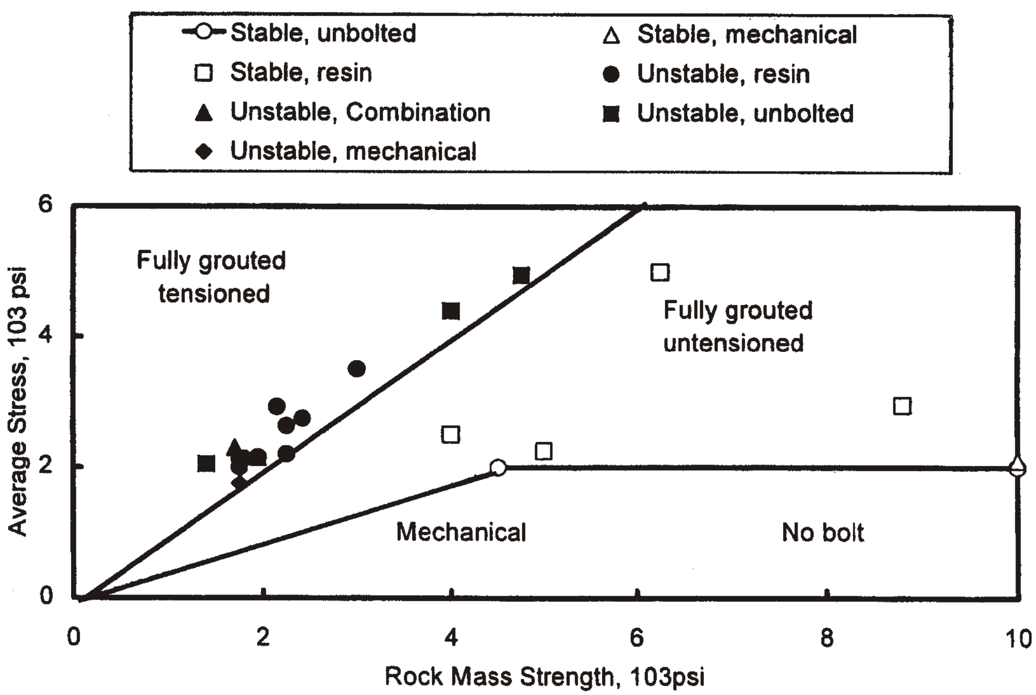

Figure 15.—Maleki's [1992] roof bolt selection chart.

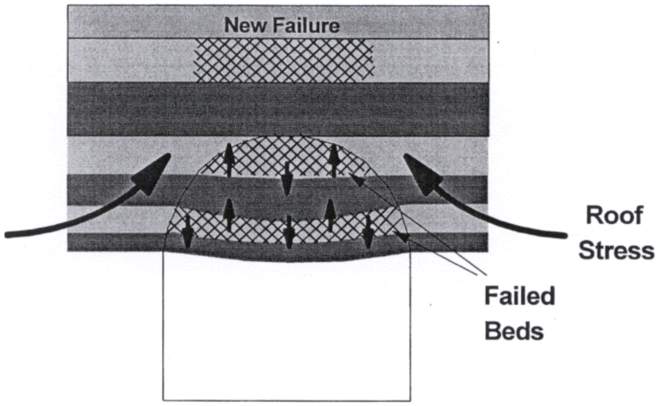

Figure 16.—Failure sequence in highly stressed roof (after Gale [1991]).

Frith proposes that the behavior of the second type of roof can be explained by the basic structural engineering concept of the Euler buckling beam (see previous section on "Installed Tension"). There have been a number of trials of high-tension fully grouted bolts in Australia, and the results are reported to be positive. Unfortunately, the field evidence that has been presented to date has been largely anecdotal (see, for example, Rataj et al. [1997]).

Numerical Modeling: As computers and software have grown more powerful, numerical modeling has become the standard design tool in many branches of engineering. Rock mechanics, however, has lagged behind. The reason is that rock engineers cannot specify the properties of the materials that they use, nor can they usually define the their loading conditions adequately.

For effective, quantitative design using numerical models, three basic prerequisites must be met [Hayes and Altounyan 1995; Gale and Fabjanczyk 1993]:

- *Model:* The model must be capable of replicating the behavior of coal measure rock, which means it must be able to simulate the various failure modes and large deformations which typically occur.
- *Material Properties and Stress:* Input rock mass properties must reflect both pre- and postfailure mechanics of the different roof layers encountered, and in situ stress levels must be measured in the field.
- *Validation:* To ensure that the model and the ground are behaving the same way, stresses and displacements must be measured. Important parameters include the magnitude and location of deformations, the distribution of bolt loads, and the behavior of interfaces at the top of the pillar and within the roof.

Numerical models used in the United States seldom meet any of these requirements. Stankus and Guo [1997] and Guo and Stankus [1997] describe a finite-element model that uses gap elements every 300 mm (1 ft), but otherwise assumes the rock to be homogeneous, elastic, and isotropic. The model looks for zero separation within or above the bolted range, which the study's authors cite as a weakness because bedding separations are commonly observed underground even where the roof is adequately supported [Stankus and Guo 1997]. The model's results are also extremely sensitive to the frictional strength coefficient [Guo and Stankus 1997]. The movements predicted by the model also seem quite small. In one instance cited by Stankus and Peng [1996], the total modeled roof deflection was <1 mm (0.032 in), and the longest bolt resulted in just 6% more deformation than the shortest.

Rigorous models that seem to meet all of the necessary requirements for quantitative design have been described overseas [Bigby 1997; Gale and Tarrant 1997]. Such models implement as many as seven rock failure modes, including bedding slip, shear failure of intact rock, tensile failure, and buckling. However, the expenses associated with such elaborate models, including the associated rock testing, stress measurement, and monitoring, are probably beyond customary U.S. practice. Moreover, the rapid changes in geology that often occur underground raise the question of the number of models and verification sites that might be needed.

Fortunately, numerical models can be very valuable tools even if there is not enough information to use them for quantitative design. As Starfield and Cundall [1988] pointed out, models can be used as controlled experiments to investigate the qualitative effects of different parameters. Well-designed model studies could be very helpful in moving the science of roof bolting forward.

ROOF MONITORING

Regardless of roof bolt design, failures are always possible. Often, an unstable area can be controlled with secondary support if the problem is detected in time. In the United States, instability is usually detected from visible and audible signals that become apparent shortly before collapse. Instruments are far more sensitive and can detect ground movements much earlier.

Routine monitoring of roof movements is much more common abroad. In the United Kingdom and Canada, two-point extensometers (often known as "telltales") are required every 20 m (65 ft) in bolted roadways and in all intersections (figure 17). The telltales have two movement indicators, one that shows displacement within the bolted height, and the other that shows movement above the bolts. Telltales are visible to everyone using the roadway, and their information can be recorded for later analysis [Altounyan et al. 1997].

The key to the effective use of monitoring is the determination of appropriate "action levels." For example, in gate roads at the Phalen Mine in Nova Scotia, Canada [McDonald and McPherson 1994]:

• *Spot bolting* when 25 mm (1 in) of movement is recorded either within or above the bolts.

• *Additional bolting and center props* when 50 mm (2 in) of movement is recorded.

• *Cable bolts* when 75 mm (3 in) of displacement is observed.

In the United Kingdom, typical action levels are 25 mm (1 in) within the bolted horizon and 10-25 mm (0.4-1 in) above [Kent et al. 1999a]. A survey of action levels in Australian mines, however, found no such uniformity. Some mines used total movement criteria; others used rates of movement ranging from 1 to 10 mm (0.04 to 0.4 in) per week [Mark 1998]. In the United States, the data are scarce, but action levels or "critical sag rates" have usually been about 5 mm (0.2 in) per week [Mark et al. 1994c].

In the United States, the lack of available personnel to install, read, and interpret roof monitors has always hindered their widespread use. However, preventing even a single roof fall in a critical belt or travel entry could justify the expense of a fairly extensive monitoring program. Hopefully, the time is not far away when computerized systems will help mines to make better use of roof monitors.

Often, roof monitoring can uncover a hidden geologic factor that can then be used directly in design. For example, a back analysis of monitoring data from the Selby coalfields in the United Kingdom found that excessive roof movements occurred where entries were unfavorably oriented relative to the horizontal stress or where the mudstone thickness exceeded 2.5 m (8 ft) [Kent et al. 1999b]. At the Plateau Mine in Utah, Maleki et al. [1987] found that excessive sag rates correlated with the presence of a channel sandstone within 1.5-2.2 m (5-7 ft) of the coal. A program of test holes helped locate the sandstone and reduced the number of sagmeters needed.

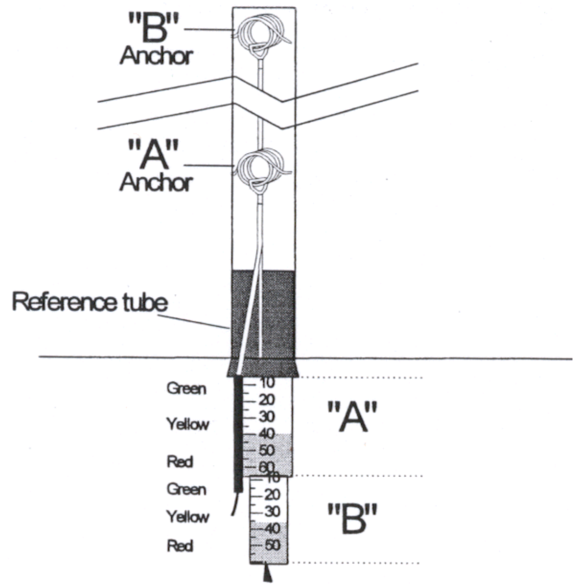

Figure 17.—A telltale (after Altounyan et al. [1997]).

GUIDELINES FOR ROOF BOLT DESIGN

Currently, there are no reliable methods for designing roof bolt systems. To begin to fill the mining community's need for better guidelines, NIOSH conducted a study of roof fall frequencies at 37 coal mines. The study's methodology, data collection procedures, and statistical analyses are reported by Molinda et al. [2000].

The study found that there was considerable scatter in the results, so that it was not possible to develop a universal design equation. In particular, it was not possible to determine the relative importance of individual rock bolt parameters including tension, length, capacity, and pattern.

However, some valuable relationships were found. It was not surprising that the geology, represented by the CMRR, was the most important variable. However, the next most important parameter was the depth of cover. With all else equal, deeper mines were more likely to have high roof fall rates. Horizontal stress could not be measured directly, but since it is known that the intensity of horizontal stress tends to increase with depth, the inference is that the depth of cover is a surrogate for the stress level. When the data were separated into a shallow cover group (<125 m (400 ft) and a deeper cover group (>125 m (400 ft), bolt design equations were determined for each.

Following are step-by-step guidelines:

1. *Evaluate the geology.* The CMRR should be determined either through underground observation or from exploratory drill core. Zones of markedly different CMRR should be delineated. If the thickness of individual beds varies within the bolted horizon, this effect should be noted. Special features, such as faults or major geologic transition zones, should be treated separately.

2. *Evaluate the stress level.* It is unusual for stress measurements to be available, so the design procedures use the depth of cover as a rough estimator. However, horizontal stress can sometimes be intensified by stream valleys or by driving in an unfavorable orientation. Roof support may need to be increased in these areas.

3. *Evaluate mining-induced stress.* Vertical, and sometimes horizontal, stresses may also be intensified by retreat mining or multiple seam interactions. These areas are likely to require supplemental support.

4. *Determine the intersection span.* An equation was derived from the data which suggests that the appropriate diagonal intersection span (I_s) is approximately:

$$(I_s) = 9.5 + (0.2 * \text{CMRR}) \text{ (meters)} \tag{8a}$$

$$(I_s) = 31 + (0.66 * \text{CMRR}) \text{ (feet)} \tag{8b}$$

If the CMRR > 65, it should be set equal to 65 in equation 8.

The intersection span can also be estimated from the entry width using table 4 where the typical spans are based on the field data:

Table 4.—Diagonal intersection spans (I_s)

Entry width, m (ft)	Ideal span, m (ft)	Typical diagonal intersection spans	
		Shallow cover, m (ft)	Deep cover, m (ft)
4.9 (16)	7.0 (23)	8.9 (29)	9.5 (31)
5.5 (18)	7.8 (25)	9.5 (31)	10.1 (33)
6.2 (20)	8.7 (28)	9.8 (32)	10.5 (34)

NOTE: The "ideal span" is determined by applying the Pythagorean theorem ($a^2 + b^2 = c^2$). "Typical" spans are based on actual measurements [Molinda et al. 2000].

As table 4 shows, the field data indicated that for the same entry width, spans at deep cover (depth > 130 m (400 ft)) exceeded the shallow cover spans by an average of 0.6 m (2 ft) due to pillar sloughing.

5. *Determine the bolt length.* Where the roof geology is such that the suspension mode is appropriate, the bolt length should be selected to give adequate anchorage in the strong rock. For the beam building mode, a bolt length formula was derived by modifying the Unal [1984] rock load height equation. The intersection span was substituted for the entry width, a depth factor was added, and then the constant was adjusted to fit the data:

$$L_B = 0.12\left(I_s\right)\log_{10}(3.25\,H)\left[\frac{100 - \text{CMRR}}{100}\right] \text{ (meters)} \tag{9a}$$

$$L_B = 0.12\left(I_s\right)\log_{10}(H)\left[\frac{100 - \text{CMRR}}{100}\right] \text{ (feet)} \tag{9b}$$

where (I_s) = diagonal intersection span (meters in equation 9a; feet in equation 9b); and

H = depth of cover (meters in equation 9a; feet in equation 9b).

These equations are illustrated in figure 18.

6. *Determine bolt pattern and capacity:* As has already been stated, the data could not determine which bolt parameter was most important. Therefore, the design variable is PRSUP, which includes both, plus the bolt length:

$$\text{PRSUP} = 29\,\frac{\text{Lb} * \text{Nb} * \text{C}}{\text{Sb} * \text{We}} \tag{10a}$$

$$\text{PRSUP} = \frac{\text{Lb} * \text{Nb} * \text{C}}{\text{Sb} * \text{We}} \tag{10b}$$

where Lb = length of the bolt (meters in equation 10a; feet in equation 10b);

Nb = number of bolts per row;

C = capacity (kilonewtons in equation 10a; kips in equation 10b);

Sb = spacing between rows of bolts (meters in equation 10a; feet in equation 10b); and

We = entry width (meters in equation 10a; feet in equation 10b).

Note that PRSUP differs from the PSUP used in past studies [Mark et al. 1994a] in that the bolt capacity has been substituted for the bolt diameter.

The suggested value of PRSUP for shallow cover is determined as:

$$\text{PRSUP} = 15.5 - 0.23\ \text{CMRR} \tag{11a}$$

and for deeper cover:

$$\text{PRSUP} = 17.8 - 0.23\ \text{CMRR} \tag{11b}$$

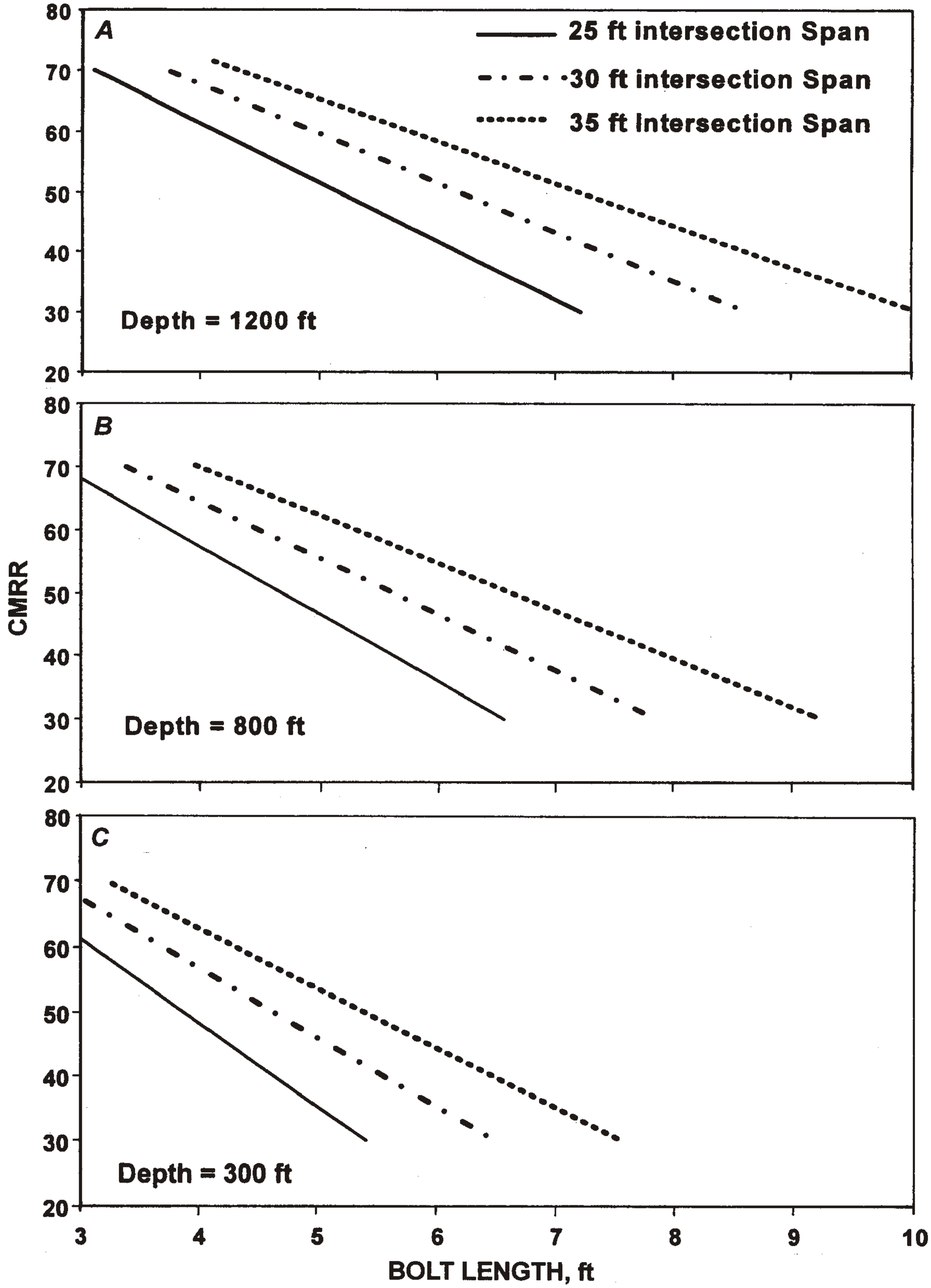

Figure 18.—Formula for selecting the bolt length. *A*, depth = 1,200 ft; *B*, depth = 800 ft; *C*, depth = 300 ft.

Figure 19 shows these equations, together with the field data from which they were derived. The design equations are slightly more conservative than the discriminant equations on which they are based.

The field data also indicated that in very weak roof, it may be difficult to eliminate roof falls using typical U.S. roof bolt patterns. When the CMRR was <40 at shallow cover and <45-50 at deeper cover, high roof fall rates could be encountered, even with high roof bolt densities. Faced with these conditions, special mining plans, such as advance-and-relieve mining (Chase et al. [1999]), might be considered.

It should also be noted that these equations have been derived to reduce the risk of roof falls in intersections. In some circumstances, it may be possible to reduce the level of support between intersections.

Finally, the minimum recommended PRSUP is approximately 3.0.

7. *Select skin support:* Plates, header, mats, or mesh should be specified to ensure that loose rock between the bolts does not pose a hazard.

8. *Monitoring:* The installation of telltales or other simple extensometers should be considered for critical intersections so that, if it becomes necessary, supplemental support can be installed in a timely fashion.

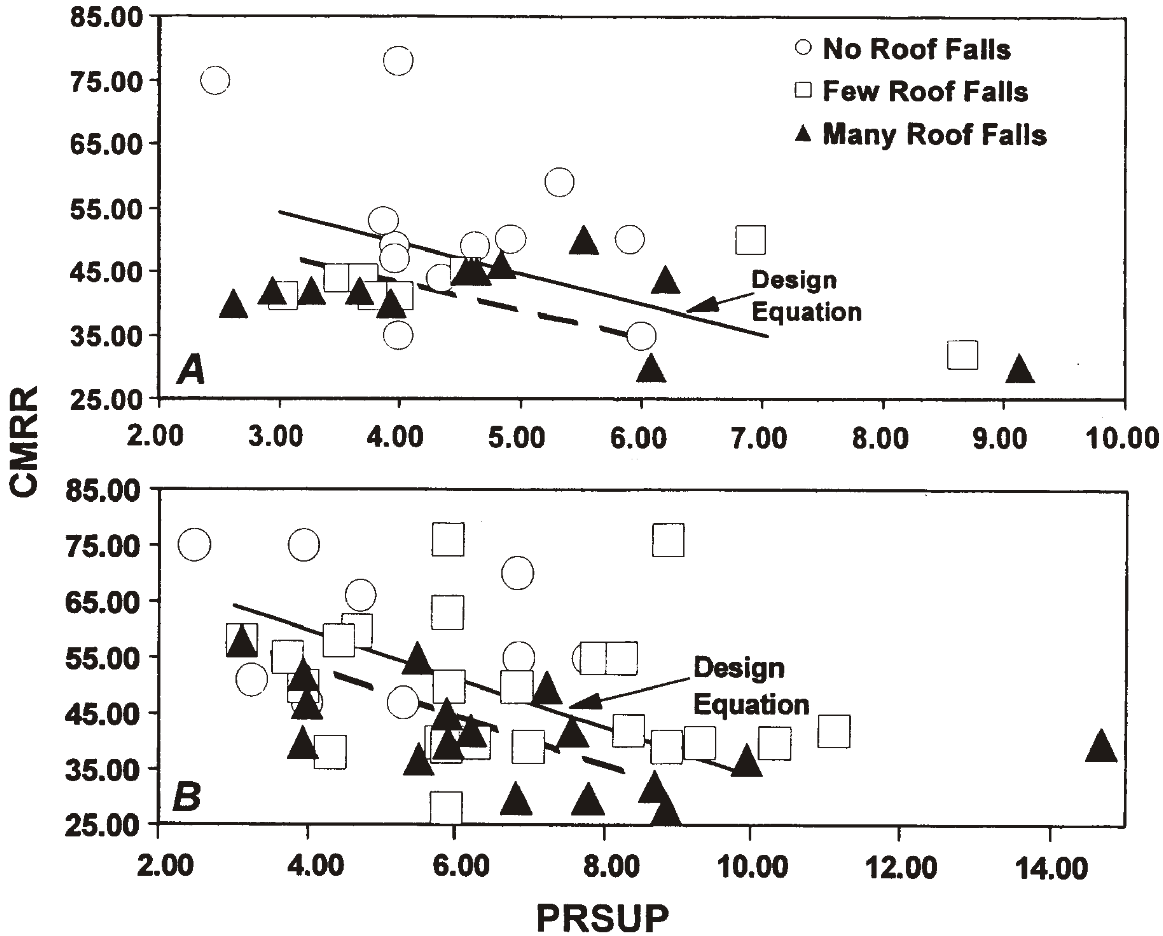

Figure 19.—Design equations for selecting bolt pattern and capacity. The field data used in the derivation of the formulas are shown, along with the original "discriminate equations" (dotted line). *A*, shallow cover (depth < 120 m (400 ft); *B*, deep cover (depth > 120 m (400 ft)).

CONCLUSIONS

1. Four support levels and reinforcement mechanisms are identified for roof bolts: *simple skin control, suspension, beam building*, and *supplemental support required*. The mechanism required for a particular application depends on the geology and the stress level.

2. The performance of fully grouted roof bolts can be determined by the load transfer effectiveness, which is indicated by the anchorage factor. Poor load transfer can reduce the effective capacity of the upper 300-600 mm (1-2 ft) of the bolt. Installations in weak rock are most at risk. Short encapsulation tests can be used to determine if the Anchorage Factor is adequate. Load transfer can be improved by optimizing the hole annulus, rifling or cleaning the hole, or roughening the bolt profile.

3. The importance of installed tension remains a subject of controversy. High tension is probably not necessary for simple skin control or suspension applications, but it may be helpful for beam building.

4. Increasing the bolt length can be effective in reducing the number of roof falls.

5. In weak roof, it is important that roof bolts be installed as soon as possible after the roof is exposed.

6. Effective skin control is an essential function of all roof support systems.

7. Proper installation is critical to the performance of roof bolting systems. Unfortunately, it is difficult to check the installation of fully grouted systems. Training and retraining of roof bolt crews is therefore essential.

8. Roof bolts may fail at the head, in the rod, or at the anchor. In addition, the system may fail if the rock breaks above it or if the support does not provide effective skin control.

9. Field measurements have shown that the loads on roof bolts commonly exceed the dead-weight loads by factors of two or more. Unfortunately, most of the other available empirical design approaches are qualitative at best.

10. Before numerical models can be used for design, they must—

- Be sophisticated enough to replicate complex rock mass behavior;
- Incorporate detailed rock property and in situ stress data; and
- Be validated by extensive field measurements.

Models used in the United States rarely meet these criteria.

11. Roof monitoring, particularly with two-point extensometers, could greatly improve our capacity to optimize the performance of roof bolt systems in the United States. However, such instruments will have to be computerized before they are widely accepted by the mines. A better understanding of the appropriate "action levels" for U.S. conditions will also be needed.

12. Guidelines are suggested for the preliminary design of roof bolt systems, based on analysis of field data collected from 37 U.S. coal mines. Formulas are provided that may be used to select appropriate intersection spans, bolt lengths, and bolt capacity/patterns. The formulas require a determination of the roof quality (using the CMRR) and the stress level (using the depth of cover). The equations should be used with caution, however, because the data used in their derivation were highly scattered.

13. The data also suggest that typical U.S. bolting systems may not always be capable of controlling roof falls in weak rock subjected to high stress.

14. Much more progress is needed before roof bolt design can truly be said to have advanced from an "art" to a "science."

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