A Stability Factor for Supported Mine Entries Based on Numerical Model Analysis

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ABSTRACT

This paper addresses the need for a method to compare the effectiveness of different support systems when designing ground support in coal mines. At present, support design methods include empirical methods based on observations of past performance of installed support systems, analytical methods where the roof is typically simulated by elastic beams and numerical model analysis. The approach presented in this paper estimates the relative stability of a support design through geotechnical evaluation of the rock mass and numerical model analysis of the interaction between the rock mass and the support system. Models are used first to simulate the design performance at the expected rock conditions. The rock strength is then reduced until collapse is indicated in the model. The stability factor is then calculated as the ratio of the expected rock mass strength to the rock mass strength at the onset of collapse, and is similar to the well-known factor of safety used in engineering practice. The stability factor can be used to assist in developing a final support design by comparing the effectiveness of various support systems and the stability of excavations under various geological and loading conditions. Examples of the method's application in three different geological settings are presented.

INTRODUCTION

Large unplanned falls of ground in the active workings of coal mines are reported to the Mine Safety and Health Administration (MSHA) if the fall extends at or above the supported horizon, if ventilation is interrupted, if it results in an injury or it impedes men and material access. More than 1,200 such large falls of ground are reported in United States underground coal mines annually (Mine Safety and Health Administration, 2011). Each fall represents a safety hazard and can be considered to be a failure of the ground support system. The problem of large roof falls is being addressed at the National Institute for Occupational Safety and Health (NIOSH), where research is being conducted to develop improved support design methods to better match support systems to coal mines' prevailing ground conditions.

Although 1,200 large roof falls represent a significant safety hazard, when viewed from the perspective of support system success rate, these roof falls represent the failure of a very small percentage of installed support systems. For example, NIOSH researchers investigated a number of mines with a high incidence of roof falls and found that between 0.2% and 2.0% of the total exposed roof fell in reportable roof falls (Klemetti, 2012). In these mines, the support systems can be said to be 98.0% to 99.8% successful against large roof falls. The success rate is no surprise given that roof support systems have seen continuous improvement, resulting in a tenfold reduction in US ground fall fatality rates since the 1930's (Pappas et al. 2000). However, the potentially fatal consequences of a single large-scale roof fall remains unacceptable, and every effort should be made to eliminate these hazardous events.

This paper presents an approach that NIOSH engineers are using to estimate the stability of supported coal mine excavations against large-scale collapse. The method provides relative results, allowing the designer to compare alternative support scenarios under varying ground conditions. The focus of the method is on stress-driven roof falls that are the result of failure of the surrounding rock material. Falls related to individual geological structures, such as large slips or faults, are not included. Smaller skin falls that occur between supports are also excluded from this study.

Success and Failure Considerations

When designing a supported excavation, it is necessary to consider the strength variability of the surrounding rocks and loading conditions. An adequate margin of safety must be included in the design so that the excavation will remain stable under an unfavorable combination of strength and loading conditions. The classic factor of safety used in engineering design is one way of accounting for the variability. The appropriate value of the safety factor is usually determined through experience or can be determined based on failure probability considerations (Harr, 1987).

The factor of safety is usually calculated by comparing the strength, also known as the capacity, of a structure to the expected load. Unlike many engineering structures, the stability of an underground excavation cannot readily be determined by comparing the "load" to the "capacity" and calculating a classic factor of safety. First, the loads or stresses are redistributed in the relatively boundless rock mass around the excavation; thus, it is not clear which loads should be considered. Also, the system's capacity is not easily defined because the support system is usually integrated within the rock mass being supported, and there is no clear distinction between capacity of the installed supports and the enhanced capacity of the reinforced rock to resist collapse.

The problem is somewhat simplified when the type of failure being evaluated is clearly defined. Mining engineers are interested in the functional success of mine excavations. The excavation must fulfill its function of providing safe access to the working face. The excavation is considered a failure if it no longer fulfills this function. Functional failure should not be confused with failure of the rock material around the opening. In many cases, damaged or failed rock material can exist around an excavation, yet the support system maintains stable conditions and the excavation is functionally successful. In the case of the large reportable roof falls under consideration, "failure" can be defined as the occurrence of a large roof fall that extends up to or above the bolted horizon. Conversely, "success" is defined as the avoidance of a large roof fall.

Since the loading and support capacity of underground mine excavations are not clearly defined, using the term *safety factor* may result in confusion. For example, unsupported excavations can be adequately stable, but the factor of safety would be zero if calculated as the ratio of support capacity to rock load. An alternative approach is presented in this paper where the excavation stability is expressed as a *stability factor*, which attempts to express the degree of safety of an excavation in similar terms as the classic factor of safety.

METHOD FOR CALCULATING THE STABILITY FACTOR

The proposed stability factor is calculated as a ratio of the average rock mass strength (the capacity) to the required rock mass strength to prevent collapse from occurring (the demand). Numerical models are used to simulate the excavation, loading conditions and support system and determine the rock strength that is required to prevent collapse from occurring. This is achieved by using the strength reduction modeling technique (Zienkiewicz, et al., 1975; Lorig and Varona, 2000) in which the modeled rock strength is gradually reduced until collapse is indicated by the inability of the model to reach a state of equilibrium. Laboratory test results, supplemented by rock classification data and field stress measurements, are used to provide inputs for the numerical models.

Numerical Models

Numerical modeling was selected as the most appropriate method for assessing the stability of an excavation for given rock mass, support and stress conditions. Empirical methods and analytical solutions are also available for evaluating the stability of the rock around coal mine excavations (Panek, 1962; Mark, 2000; Canbulat and van der Merwe, 2009; Frith and Colwell, 2010). While these solutions are useful in design decision-making, they seldom capture the complexity of the interaction between a support system and the failing rock mass. Numerical models have the capability of simulating rock failure development, post failure response and ultimate collapse of the surrounding rock while tracking the interaction of the support system with the failing rock mass. The FLAC3D finite difference software (Anon, 2011) was selected to conduct the model analyses.

It is recognized that in most mining situations, data on the rock strength, deformability and loading conditions are limited. In such data limited situations numerical models provide a method of understanding the mechanisms that affect the behavior of a system (Starfield and Cundall, 1988). Ultimately, the model results should only be used as a tool to assist in developing a final design.

Strength Reduction Technique

The strength reduction technique has a long history in numerical model analysis (Zienkiewicz, Humpheson and Lewis, 1975) and has often been applied in rock slope stability engineering to determine the safety factor of rock slopes (Lorig and Varona, 2000), but has not been widely used for underground excavation analyses. The technique is applied by first conducting a stability analysis using average rock strength properties. Depending on the outcome, the analysis is repeated using either a decreased or increased rock mass strength until the point of collapse is satisfactorily bracketed. Strength adjustments are achieved by simultaneously reducing or increasing the cohesion, tensile strength and the coefficient of friction of the rock matrix and the bedding planes. The stability factor is simply calculated as the inverse of the strength adjustment factor at the point of collapse of the modeled excavation. For example, if collapse occurs when the strength is reduced by a factor of 0.8, the stability factor would be 1.25.

The occurrence of collapse is indicated by the inability of the model to reach a state of equilibrium after an extended number of solution cycles. Usually, this means that a roof collapsed at a constant velocity and equilibrium could not be reached. When applying the technique to underground coal mine excavations, it was found that floor heave was sometimes the mode of ultimate instability. This occurred because the floor rocks were weaker than the roof rocks. All analyses were therefore conducted by only reducing the rock mass strength above the roof of the excavation while leaving the coal and floor rocks unadjusted.

NUMERICAL MODEL DEVELOPMENT

The general approach for selecting model inputs and model analysis followed the procedures developed at NIOSH (Zipf, 2006). The modeling approach closely resembles the "synthetic rock mass" concept (Pierce et al., 2007) in which the rock matrix and the discontinuities are modeled separately. This approach is necessary to capture both rock matrix failure and bedding-related slip in the sedimentary coal measure rocks. One significant addition to the approach outlined by Zipf is that strength anisotropy of the rock matrix was included in the models.

The strain-softening ubiquitous joint constitutive model in FLAC3D was used to simulate the bedded coal measure rocks. The models were run in the small-strain mode to alleviate the formation of unrealistic kink-bands in the ubiquitous joint materials. Support units were modeled using the built-in finite element structural components available in FLAC3D. Interfaces were modeled between the different rock units, allowing joint opening and closing to be simulated. The built-in programming language in FLAC3D was used to incorporate the strength anisotropy of the models' bedded rock.

Inputs for the numerical models were obtained from laboratory strength testing, field monitoring and rock mass classification data where available. Unavailable data were based on best estimates from published and other sources.

Anisotropic Rock Strength

The inclusion of anisotropic rock strength in the model analyses resulted in improved agreement with monitored field performance of excavations. The strength of bedded rocks can be highly anisotropic, and this phenomenon has an especially important impact on roof stability when subject to the high horizontal stress found in many coal mines. Field tests on rock core using the point load index apparatus have shown that strength anisotropy can vary from a ratio of approximately 1.0 down to a ratio of less than 0.2 for highly laminated shale (Molinda and Mark, 1996). A review of strength anisotropy data obtained by standard uniaxial compression testing and point load testing seems to indicate that the point load method underestimates the compressive strength in the direction parallel to the bedding. This is not unexpected, since the point load apparatus essentially tests the tensile strength of the rock. However, the point load strength in the direction parallel to the bedding has been found to be an excellent indicator of bedding strength and is used in the determination of the Coal Mine Roof Rating (Molinda, Mark and Barton, 2002). For modeling the compressive strength of bedded rocks parallel to the bedding, the strength reduction indicated by the point load test was halved. For example, if the point load tests indicated that the strength parallel to the bedding was 50% of the perpendicular strength, then the strength was reduced by 25% in the models.

Where data on the point load strength parallel to the bedding were not available, the discontinuity intensity rating (DIR) used in the Coal Mine Roof Rating (CMRR) was used to obtain an initial estimate of the strength anisotropy. The DIR varies from a rating of 9 for closely spaced persistent beds to 35 for widely spaced discontinuous beds. The strength anisotropy ratio was calculated as:

$$R = 0.28\, e^{0.04 \times DIR} \qquad (1)$$

The value of R varies from 1.0 for massive rocks to 0.4 for strongly bedded rocks with a DIR value of 9. This relationship was found to result in realistic roof performance in models simulating high horizontal stress conditions.

The implementation of anisotropic rock strength in the numerical models was achieved by creating a user-defined function that reduced the rock strength to the bedding-parallel strength when the angle between the major principal stress and the bedding surface was less than 15 degrees.

Model Layout

The models presented here were set up to simulate cross-sections through coal mine entries in various geological settings encountered in the United States. More elaborate models of entries and cross-cuts can be created if desired. The model boundaries were located at least three entry widths away from the edges of the entry and extended about 20 meters (65 feet) above and 10m (33 ft) below the entries. The outer boundaries of the models were fixed against normal displacement, while allowing displacement to take place along the boundary surface (roller boundaries). The models were typically 1.2 m thick in the direction perpendicular to the section, which represents the typical slice of rock supported by a single row of bolts. The element (zone) sizes in the vicinity of the entry were between 0.25 and 0.35 m (10 to 14 in). The element width was increased up to about 1.2 m (4 ft) at the excavation's remote locations where rock failure was not expected to occur. In relatively thin geologic beds, the element height was constrained to the thickness of the bed.

Rock bolt supports were modeled across the entry roof, located in the center of the slice. The boundary conditions caused the model to effectively simulate a long entry excavation with repeating rows of supports.

The stress within the models was initialized based on field stress measurements, where available, or using the best published data available for the geological region. When field stress measurements were not available, the horizontal stress was based on the relationships published by Mark and Gadde (2008) in which the horizontal stress is affected by the modulus of elasticity of each rock layer.

Model Calibration

The response of numerical models is highly dependent on the strength and loading parameters used as input. The accuracy of input parameters used in numerical models are affected by the uncertainties associated with projecting laboratory scale rock strength to the field scale, the post failure properties of the broken rock and the natural variability of rock properties and stresses in the field.

A model based on average properties and stress conditions is therefore unlikely to match the rock mass response at a particular experimental location in a mine, and it is necessary to adjust some of the model parameters to better match the specific site response. As more experience is gained converting laboratory tests to field scale properties, the amount of calibration required is likely to decrease. In the case studies presented below, the most common adjustment was the large scale cohesion of the rock mass. In some instances, it was necessary to also adjust the strength of the rock parallel to the bedding to match cutter-type failure initiation. However, all adjustments were maintained within realistic limits.

In each case presented below, the calibrated model response was required to qualitatively match the observed field conditions, such as roof damage, coal rib spalling and bolt loading. Where available, the model response was also required to reasonably match the field measured displacements within the rock mass.

CASE STUDY APPLICATIONS

Three examples are presented where the stability factor approach has been used to evaluate entry stability in wide-ranging geological and stress settings. For each case, the model response is discussed for the observed conditions and then the strength reduction technique is applied to determine the stability factor of the excavation.

Pittsburgh Seam Case Study

An excavation in the Pittsburgh seam monitored in great detail was selected as a modeling case study. The entry was selected for detailed stress and deformation monitoring because it was anticipated the entry would be subject to significant horizontal stresses as a second longwall face approached and passed it (Oyler et al., 2004). The entry was located at a depth of 180 m (600 ft) and was 4.8 m (16 ft) wide by approximately 2.1 m (7 ft) high.

The immediate roof may be subdivided into three units: the lower 2.7 m (9 ft) consisted of weak carbonaceous shale and coal, a slightly stronger claystone sequence from 2.7 to 5.4 m (9 to 18 ft) and a significantly stronger limestone above 5.4 m. The initial horizontal stress was estimated based on published equations for the eastern United States (Mark and Gadde, 2008). The initial vertical stress was 5 MPa (725 psi) and the horizontal stress within the stiffer upper shale unit was 9 MPa (1,300 psi) with the stress in other layers being higher or lower, depending on their elastic modulus. The excavation was supported using 22mm (7/8 in) diameter 2.4 m (8 ft) long grouted combination bolts which had 1.2 m (4 ft) of resin grout and 3.6 m (12 ft) long cable bolts with 1.2 m (4 ft) of resin grout. Initial conditions were good in the entry, but some roof cutters were observed in the vicinity. As the longwall approached, conditions deteriorated but the entry did not collapse during the entire monitoring period; that period ended when the longwall face passed the entry. Stress change monitoring results indicated that the horizontal stress increased by up to 10 MPa (1,450 psi) as the longwall approached.

A model of the entry and installed supports was created using rock strength and stress data from the detailed monitoring experiment. Table 1 summarizes the main rock units and the strength parameters used in the final calibrated model. The model was subject to increasing loads to simulate the effects of the approaching longwall, and displacements within the rock mass were determined and compared to field monitoring results. Figure 1 shows the central detailed part of the model in which the bolt and cable locations can be seen. The figure also displays the extent of rock mass damage around the excavation when the horizontal stress had increased by 7 MPa (1,015 psi).

Figure 2 shows the field measured and model determined displacements in the roof as the stress was increased to simulate the approach of the longwall; the model results closely resemble the monitored results. The detailed shape and magnitude of the displacements depend on selection of strength properties for the roof beds in the model. Small changes can be made to the model to match a specific set of field data. The model also provided satisfactory qualitative results. It indicated stability of the overall roof with minor spalling or cutter formation at the corners during the development stage. It also indicated that the entry would remain open, but with considerable rock damage and support yield at the final loading stage when the longwall face would be at its closest location relative to the entry. The rock damage at this elevated stress conditions formed a dome-shaped zone in the roof, which is typically observed when roof collapses occur.

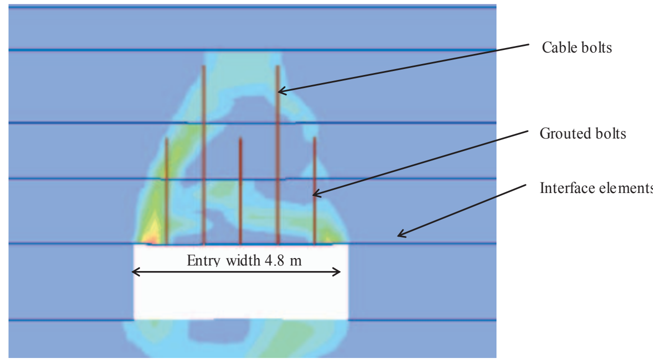

Figure 1. Pittsburgh seam case study showing rock damage contours when the vertical stress had increased to 7 MPa by approaching longwall. Warmer colors indicate increasing damage (plastic strain).

The stability factor of the supported entry was first determined for development conditions. Collapse occurred in the model after the rock strength had been reduced by a factor of 0.34 resulting in a stability factor of 2.94. Considering that the excavation was supported to survive elevated loading conditions induced by the approaching longwall, the relatively high stability factor under development conditions was expected.

Additional scenarios were evaluated where the stability factor was determined for the entry supported by bolts alone, without

Table 1. Key rock properties for modeling the Pittsburgh seam case study.

Rock type	Thick-ness (m)	UCS Lab (MPa)	Elastic modulus (GPa)	Friction angle (deg)	Cohesion (MPa)	Tensile strength (MPa)	Bedding Cohesion (MPa)	Bedding friction (deg)
Shale	18	40	15	28	6.97	2.32	1.98	23.0
Limestone	6	100	35	42	12.91	4.30	11.26	39.1
Gray shale	1	70	20	30	11.72	3.91	3.91	25.0
Gray shale	1.7	40	15	28	6.97	2.32	1.98	23.0
Black shale	1.3	25	10	28	4.36	1.45	0.88	23.00
Shale/coal	1.5	20	8	25	3.70	1.23	0.51	20.00
Coal bed	1.8	16	3	35	2.42	0.81	1.09	30.45
Shale	3	15	15	28	6.97	2.32	1.98	23.00
Shale	16	15	15	28	10.45	3.48	2.97	23.00

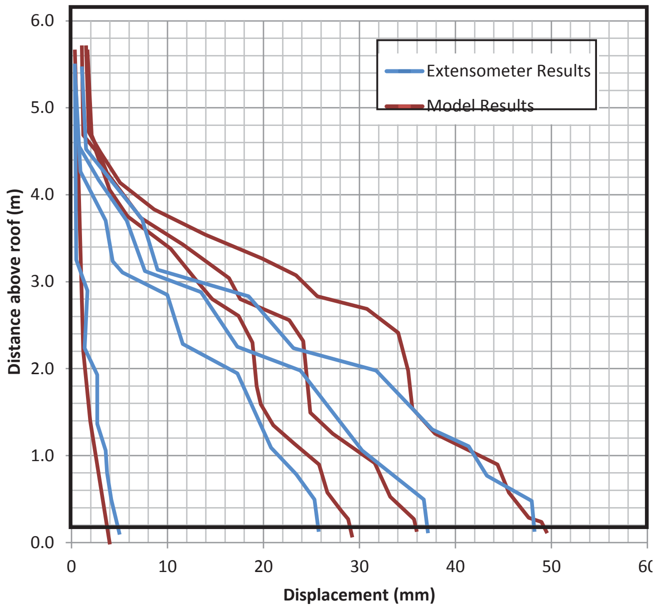

Figure 2. Graph showing comparison of model and field monitoring results of displacements in the roof as loading was increased by an approaching longwall for the Pittsburgh seam case study.

cable bolts. The stability factor reduced to 2.54. A final evaluation was made where the entry was modeled with no support, under initial development conditions. For this case, the stability factor was found to be 1.31. The result was not surprising, as the entries are able to remain stable for one or two shifts with only side bolt support, before the center bolt is installed. A stability factor of 1.31 seems to indicate that temporary stability is possible with limited support.

Thick-Weak Strata Case Study

The second case study is a room and pillar mine in the Illinois Basin where a bolt performance experiment was conducted (Spearing et al., 2011). This case was selected to test the modeling method and the derived stability factor in thick-weak strata. The mine is located in the Danville No. 7 coal bed with 6.1 m (20 ft) wide entries and 24.4 m by 24.4 m (80 ft by 80 ft) center-to-center pillars at a depth of about 100 m (330 ft) in the test area. The seam thickness was approximately 1.5 m (5 ft) in the test area and approximately 0.8 m (2.6 ft) of roof shale was extracted to provide adequate clearance for installing instrumentation.

The immediate roof consists of medium gray, silty shale with a total thickness of approximately 1.5 m (5 ft). This is overlain by a medium gray to dark gray shale unit with gradational contact between the two units. The shale was generally featureless with some lighter gray and possible sandier zones. Stress measurements were conducted at the site and found that the major horizontal stress was aligned nearly parallel to the development direction. For modeling purposes, the vertical stress was 2.4 MPa (350 psi) and the horizontal stress in the plane of the model was 4.5 MPa (650 psi).

The rock properties used in the model are summarized in Table 2. Note the low modulus of elasticity of the shale rocks determined by laboratory tests. Various support types were tested in the experiment. The model simulated full column resin grouted bolts 19mm (3/4in) in diameter and 1.8 m (6 ft) long, with five bolts in a row across the entry width spaced 1.2 m (4 ft) apart. During the test the supports were subject only to the stress changes normally encountered during room and pillar mine development. This implies that the average vertical stress in the surrounding coal increased from the initial cover load of 2.4 MPa to 4.3 MPa, based on the extraction ratio of 43.7%. The ground conditions during support installation were good, with minor cutters formed along the entry edges. During the course of the experiment, the ground conditions remained good and no significant instability was reported.

A model was created of the test entry with supports installed under the initial stress conditions. The model demonstrates that some bedding slip occurs in the immediate roof and limited shear failure near the corners. This agrees well with observed conditions of minor cutters near the entry corners. The vertical loading in the model was increased to 4.3 MPa to simulate the full development of the room and pillar workings, and this additional loading resulted in 2-3mm (1/4 in) of additional roof displacement and an increase in the loading of the supports near the center of the entry from about 20 kN to 40 kN (2 t to 4 t). This agrees well with monitoring results. The supports near the edges of the entry encountered loads of up to 80 kN (8 t), which appears to have been caused by additional damage at the corners. The model indicated minimal changes to the overall stability conditions with the increase in stress.

The stability factor was calculated for the entry under development conditions, and it was found to collapse when the strength was reduced by a factor of 0.51, resulting in a stability factor of 1.98. At the collapse point of the excavation, a chimney-type zone of failure started to form, as shown in Figure 3. This chimney type collapse is typical of bedded materials.

Deep Cover Mine Case Study

The studied mine is a longwall operation where the response of the gateroads and pillars was monitored as part of a NIOSH research effort (Lawson, Zahl and Whyatt, 2012). The depth of cover in the study area varied from 450m to 700 m (1500 to 2300 ft). The roof consists of gray shale overlain by fine-grain sandstone with inter-bedded shale streaks. Coal bands may appear in the shale unit at various locations. The study location is referred to as the 2 North Outby Instrumentation Site. At the study location, a 15 cm (6 in) coal stringer was located at approximately 2.7 m (9 ft) above the entry roof. The sandstone is overlain by further shale beds and coal beds. Table 3 shows the layer thicknesses and rock properties used in the model analyses. The support system consisted of 1.8m fully-grouted rock bolts supplemented by 3.6 m (12 ft) grouted cable bolts. Steel straps and screen were used to provide additional area support.

The horizontal stress in the region is known to be highly anisotropic, with the major horizontal stress being approximately equal to the vertical stress and the minor stress being about one-third of the vertical stress (Maleki, Stewart and Stone, 2007). The major horizontal stress was aligned approximately parallel

Table 2. Key rock properties for modeling the Illinois basin case study.

Rock type	Thick-ness (m)	UCS Lab (MPa)	Elastic modulus (GPa)	Friction angle (deg)	Cohesion (MPa)	Tensile strength (MPa)	Bedding Cohesion (MPa)	Bedding friction (deg)	Bedding tensile strength (MPa)
Shale	15.0	40	3.5	28	6.97	2.32	1.98	23.0	0.10
Shale	4.6	30	3.5	28	5.23	1.74	1.05	23.0	0.05
Shale	0.8	30	3.5	28	5.23	1.74	1.05	23.0	0.05
Shale	1.5	25	3	28	4.36	1.45	0.60	23.0	0.05
Coal bed	1.8	16	3	35	2.42	0.81	1.09	30.45	0.10
Shale	5.2	30	3	25	5.54	1.85	0.50	20.00	0.05
Shale	1.8	30	3	25	5.54	1.85	0.50	20.00	0.05
Shale	5.0	30	3	25	5.54	1.85	0.50	20.00	0.05

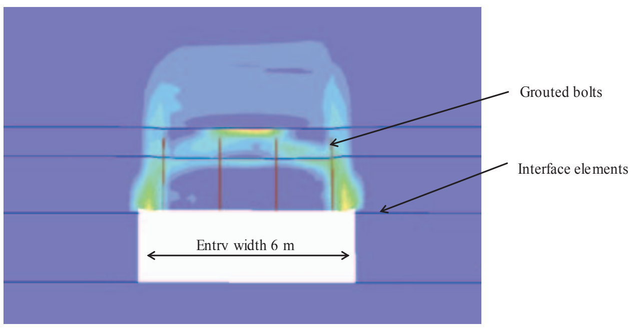

Figure 3. Thick shale case study showing rock damage contours at point of collapse. Warmer colors indicate increasing damage (plastic strain).

to the gate roads. Detailed observations were made of the ground conditions prior to the approach of the longwall and during longwall advance. Initially the roof of the gate roads was undamaged, while the coal ribs showed minor deterioration with less than 50 cm (1.5 ft) of spalling. Conditions deteriorated considerably as the longwall approached, with floor heave and increased rib spalling occurring. However, the roof only showed minor damage with the appearance of tension cracks. Roof displacement monitoring using multiple borehole extensometers showed very limited extension of the roof, less than 5 mm (0.2 in), possibly indicating that the roof rocks did not experience significant stress change.

The numerical model used to simulate this case study is shown in Figure 4, which also shows the principal stress orientation and

Table 3. Key rock properties for modeling the western coal mine case study.

Rock type	Thick-ness (m)	UCS Lab (MPa)	Elastic modulus (GPa)	Friction angle (deg)	Cohesion (MPa)	Tensile strength (MPa)	Bedding Cohesion (MPa)	Bedding friction (deg)	Bedding tensile strength (MPa)
Shale	3.0	70	20	30	11.72	3.91	3.9	25.0	0.10
Coal	1.2	30	3	35	4.53	1.51	2.0	30.4	0.10
Shale	0.9	60	15	28	10.45	3.48	2.9	23.0	0.10
Sandstone	4.8	150	40	42	19.37	6.46	19.3	40.8	6.46
Shale	0.6	36	10	28	6.27	2.09	1.2	23.0	0.05
Coal	0.3	30	3	35	4.53	1.51	2.0	30.4	0.10
Shale	2.8	55	15	28	9.58	3.19	2.7	23.0	0.10
Coal-Mined	2.4	30	3	35	4.53	1.51	2.0	30.4	0.10
Shale	3.0	100	25	32	16.07	5.36	6.2	27.1	0.10
Sandstone	1.2	130	40	42	16.79	5.60	16.7	40.8	5.60
Coal	0.6	30	3	35	4.53	1.51	2.04	30.4	0.10
Shale	10.0	80	25	32	12.86	4.29	5.0	27.1	0.10

the colors indicate the magnitude of the minor principal stress at the initial development loading stage. The figure shows that tensile stress exists in the immediate roof of the excavation, which can result in tensile failure of the roof shale. Although tensile failure was not observed under the initial loading conditions, the first sign of deterioration of rock conditions was typically the formation of tensile cracks in the roof. Tensile fracturing at these depths of cover might be considered to be unusual, but considering that the horizontal stress in the model's plane is about one-third of the vertical stress, such large tensile stresses are not unexpected in the immediate roof of the square excavation. The laboratory scale coal strength was adjusted to 30 MPa (4,350 psi), giving an in-situ coal strength of 17.4 MPa (2,500 psi) in the model, to match the observed minor spalling of the coal ribs.

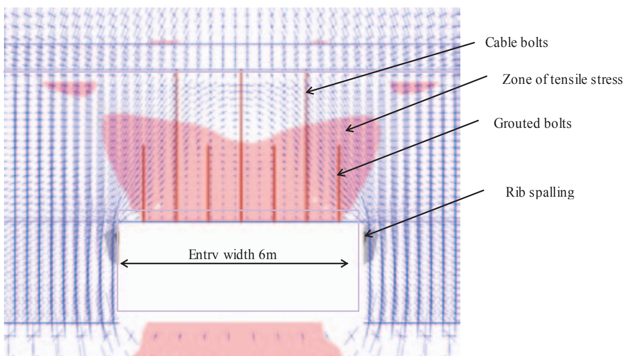

Figure 4. Principal stress plot for the deep western mine case study showing tensile stress zones in the roof and floor of the excavation under development conditions. Tensile stress zones shown in red, rib spalling in gray.

The calibrated model was loaded by an additional 7 MPa (1,015 psi) vertical stress to simulate increased loading caused by the approaching longwall. This stress state caused the model to respond with significant failure of the rib coal, which increased the effective excavation span and induced further tension in the roof. Floor damage was indicated, also in the form of tensile cracking. This model behavior agrees well with the observed deterioration

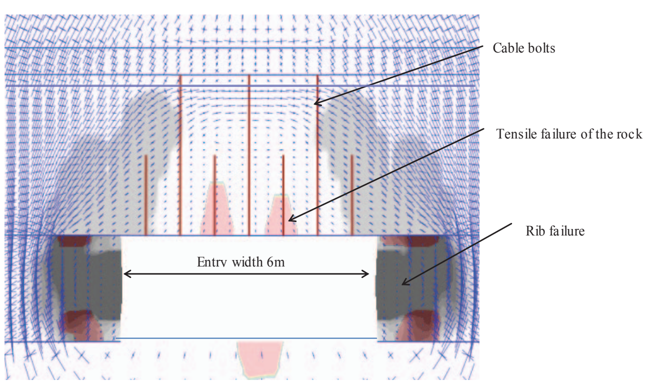

Figure 5. Zones of tensile failure (red zones), rock damage (gray shading) and stress trajectories (blue) for the deep western mine case study under increased vertical loading.

of the entry. Although the additional tensile stress caused vertical fractures to develop in the models up to approximately mid-height of the grouted supports, the grouted bolts and cable bolts did not experience a significant increase in loading. Inspection of the model results showed that the cable bolts extended by no more than 4mm (0.15 in), which concurs with the monitoring results. Figure 5 shows the failure state of the rock around the entry. The stress trajectories indicate that an arch has formed over the entry and the bolts are contained within this arch.

The model results discussed above seem to indicate that the roof of the excavation was in a state of tension, which increased as the longwall abutment loading increased. The roof, therefore, did not experience any direct load increase from the added vertical loading. The function of the support system is to hold the fractured rock in place, which it achieved admirably with the screen and straps attached to the bolts. One would expect the stability factor to be relatively high because the support system is not directly loaded by the surrounding rock mass; stability factor values of 1.63 were calculated for the development loading condition.

At the elevated stress condition, the stability factor of the roof was found to be also 1.63, which at first was surprising. However, it was clear that the roof over the excavation was only affected by increased tension, which caused additional tensile fracturing. This did not cause any significant additional loading of the supports.

Ultimate collapse occurs in the model when the rock strength is reduced to such a state that a circular shear surface forms through the coal ribs and the roof moves downwards as a unit with the cable bolts and grouted supports embedded within, as shown in Figure 6. This collapse mode was identical to the mode of collapse of the roof under development loading conditions, hence the same stability factor.

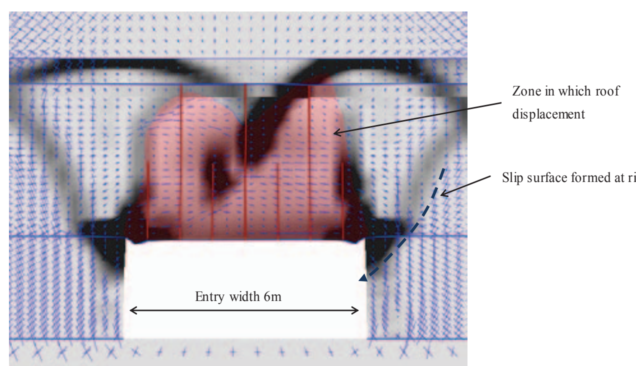

Figure 6. Rock damage (gray-black shading), roof displacement (red shaded area) and stress trajectories (blue) at the point of ultimate collapse of the roof in the deep western case. The red contour indicates displacement of greater than 50mm (2in).

CONCLUSIONS

The method of determining the stability factor through the strength reduction method is not novel, and has been used in other branches of geotechnical engineering. This study showed that the approach can be used as a tool to assist in support design by estimating the relative stability of excavations in various ground conditions, stress conditions and with various support systems. The success of the method depends on developing numerical models that capture the essential failure mechanics of the rock surrounding the excavation and the response of the support system to these changes. The reliability of the models can be improved through a process of calibration against actual observations, as demonstrated in this paper.

Stability indices for the case studies examined were relatively high, which concurs with observations that only a small proportion of mine entries actually collapse during their operational life. The method can sometimes produce unexpected results when a high stability factor value is indicated, but local instability issues, such as unraveling of failed rocks between support units, may exist. It is necessary to clearly separate local stability issues from large-scale collapse evaluations.

The application of numerical models in the design of structures in rock is highly dependent on the quality of the input parameters, as in any engineering design. When designing in variable rock masses, the input values are not fully known and models should be used as a tool to better understand the system under consideration. The determination of a stability factor for supported entries is a step in this direction.

DISCLAIMER

The findings and conclusions in this paper are those of the authors and do not necessarily represent the views of the National Institute for Occupational Safety and Health. Mentioning any company name, product or software does not constitute endorsement by NIOSH.

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