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PRACTICAL APPLICATION OF SUPPORT SYSTEMS TO ADDRESS WEAK ROCK MASS IN UNDERGROUND MINES (UPDATE)

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Abstract: The topic of underground excavation in weak rock mass for civil applications is well analyzed and studied by researchers and engineers. However, limited studies are available for similar ground conditions in underground mining environment. Most of the rock engineering and rock mass classification systems were developed based on tunneling and civil applications of rock, soil mechanics and case histories. The current challenges faced by many underground mine operators are addressing the issues of excavations in poor ground conditions to ensure safe working environment, overall stability of the underground infrastructures and preservation of the mineable reserve conditions and to perform this economically from a mining perspective.

This updated paper provides perspectives on the application of commonly used support systems updated with information on thin spray on liners to address weak rock mass conditions for underground mines based on a practical engineering approach with concepts from mining, civil and tunneling engineering.

1. INTRODUCTION

The increase in rock mechanics understanding, evolution in ground support and reinforcement systems when integrated with practical experience has increased safety and engineering aspects of underground excavations in rock and mining. This has pushed engineering limits to approach excavations in more complex geology, ground water and weathering conditions, higher in-situ stresses and made excavation in weak rock mass possible.

The focus of this paper is to establish a practical perspective of the utilization of modern rock support and reinforcement systems in underground metalliferous mines to address weak rock mass conditions. This paper is an expanded version of Foo et al.'s (2010) publication with the introduction on thin spray on liners which is currently gaining popularity.

The objective of rock mechanics regardless of the mining method is to ensure overall stability of mine structure and safety of working area, protect major underground infrastructure, and preserve the condition of ore reserve for mining (Brady and Brown, 1985).

Rock support and reinforcement are not the only elements that enable underground excavations to meet these objectives. The geological conditions, geotechnical and engineering parameters of the excavated material, excavation orientation, shape, size, sequencing (Brown, 1999) and costs can be some of the governing factors.

The terminology between rock support and reinforcement has been used interchangeably. However, it is essential to differentiate both since they are different in terms of excavation stabilization. Window and Thompson (1993) define "Support" as the application of "reactive force at the face of the excavations" to stabilize the opening, and "Reinforcement is considered to be an improvement of the overall rock mass properties from within the rock mass".

Commonly, rock support or reinforcement is installed in two phases in underground mines as primary, secondary and sometimes to tertiary support system. "Primary" support systems are installed during or immediately after the excavation to achieve stability and improve safety for subsequent excavations. The system is mobilized to conserve rock mass strength by controlling convergence. Additional support installed is classified as the "Secondary" support system, consecutively tertiary and etc. In weak rock mass conditions, secondary to tertiary systems are incorporated to address additional displacement in excavation.

The support system installed can be "Active" or "Passive" depending on its application and types. Active supports are installed with predetermined load to the rock surface while passive support systems depend on the ground deformation to activate the load carry capacity of the support installed.

2. DATA COLLECTION AND ROCK MASS CLASSIFICATION

Data collections and analyses carried out on drill cores provide engineers with preliminary two-dimensional prediction of the rock mass quality prior to excavation. This initial geotechnical information is commonly accompanied by underground mapping when the excavations are made. The mapping reveals actual ground conditions which may not be captured during preliminary assessments. This allows the engineers to fine tune the geotechnical domains, their ground support and reinforcement design to address the ground conditions encountered especially in weak rock mass conditions.

The two most commonly used rock mass classification systems are:

- Bieniawski's (1976, 1989) Rock Mass Rating (RMR).
- Barton et al.'s (1974) Tunneling Quality Index, Q-system.

These classification systems were initially developed for use in tunneling and civil applications where modifications have been made adapting them to mining applications.

The differences between the two classification systems are in the rating weights allocated to similar parameters and usage of distinct parameters on one or the other. The classification systems deal with geology and rock mass geometry in slightly different ways. The RMR incorporates rock mass compressive strength while the Q-system uses in-situ stress. The discrepancy between the two is that the RMR system does not consider stress parameters (Hoek et al., 1995).

Both of these classification systems incorporate Deere et al.'s (1967) Rock Quality Designation (RQD) as a component of their rock mass classification system.

2.1. ROCK QUALITY DESIGNATION (RQD)

Since most drilled cores for exploration and definition drilling vary in core diameter, a threshold correction factor of core length equal to twice the core diameter has been introduced to determine RQD for cores other than NX-size. These threshold values depend on the sensitivity of small diameter cores which tend to break due to drilling and handling (Milne et al., 1989).

Milne et al. (1989) further reports that "the only time the use of threshold values other than 100 mm may be justifiable occurs when it is impossible to differentiate between natural breaks and drill induced breaks". Robertson in 1988 had proposed the Handled RQD (HRQD) for very weak rock mass which is similar to conventional RQD measurement where the core is firmly twisted and bent without substantial force or the use of instruments and tools prior to measurement.

In the absence of RQD measurements, line mapping data can be used to estimate RQD through methods proposed by Priest and Hudson (1976), Bieniawski (1989), Palmström (1982) and Hutchinson and Diederichs (1996).

Priest and Hudson (1976) found that RQD estimation can be obtained from discontinuity spacing measurement made through surface exposure using the following theoretical (1) and linear equation (2):

$$RQD = 100\, e - 0.1\, \lambda\, (0.1\, \lambda + 1), \tag{1}$$

$$RQD = -368\,\lambda + 110.4, \tag{2}$$

where λ is the average number of discontinuities per meter.

The limitation to this estimation is that the database is based on sedimentary rock mass in United Kingdom.

Bieniawski (1989) incorporated the work by Priest and Hudson (1976) by correlating average joint spacing to RQD and it should be noted that the maximum possible RQD corresponds to the best-fit relationship suggested by Priest and Hudson (Milne et al., 1989). RQD estimation based on the proposed method by Bieniawski will lead to a conservative estimate and RQD should not be estimated from line mapping based on the same approach because of the directional nature of the line mapping procedure (figure 1).

Palmström (1982), however, proposed a Volumetric Joint Count, J_v for estimating RQD, where J_v is the measure of the sum of the number of joints per meter of each major joint set or inversely the true spacing for each joint set can be used where

$$RQD = 115 - 3.3\,J_v. \tag{3}$$

The true joint spacing must be used and this measure is valid for rock mass with three (3) or better defined joint sets. The estimated value represents the maximum RQD value (Hutchinson and Diederichs, 1996).

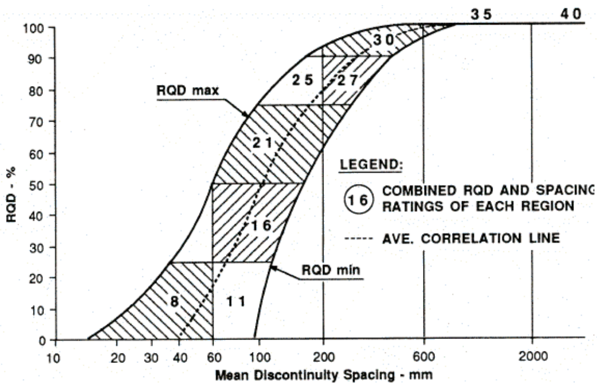


Fig. 1. Relationship between discontinuity and spacing (Bieniawski, 1989)

Hutchinson and Diederichs (1996) proposed another simple method for estimation of RQD with a two meter graded rule placed on the rock surface. The measurement is made similar to RQD, by considering well developed and defined joint sets, omitting any induced or blasting related fractures and fractures less than 300 mm – yielding the maximum value of RQD or “best-case scenario”. The average two (2) to three (3) measurements are recommended to be recorded to reduce directional insensitivity of RQD.

According to Palmström (1982) and Hutchinson and Diederichs (1996), RQD measurements can be utilized as lower and upper bounds to determine the local rock quality for support design.

The disadvantage of RQD is its sensitivity to directional measurement and joint spacing but RQD has been utilized widely to provide initial signalization of rock mass quality. RQD is a starting point and one of the parameters to RMR and Q-system for rock mass classification. A more comprehensive classification system addressing the number of joints, joint conditions, ground water and in-situ stresses will be required to fully quantify the rock mass quality which will be briefly discussed in the following sections.

2.2. ROCK MASS RATING (RMR)

The RMR_{89} (Bieniawski, 1989) incorporates six (6) parameters to classify a rock mass:

1. Uni-axial Compressive Strength (UCS) of rock material.
2. RQD.
3. Spacing of discontinuities.
4. Conditions of discontinuities.
5. Ground water conditions.
6. Orientation of discontinuities.

The summation of all the six (6) parameters yields an RMR value ranging from 8 to 100. In 1989, Bieniawski published guidelines for the selection of ground support in horseshoe shaped rock tunnel with a span of 10 m excavated under drill and blasting subjected to vertical stress of less than 25 MPa (Hoek, 1995). Based on Milne et al. (1989), the advantage of the RMR system is that it is easier to use while some criticisms are that the system is "insensitive to minor variations in rock quality" and the proposed support system appears conservative without recent revisions to reflect new ground support and reinforcement tools.

Because the RMR classification has been updated several times since its initial publications, it is always referred to with a subscript indicating the year (i.e., RMR_{Year}) to identify the version of the classification being used.

2.3. NORWEGIAN TUNNELING QUALITY INDEX, Q-SYSTEM

The Q-system classification system was developed primarily for tunneling design in 1974 by Barton, Lien and Lunde. This classification system can be expressed by:

$$Q = (RQD/J_n) \times (J_r/J_a) \times (J_w/SRF) \tag{4}$$

where the six (6) parameters are:

1. RQD – Rock Quality Designation,

2. J_n – Joint number,
3. J_r – Joint roughness factor,
4. J_a – Joint alteration factor,
5. J_w – Joint water factor,
6. SRF – Strength Reduction Factor.

The initial factor (RQD/J_n) is reflective of the block average size, J_r/J_a factor represents the interblock shear strength and the final factor (J_w/SRF) is more complex representing rock mass active stress conditions in-situ and may be altered by the presence of ground water and structural weakness (Hutchinson and Diederichs, 1996; Hoek and Brown, 1996; Milne et al., 1989).

Based on Milne et al. (year – unknown), the Q-system uses a more rigorous assessment for joint conditions leaving less room for subjectivity. The application of this classification system requires experience and determination of the number of joint sets (J_n) may require extensive geotechnical mapping data which can be a disadvantage for inexperienced users.

The advantage of the system is that it incorporates a well documented database and the support system proposed by Barton and Grimstad (1993) had been updated to account for Fiber Reinforced Shotcrete (FRS) which is currently gaining popularity as part of underground support system.

For the purpose of numerical modelling during excavation design, the Stress Reduction Factor (SRF) in the Q-system becomes redundant (Hutchinson and Diederichs, 1996) since the stress factor will be considered in the numerical model, the SRF value can be set to 1.0 which is equivalent to a moderately clamped but not overstressed rock mass. In most cases, the J_w factor can also be set to 1.0, especially in underground hard rock mines where the excavation is dry or having less than 5 litres/min of local inflow at less than 100 kPa. Hence, the Q-system by setting SRF and J_w factor to 1.0 is called the modified Tunneling Quality index, Q'.

The Q' is commonly used to determine rock mass modulus, strength and has been the empirical open stope dimensioning tool for many mining operations with the introduction of the Stability Number and Graph (Mathews et al., 1981; Potvin, 1988; Bawden, 1993).

The implementation of both rock mass classification systems (RMR's and Q-system) are recommended especially during the initial stages of a project. When site experience is gained, the use of only one classification system best predicting the monitored rock mass can be employed. Despite the difference in the rating values and the measured parameters, it is a common practice in the mining industry to correlate from one classification system to another.

Hoek et al. (1995), recommended the rock mass to be accurately characterized and described during data collection stages. This allows the rock mass attributes, parameters and ratings to be assigned at a later stage enabling the classification to be made in either RMR, Q-system or both. The next section looks into the proposed correlations linking the RMR to Q and their relationship.

2.4. CORRELATION BETWEEN RMR AND Q

The most popular mathematical correlation between RMR and Q was proposed by Bieniawski in 1976 and 1993 where

$$RMR = 9 \ln_e Q + 44. \tag{5}$$

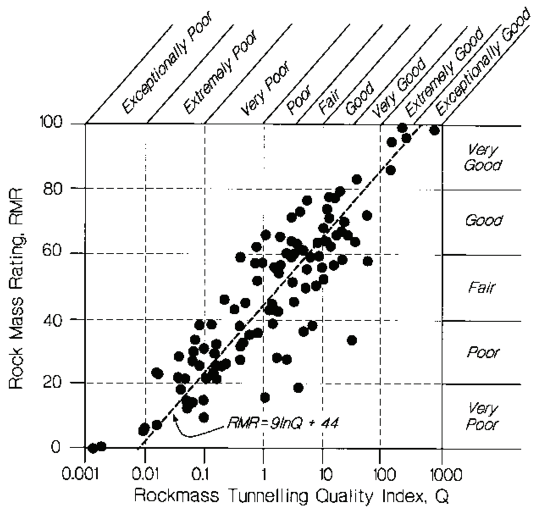


Fig. 2. Comparison between Q and RMR (Hutchinson and Diederichs, 1996)

Table 1

RMR and Q Correlations

Correlation	Source	Comments
RMR = 9 ln Q + 44	Diverse origin	Tunnels
RMR = 13.5 log Q + 43	New Zealand	Tunnels
RMR = 12.5 log Q + 55.2	Spain	Tunnels
RMR = 5 ln Q + 60.8	South Africa	Tunnels
RMR = 43.89 – 9.19 ln Q	Spain	Mining (soft rock)
RMR = 10.5 ln Q + 41.8	Spain	Mining (soft rock)
RMR = 12.11 log Q + 50.81	Canada	Mining (hard rock)
RMR = 8.7 ln Q + 38	Canada	Tunnels, sedimentary rock
RMR = 10 ln Q + 39	Canada	Mining, hard rock
Source: Choquet and Hadjigeorgiou, 1993		

There is evidence that the correlation above does not provide a unique relationship between RMR and Q due to the overall intact rock, discontinuity properties and spacing. Table 1 summarized the correlations derived from various tunneling and mining projects between the two classification systems (Choquet and Hadjigeorgiou, 1993).

The relationship between RMR and Q proposed by Bieniawski (1976 and 1993) is widely used in literature and can be utilized for comparison purposes or where correlation of available information on rock mass modulus is from only one classification system. This correlation should not be used to obtain recommendations based on the other system (i.e., converting RMR to Q for support guideline). This is because of the differences in parameters accounted for in each classification system (figure 3) and the historic database which constructed the correlation (Hutchinson and Diederichs, 1996).

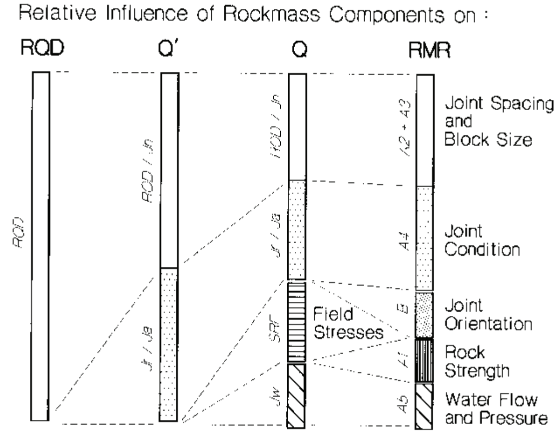


Fig. 3. Rock mass components captured in different classification schemes.
(Hutchinson and Diederichs, 1996)

A database of detailed rock mass characterization or utilization of both classification systems on a specific site or mine can be utilized to mathematically derive the site specific RMR to Q relationship.

3. WEAK ROCK MASS

The definition of rock quality is very well formulated by the classification system, but the definition of weak rock mass is not. RQD, RMR's and Q-system do not specifically address some of the unique characteristics of weak rock related to potential overstressing or deterioration (Klein, 2001). However, the Q-system further breaks down "Very poor" rock quality category to "extremely and exceptionally poor" and introduction of the SRF factor which provides more focus for classification of rock mass having poorer mechanical characteristics compared to RQD and RMR.

Table 2 summarizes the rock quality category and the rating values for the classification systems. Klein (2001) and Clerici (1992) defined weak rock based on their Uni-axial Compressive Strength (UCS) values. The classification for weak rock was based on International Society for Rock Mechanics (ISRM, 1981) rating where 0.25–25 MPa (36–3,626 Psi) UCS is considered as "extremely weak to weak" for tunneling projects (Klein, 2001) or below 20 MPa (2,900 Psi) (Clerici, 1992).

Weak rock can be defined as all rocks having poor mechanical characteristics ranging from soft or weathered rocks which are intensely fractured or altered rock masses, to faults and rocks having characteristics that somehow make them comparable to soil. Weak rocks are often overstressed in a low stress environment due to their low strength characteristics, high deformability (Klein, 2001) where failures are progressive unless there is a significant change in rock mass properties (Hoek, 2006; Bétournay, year – unknown).

In other terms, rocks that have UCS strength of less than 20 MPa (2,900 Psi) or static modulus of elasticity of 150 to 2,000 MPa (21,756–290,076 Psi) (Clerici, 1992) can be categorized as weak rock mass. A rock mass is considered weak when the in-situ UCS is less than approximately one third of the in-situ stress acting on the rock mass being excavated (Hoek, 1999).

Table 2

Rock quality and rock mass classification ratings

Rock quality	RQD	RMR_{89}	Q-System
Exceptionally poor	N/A	N/A	0.001–0.01
Extremely poor	N/A	N/A	0.001–0.1
Very poor	<25%	< 20	0.1–1
Poor	25–50%	21–40	1–4
Fair	50–75%	41–60	4–10
Good	75–90%	61–80	10–40
Very good	90–100%	81–100	40–100
Extremely good	N/A	N/A	100–400
Exceptionally good	N/A	N/A	400–1000

The definition of weak rock mass can also be viewed in the context of ground support requirement as opposed to good quality rock possessing self-supporting capabilities. Based on failure mechanisms of shallow underground mines in weak rock mass, Q values of less than 0.1 can be considered as weak rock mass based on mechanical properties of the rock, failure mechanism and ground support requirements (Bétournay, year – unknown).

The only definition of weak rock mass in relation to rock mass classification system was reported by Mathis and Page in 1995, and Ouchi and Brady in 2004, where rock mass is considered weak when the rock mass rating (RMR_{76}) is below 30 and 50.

4. EMPIRICAL DESIGN IN WEAK ROCK MASS FOR UNDERGROUND MINES

The topic of excavations in weak rock especially in tunneling and civil engineering applications is more widely studied and analyzed when compared to the underground mining environment.

The Spokane Research Laboratory of the National Institute for Occupational Safety and Health (NIOSH) and the University of British Columbia Mining Engineering Department are currently focusing research to incorporate weak rock mass into existing design relationships with the development of the "Critical Span Curve" (Brady, et al., 2005; Ouchi and Brady, 2004).

4.1. CRITICAL SPAN DESIGN IN WEAK ROCK MASS FOR MAN ENTRY

The Critical Span Curve (Lang, 1994) was developed to evaluate the roof or back stability in cut and fill mines based on 172 data sets classified with Bieniawski's 1976 RMR classification system (RMR_{76}) for rock mass ranging from 60–80 or good rock. In 2003, the database was updated with 292 data sets, where more than 60% of the database consists of good rock, less than 10% and 20% with RMR_{76} below 40 and 55, respectively (Wang et al., 2002; Brady et al., 2005; Ouchi and Brady, 2004).

By 2004, the database was further augmented with an additional 135 data sets in the weak rock mass range ($RMR_{76} < 50$) which increases the accuracy and reliability of the Critical Span chart. The weak rock mass database is composed of RMR_{76} values ranging from 15 to 62. The excavation span for this data set varies from 1.5 m to 12.8 m. with 93% of the case histories having excavation span of 7.6 m (Ouchi and Brady, 2004). Figure 4 presents the critical design span curve based on the augmented weak rock mass data set.

The term "critical span" for the use with the empirical design chart of figure 4 refers to the largest circle which can be inscribed or drawn within the boundaries of the excavation and the term "Design Span" refers to spans that have no support to limited support consisting of pattern bolting with 1.8 m long bolts on 1.2 m × 1.2 m spacing.

The value of RMR_{76} is recommended to be reduced by a value of 10 for excavations with shallow dipping joints and in areas where high stress environments are expected (Ouchi and Brady, 2004; Brady et al., 2005).

This database is perhaps the only systematic data collection approach made as an empirical design tool in rock mechanics for underground mines correlating weak rock mass with classification system to determine the excavation span.

It is imperative for mines to adopt caution when utilizing the empirical design tools for actual design due to variability in field conditions and ground support systems utilized. However, this Critical Span Curve provides engineers with an initial empirical tool to determine excavation size in such rock mass conditions.

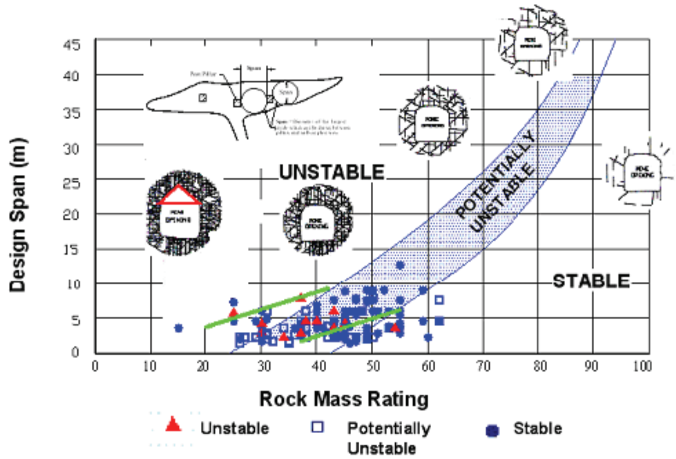


Fig. 4. Weak Rock Mass Design Span Curve for Man Entry (Ouchi and Brady, 2004)

5. UNDERSTANDING WEAK ROCK MASS SUPPORT SYSTEM

The behaviour of weak rock deformation is predominantly controlled by rock mass properties such as: low strength, high deformability, discontinuities (Klein, 2001), weathering or alteration conditions and mechanical disruption such as blasting and excavation stand up time (Mathis and Page, 1995). Hence, the support and reinforcement installed will have to be designed to address these conditions. Good engineering judgment and understanding of the rock mass constraints by proper excavation shape and mine layouts can drastically improve stability and reduce costs. In underground mining, the ground deterioration can be accelerated with improper mine design or excavation sequencing. A good prediction and understanding of the estimated induced stress acting on pillars or openings, proper mine sequencing and arching of excavation roofs can greatly improve stability.

5.1. DEFORMATION CHARACTERISTICS OF ADVANCING EXCAVATION

Rock starts to react into incoming excavation about one-half excavation diameter ahead of the advancing face (figure 5). At the excavation face, one third of the total deformation will occur and the maximum plastic deformation is achieved by one and one-half diameter behind the excavated face (Hoek, 1999). In scenarios with brittle rock under high stress conditions, distressing can be performed to relief and redistribute high stress concentration which can reduce the potential of rock bursting.

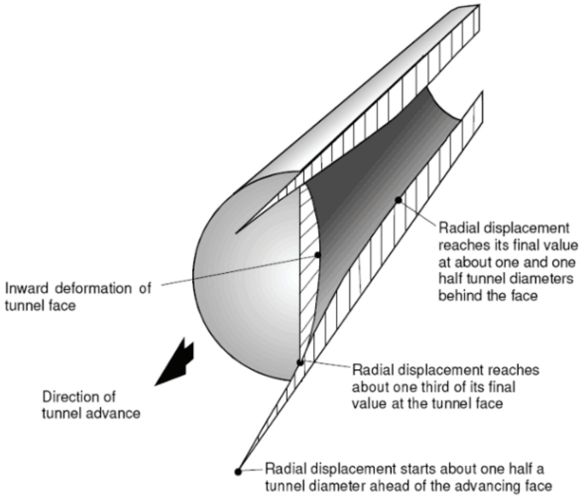


Fig. 5. Deformation of rock mass into oncoming circular excavation (Hoek et al., 1995)

However, the potential instability issues of the excavation under the plastic deformation in weak ground depend on the ratio between rock mass strength and the state of in-situ stress.

Brittle rock subjected to overstress tends to fracture under slabbing or spalling and if support or reinforcement is not installed on time, it will cause detachment of the slabs or blocks. Rock mass exhibiting ductile characteristics will yield and converge or squeeze around excavations. Weak rock types are more subjective to progressive failures. In some cases, all the failure modes described above can be observed within a mine possessing variable geological and rock types.

5.2. SUPPORT AND REINFORCEMENT SYSTEM

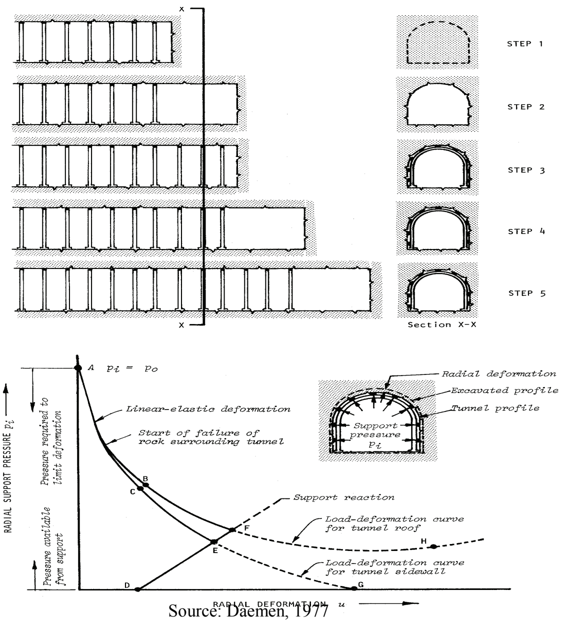
When the critical span of the excavation is determined, the support design and reinforcement system installed has to be in harmony with the ground conditions and deformation. The attempt to achieve “zero” displacement with stiff support system is not possible and will induce unnecessary high support pressures.

The progressive failure nature of weak ground is complex, difficult to analyze with no straightforward mathematical or numerical analysis models or recommended factor of safety defining an acceptable limit for the failure process (Hoek, 2006). The adequacy of the support designed, selection of support types and reinforcement installed depends on the stress induced, the deformation magnitude and support pressures. All of these will be based on the judgment and experience of the engineers and the purpose of the excavation.

Fig. 6. High stress condition in brittle rock mass

5.2.1. GROUND CHARACTERISTICS AND SUPPORT REACTION

The rationale behind the successful support and reinforcement design relies on the understanding of the interaction between the support or reinforcement elements and the behavior of the excavated rock mass. This is best described through the ground characteristic and support reaction curves on a hypothetical excavation being advanced by drilling and blasting supported by steel sets (Daemen, 1977).



Source: Daemen, 1977

Fig. 7. Hypothetical drilling and blasting tunnel excavation with steel set support and load deformation curves

The ground characteristic or required support is given by ABFH, ACEG and DEF curves (figure 7). Points ABFH and ACEG represent the behaviour of the roof and side wall of the excavated rock mass. Extra support pressure is required to limit displacement from the roof because of the loosening of material from the roof induced by gravity (Brady and Brown, 1992). The ground characteristics and support requirement is described by stages below.

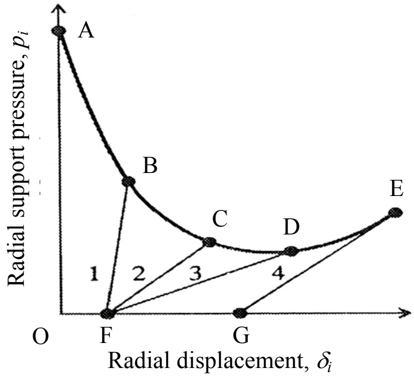
At step:

1. Prior to excavation at point A – the "system" is in equilibrium where the internal support pressure, P_i is equal to and acting opposite of pre-mining stress state, P_o.
2. When the excavation of rock passes X-X, the excavation will converge and displace along line AB for the roof and AC for the wall, with a radial displacement value of D.
3. At point D, when the heading is being mucked out and support been installed close to the face, the support installed does not carry load because no deformation has occurred since installation.
4. Once the excavation advances one to one and a half diameter from X-X, the support provided by the proximity of the face is negligible. Hence, further radial displacement occurs and will continue along line BF and CE. With installed support, the radial displacement induces load onto the support system which shows linear stress-displacement behavior along path DEF. This path (DEF) is known as the support reaction or available support line. Equilibrium between the support installed and rock convergence is achieved at point E for the side wall and F for the back.
5. If no support is installed the displacement will increase and continue along the dashed line EG and FH. The side wall will achieve equilibrium at point G or may continue to increase depending on rock mass type. However, the displacement from the roof will reach minimum and then begins to increase as rock becomes loose and the weight of this material will increase the support pressure.

5.2.2. SUPPORT IN YIELDING ROCK

The stiffness of the support elements and the installation timing plays an important role in displacement control. Figure 8 illustrates a similar support interaction diagram with line ABCDE demonstrating the ground characteristics followed by different stiffness support system and installation periods. The earliest support installation time is at point F, after experiencing an initial radial displacement value of OF.

1. Support No. 1 installed after the excavation was cleaned out reaches equilibrium at point B. This support is too stiff, attracts excessive load redistribution and may fail causing catastrophic rock mass failure surrounding the excavation.



Source: Brady and Brown, 1993

Fig. 8. Support stiffness and installation timing

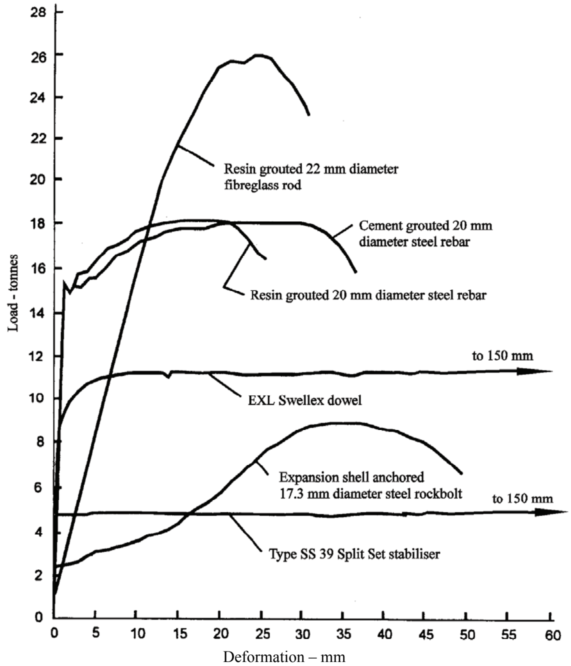
2. Support No. 2 with less stiffness, installed at point F reaches equilibrium at point C. This system provides good support only if the displacement is acceptable operationally wise.
3. Support No. 3 has lower stiffness than support No. 2, installed at point F, reaches equilibrium at point D when the rock mass has loosened can create dangerous situations because of excessive loading by stress redistribution.
4. Support No. 4 has similar stiffness to support No. 2 but not installed until excessive radial displacement had occurred until OG. The support installed is too late and will likely be overstressed before equilibrium is reached.

Eberhardt (2005/6) indicated that another conclusion can be drawn from the ground response and support curves for the case of unstable plastic deformation conditions; a stiffer support system may be more successful in controlling the ground than a “soft” support system. As highly jointed or foliated weak rock mass becomes more fractured and experienced loss of strength, the ground characteristics curve will become progressively flatter which is similar to the reduction in rock modulus and higher support pressures will be required to reach equilibrium.

6. PRACTICAL APPLICATION OF SUPPORT SYSTEMS IN WEAK ROCK MASS

Most design charts for ground support and reinforcement provide excellent guidance on support selection based on rock quality. However, very limited research and literature exists on the practical application of a support system under weak rock or yielding ground conditions.

Based on practical engineering experience in weak ground conditions, the application of a primary and secondary support system with additional support retention system will be required to stabilize the excavation (Foo et. al., 2006). This is due to the nature of the yielding ground behavior, support and rock properties and the stresses acting on the opening. The ground characteristic and support reaction curves illustrated in the previous sections simplifies a very complex and difficult topic but provides an essential input to the engineer during design.



Source: (Stillborg, 1994)

Fig. 9. Examples of non-linear reaction curves for various support types

The objective of a practical application of support and reinforcements in weak rock mass conditions is to:

- Avoid the achievement of "zero displacement" by introducing support with sufficient stiffness
- Control the displacement of surrounding rock with a combination of support systems by maximizing the support elements mechanical characteristics.
- Allow the rock mass surrounding the excavation to reach assisted self-supporting stage with the system installed.

Figure 9 presents some examples of non-linear support reaction curves for various support elements or types (Stillborg, 1994). The system installed should be in harmony with the ground behavior to achieve optimal equilibrium in a timely and economical manner without jeopardizing safety.

6.1. SHOTCRETING APPLICATIONS

The applications of shotcrete in weak rock mass have enabled many underground mines to be excavated safely. The initial layer of shotcrete reinforces the inherent rock mass strength and renders it self-supporting by stabilizing the excavation as a primary support. However the nature of shotcrete and it stiffness limits the amount of displacement capacity of this system. Figure 10 shows the limitation of Steel Fiber Reinforced Shotcrete (SFRS) as primary support in high deformation Talcy rock mass. Shotcrete is stronger under compression typically at 40 MPa (5800 Psi) whereas their flexural strength is around 5 MPa (725 Psi) while their adhesion strength is at approximately 5 MPa (725 Psi) (Clement, 2003).

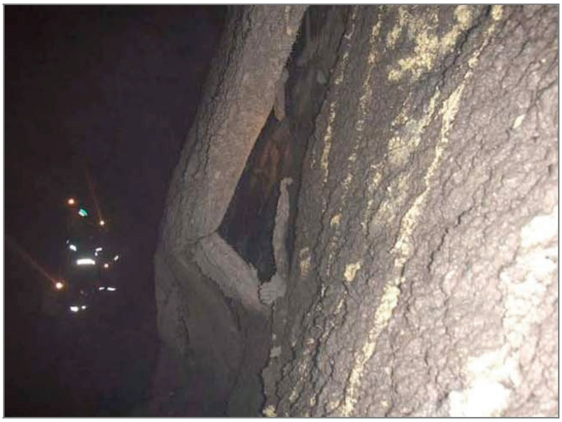


Fig. 10. Support is too stiff to the intended ground deformation

In underground mining applications where the support systems are expected to yield beyond their yielding capacity, additional support systems with surface retention (i.e., straps or modified plate), welded mesh and in some cases cable bolts are installed to control convergence (Foo et al., 2006). In extremely to exceptionally poor ground, spilling in advance of the face may be required to control cave-in, however, the use of spilling in underground mining applications are very limited.

A “Sandwich” or “Two-layer” shotcrete application concept is imperative for controlling highly deformative ground. The first layer of shotcrete is commonly sprayed at the excavation immediately after blasting and when the face is being cleaned-out. When this layer achieves its initial design strength, welded mesh is installed with suitable bolt types and face plates. The bolt types vary from frictional bolts, grouted rebar, mechanical bolts or a combination thereof. A second layer of shotcrete is then applied over the mesh and bolts. Shorter frictional bolts such as Split Sets are commonly used to hold and deform the mesh in place onto the rock surface during installation to minimize voids between the rock and mesh.

To improve stability of excavations in very weak ground conditions, Fiber Reinforced Shotcrete (FRS) can be applied during the first pass and the second pass. For most cases, the second pass shotcrete can be without fiber to reduce rebound on the welded mesh. However, this depends on the performance of the applied support and reinforcement system which can only be determined through reconciliation and monitoring of the ground-to-support reaction.

This “Sandwich” or “Two-layer” shotcreting concept has been used to excavate through broken ore, fault zones, poorly consolidated backfill to partially oxidized muck pile (Langille et al., 1997) and has been able to control deformation of up to 300 mm in 90 days (Clement, 2003). The imbedded welded wire mesh provides more deformation and better load-carry capacity to the shotcrete (figure 11).

Fig. 11. FRS “Sandwich” to control deformation

The interaction between the face plates and the rock surface is important in restricting convergence. To achieve the maximum benefit of shotcreting, rock bolting is recommended to be placed above the shotcrete after the initial layer (Clement, 2003). If deeper anchoring bolts are required, cable bolts are commonly incorporated. Cable bolts are installed and tensioned after the second layer of shotcrete. Hence, the implementation of an “Active” support system is recommended. The face plate dimension can be increased to prevent “punch-through” of the plate and to distribute the pressure exerted by cable bolts.

The authors’ experience in dealing with weak, highly deformative rock mass subjected to high stress conditions show that once the crack propagation of shotcrete exceeds the tensile strength (fiber or fibreless), the excavation stability depends purely on rock bolts and surface retention systems. This observation was also reported by Clement (2003). FRS alone without any bolting or supplementary retention system has limited capability to control ground conditions under such circumstances (Foo et al., 2006; Clement, 2003; O’Donnell, 2003).

Fig. 12. Example of steel sets and concrete support (top)
vs. cable bolting and shotcrete (bottom)

The use of shotcreting, rock bolting and cable bolting systems has also been successfully implemented to replace steel sets in weak rock mass (figure 12). This had increased excavation to support installation cycle time and drastically reduced costs without jeopardizing safety (Lima et al., 2008). The deep anchoring reach of cable bolts into competent ground assisted in controlling the yielding weak rock (Foo et al., 2006; Yumlu and Bawden, year – unknown).

Figure 13 shows examples of the applications of bolting and shotcreting as primary and secondary support. The excavations on the top three photos are supported with passive bolting followed by plain shotcrete. The stiffness and the lack of flexural capabilities of the shotcrete were not able to control the side wall convergence and the loading from the roof. As comparison, the addition of steel fiber into the shotcrete increases the ductility of the support system to control the crack propagation (lower left picture). The addition of steel fiber more than doubles the fracture energy of plain shotcrete (Melbye, 1999). The “sandwich” combination of shotcreting, bolting, meshing with cable bolting system was more effective, able to take higher deformation and limiting the ground movement even when the cable bolts were not tensioned.

Shotcrete without Fiber Reinforce is subject to cracking when ground deformation occurs. This detaches the shotcrete from the rock, generating slabs which eventually fall under the influence of gravity.

Left: SFRS with mesh, bolts & cable bolts to control high ground deformation even when cable bolts tensioning were delayed. The support system provides sufficient retaining forces.
Top: Initial stage of crack propagation in SFRS, support system still controlling deformation.

Fig. 13. Examples of shotcrete applications in weak rock mass

6.2. THIN SPRAY ON LINER (TSL)

TSLs (figure 14) are rapid setting, thin (2–4 mm) polymeric liner materials that have been used for underground rock support since 1990's and gaining a lot of interest amongst operating mines.

Fig. 14. Typical TSL application (Picture by: Tony White & Associates/Concor)

TSls has proven economic benefits such as reduced cost, logistics, and shorter cycle times but its wide application use, however is still on trial basis. Esterhuizen and Bosman (2009) reported the following cost comparison of shocrete and liner (table 3).

Table 3

Cost comparison

	4 mm TSL	8 mm TSL	25 mm shotcrete**	50 mm shotcrete**
Total cost (R*/m^2)	140	205	190	230

The above cost does not include transportation cost
* – South African Rand
** – Not fibre reinforced

Before TSL application in underground, the surface must be clean and water pressure scaling of the surface is recommended. Components of the liner product are mixed in a mixer and pumped to the surface with a nozzle. The curing time of TSLs changes from minutes to hours, depending on the product.

TSL provides a combination of ground support and reinforcement functions similar to shotcreting, through strong adhesive bond between the rock and liner interface with high tensile strengths. Both shotcrete and TSL have functional similar characteristics to certain extend since this support or reinforcement system creates a support resistance at small displacements in the order of millimeters, but unlike conventional bolts or shotcrete with brittle characteristic, the high degree of liner plasticity feature of TSL allows it to distribute loads over a greater coating area. When used in conjunction with screen, TSL achieves a high load-carrying capacity, equivalent to or exceeding the strength of reinforced shotcrete. Commonly, a thin layer of shotcrete fails suddenly in shear or tension, in contrast to polyurethane and polyurea TSL which deform with the rock and continue to provide support even after the rock fractured, displaced and became “loose”. Another important component of the support function is the liner’s membrane ability to deform yet carry load like a suspension bridge (Tannant et al., 1999).

Tannant (2001) schematically compared (figure 15) different load displacement performance for various areal support types. Liners are expected to have performance characteristics that lie between mesh and shotcrete.

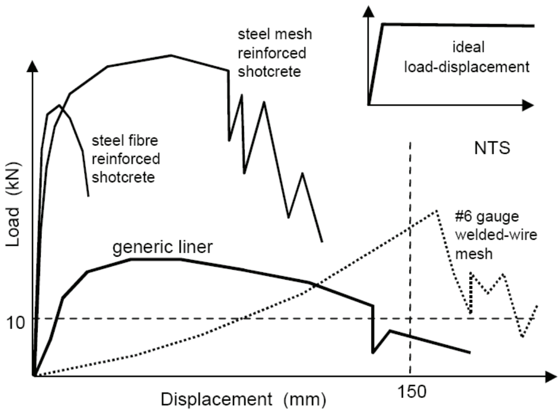


Fig. 15. Schematic load displacement curves of areal supports

A TSL’s tensile strength, adhesive strength, and strain are properties that are important to the liner’s ability to hold loose rock in place (Ozturk and Tannant, 2010).

Given the importance of achieving sufficient coverage across fractures between rocks in order for a liner to work effectively, liner use is not recommended in environments where wide rock fractures exist. While the load capacity of a sprayed-on membrane may exceed that of welded-wire mesh, a membrane’s support capacity is also quite sensitive to many factors that do not affect mesh. For example, liner effectiveness can be degraded by poor adhesion to the rock, non-uniform liner thickness, and incomplete surface coverage. A liner that contains thin areas or is discontinuous will contain inherent zones of weakness that may lead to premature failure of the liner’s support function. Similar to shotcreting, the effectiveness of TSL requires higher quality control, attention to the procedures and equipment used to prepare and spray the material on the rock.

Table 4 presents some tentative application guidelines for TSL as a ground support and reinforcement tool based on field investigations performed by Espley et al. in 1999.

Table 4

Tentative application guidelines for TSLs

<table>
<tr><th>Description</th><th>RMR_{76}</th><th>TSL thickness (mm)</th><th>Bolting pattern[1)]</th></tr>
<tr><td rowspan="2">Development drift (walls)</td><td>45–65</td><td rowspan="2">2–3</td><td>1.8 m × 1.5 m[2)]</td></tr>
<tr><td>> 65</td><td>1.8 m × 1.5 m[3)]</td></tr>
<tr><td rowspan="2">Development drift (roof or back)</td><td>45–65</td><td rowspan="2">3–4</td><td>1.8 m × 1.1 m[4)]</td></tr>
<tr><td>> 65</td><td>1.8 m × 1.3 m[4)]</td></tr>
<tr><td rowspan="2">Prod. headings (lower wall)</td><td>45–65</td><td rowspan="2">2–3</td><td>1.8 m × 1.5 m[5)]</td></tr>
<tr><td>> 65</td><td>Boltless or spot bolting[5)]</td></tr>
<tr><td rowspan="2">Prod. headings (roof and back)</td><td>45–65</td><td rowspan="2">3–4</td><td rowspan="2">1.8 m or 2.4 m × 1.1 m[4)]</td></tr>
<tr><td>> 65</td></tr>
<tr><td colspan="4">Note:
1) Mechanical bolts.
2) Bolting after every two rounds of advance.
3) Indefinitely delayed with bi-annual audits.
4) Install before or immediately after liner.
5) Installation can be delayed.</td></tr>
</table>

Currently, the use of TSLs as a ground support and reinforcement tool in underground mines is in initial phases and its use is limited. TSLs had yet to be proven suitable for high-stress and squeezing conditions, and therefore until controlled field trials validate the use of TSLs in these conditions, mesh or shotcrete combined with rock bolting is still more reliable as the preferred support system. Similarly, TSLs are not recommended in areas requiring rehabilitation work until further experience is gained with liners as a support component.

6.3. BOLTING AND MESHING APPLICATIONS

Systematic application of bolting and meshing accompanied by secondary support with surface retention system such as steel or mesh straps also has the capability of controlling high deformation. This system has less stiffness compared to plain or FRS but has higher yielding capacity in limiting progressive ground movement. Wire mesh has the tendency to “sag” or “bag” when loaded with loose rock during ground convergence. This does not mean that the bolts installed had failed, if the bolts are designed to the correct anchorage depth (figure 16).

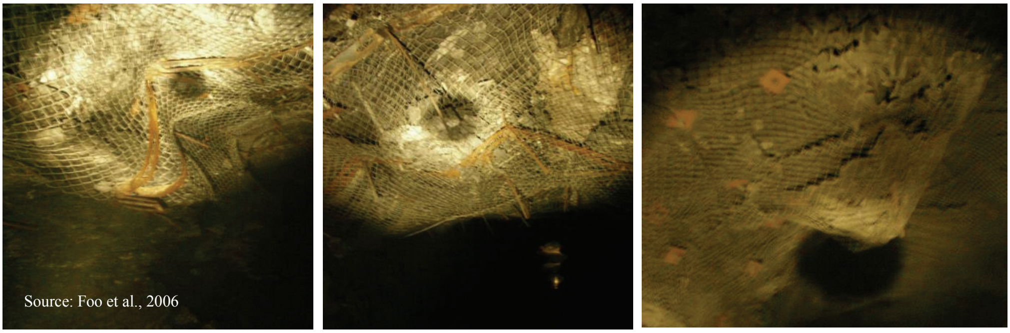


Fig. 16. Bagging of loose rock in bolting, meshing with cable bolting and straps

Through monitoring of loading and displacement with SMART Cable bolts, Yumlu and Bawden (year – unknown) had reported that the plastic deformation zone ranges from 1 to 3 m beyond the excavation boundary for the Cayeli Mine excavated in weak rock mass. This zone is just slightly deeper than the average mines' standard support length of 2 to 3 m. In order to control the deformation more satisfactorily, the support has to have tougher surface retention capabilities with deep-seated high capacity anchorage properties. The depth of the plastic deformation zone at Cayeli has also been observed by the authors of this paper in Canadian and Latin American mining operations in weak rock mass.

Figure 17 shows the effectiveness of the combination of utilizing bolting, meshing, cable bolts and straps to control high deformation ground in serecite schist located in pillars between transversal drawpoints in contrast to shotcreting. The convergence of the pillar is caused by the low modulus of the rock properties and the induced stresses as mining progresses. For this case, the rock mass was predicted to yield, excavations were monitored and support systems were designed to achieve equilibrium at different stages of the ground characteristic line. The application of steel straps for this scenario was more effective with cable bolting when installed perpendicular to the schistosity of the rock mass.

This paper lists some of the successful practical examples in the utilization of a combination of ground support and reinforcement systems in addressing the challenges faced by engineers working in weak rock mass environment. The support elements addressed in this paper are common ground support or reinforcement. Some products are commonly used while others are still in field investigation stages.

Depending on the trained personnel resources, complete replacing or switching from one support or reinforcement system (i.e., shotcreting to bolting and meshing or vice versa), can be time-consuming and dangerous. A detailed engineering analysis, an understanding of the site conditions to availability of trained personnel and supervision will be required prior to such major modification. A phase by phase modification process is recommended and in most cases if the support or reinforcement systems are not capable of achieving equilibrium, then a combination of support will likely be required to control deformation.

Fig. 17. Effectiveness of bolting and meshing (left) vs. SFRS in controlling ground deformation (top and bottom right)

7. CONCLUSION

Underground mineral extractions in weak rock mass have been performed traditionally with timbers and square sets successfully for many years. With increased demand for higher production rates, increased safety, economic incentives, modern support elements and better understanding of rock mechanics have enabled engineers to push the boundary to approach excavations in more complex geology, ground water and weathering conditions, higher in-situ stresses and made excavation in weak rock mass possible.

The evolution of rock mass classification systems and their support guidelines provided engineers with a standardized approach in characterization of rock quality to preliminary design guidance. Rock mass is recommended to be accurately characterized and described during data collection. From there, the application of the most appropriate classification systems can be made. Some mining operations incorporate various classification systems to derive site specific correlations.

Caution is required when incorporating empirical design tool such as the Critical Span Curve because of the variability in field conditions, rock types, mining methods and ground support systems utilized. However, this tool provides initial parameters for mine design. Detailed engineering analyses will be required to ensure overall stability of the excavations and safety of personnel.

The objective of the support system is to achieve harmony and equilibrium with the ground deformation in a timely and economical manner without jeopardizing safety. The capability of engineers to predict the ground behavior prior to excavation, verification of in-situ conditions and opportunities to reconcile and optimize the design is imperative as conditions vary.

All support systems have their advantages; shotcrete is effective in providing rapid support with limited direct personnel exposure in unsupported ground. However, its limited ductility makes it less suitable in progressive yielding ground conditions that causes load carrying capacity to be transferred to the nearest point of support; bolts, cables and the surface retention system. Rock bolting and screening have been the common prescribed rock support element in many operations partially due to cost effectiveness and availability of trained personnel. But the issues pertaining to bolting and screening are the exposure of personnel under unsupported ground and a bolting process is slower. The current advancement in shotcreting technology is converting the view of many engineers. TSLs with their low cost, logistics and shorter cycle time advantages are being preferred as an aerial support. In addition, TSLs large deformation capability without failure makes them very attractive.

An attempt to achieve "zero" displacement with stiff support system is not possible and will induce unnecessary high support pressures. Hence, if the current support system is not capable of achieving equilibrium, then a combination of support systems will likely be required to control deformation.

A complete modification of support system or switching from shotcreting to screening or vice versa can be time-consuming and dangerous. A phase by phase approach is recommended during the introduction of a new support system to ensure smooth transition in the training, supervision and quality control in the introduction of the new support element.

This paper provides some examples and practical approaches in utilizing the available support and reinforcement systems to address weak rock masses through experience gained from various operating mines. Some of the concepts outlined are hypothetical and empirical but provide an understanding of the engineering concepts, while other practical applications have been utilized successfully.

The topic of underground excavation support in weak rock masses for mining engineering applications offers ample opportunities for research.

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