ISSN 2303-8403 Broj 3, 2017

DESIGN PRINCIPLES FOR UNDERGROUND ROCK SUPPORT

Prof Ph.D. Charlie C. Li, rock mechanics for civil and mining engineering
Department of Geoscience and Petroleum, Norwegian University of Science and Technology

Summary:

The paper introduces some principles for underground rock support design. The topics include underground loading conditions, the natural pressure arch in the rock mass, design methodologies, determination of the factor of safety and compatibility between support elements. A natural pressure arch is formed in the rock mass in a certain distance behind the tunnel wall. The methodology of ground support in an underground opening is dependent on the size of the failure zone and the boundary depth of the natural pressure arch. In the case of a small failure zone, rockbolts should be long enough to reach the natural pressure arch. In the case of a vast failure zone, an artificial pressure arch could be established in the failure zone with tightly spaced rockbolts and the artificial pressure arch is stabilised with long cables anchored on the natural pressure arch and/or by external support elements like shotcrete liners, girdles, steel arches and shotcrete arches. In addition to the factor of safety, the maximum allowable displacement in the tunnel and the ultimate displacement capacity of support elements should be also taken into account in the design. Finally, the support elements in anunderground support system should be compatible in displacement and energy absorption.

Key words:

Rock support, Support design, Pressure arch, Rock bolting, Factor of safety, Compatibility

1. INTRODUCTION

Rock support design is associated with the rock mass quality, the in situ stresses and the size and geometry of the underground opening. Knowledge of the in situ loading condition is crucial for the design of underground rock support. The methodology and design principles of a rock support system are determined by the potential failure mode and failure extent of the rock mass as well as the engineering requirements to the maximum allowable displacement. In this paper, some key parameters for underground rock support design are presented. The parameters include the natural pressure arch, the artificial pressure arch established in the failure zone, support layers, the factor of safety, and the compatibility between support elements.

2. UNDERGROUND LOADING CONDITIONS

2.1. In low stress rock masses

Low stress hereby refers to the state of in situ stress that cannot stabilize rock blocks in the roof and may fall under gravity. In locations close to the ground surface, the rock mass often contains well-developed rock joint sets and the in situ rock stresses may be low. Wedge blocks may be formed in the tunnel roof/walls after excavation. The major instability issue is thus falling of loosened blocks under gravity. The task of rock support in this case is to prevent the loosened blocks from falling. To do so, the maximum load exerted on the support elements, such as rockbolts, is the deadweight of the potentially falling block (Figure 1). This is a load-controlled condition.

From the point of view of mechanics, the rockbolts must be strong enough to bear the deadweight of the loosened rock block. Therefore, use of a factor of safety, defined by the strength of the support system and the weight of the block (that is, the load), is appropriate for rock support design in a load-controlled condition. This is essentially the design principle in structure mechanics, which states that the load applied to a structure should not be higher than the strength of the structure, that is, the strength-to-load ratio, otherwise known as the factor of safety, should be larger than 1. This principle is valid in structure constructions on the ground because the total load on the structure in question is usually known or easily found out. In shallow underground openings, this principle is also valid since the maximum load on the rock support system is the deadweight of loosened rock blocks.

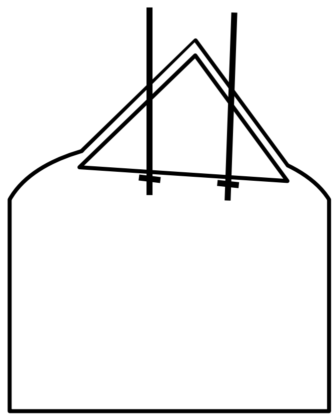

Figure 1.Gravitational load on a rockbolt in low stress condition.

2.2. In high stress rock masses

In the period of his work in a deep metal mine, the author observed that geological discontinuities in the rock mass become less in density and less opened with an increase in depth. For instance, at a depth of 1000 m, it was observed that all discontinuities (not many) exposed on excavation faces were completely closed. Therefore, it can be said that the rock mass quality is improved at great depth because of the reduction in the number of geological discontinuities. However, the in situ rock stresses increase with depth. At depth, the major instability issue is no longer the falling of loosened rock blocks but rock failure caused by stress. High stresses could lead to two consequences in underground openings: large deformation in soft and weak rock and rockburst in hard and strong rock (Figure 2). It was observed in some metal mines in Sweden that strain burst usually occurs below a depth of 600 m and becomes intensive below 1000 m. Rock failure is unavoidable in high stress conditions. The task of rock support at depth is not to equilibrate the deadweight of loosened blocks but to prevent the failed rock from disintegration. In high stress rock masses, the support

ISSN 2303-8403 Broj 3, 2017

system must be not only strong but also deformable in order to deal with either stress-induced rock squeezing in weak rock or rockburst problems.

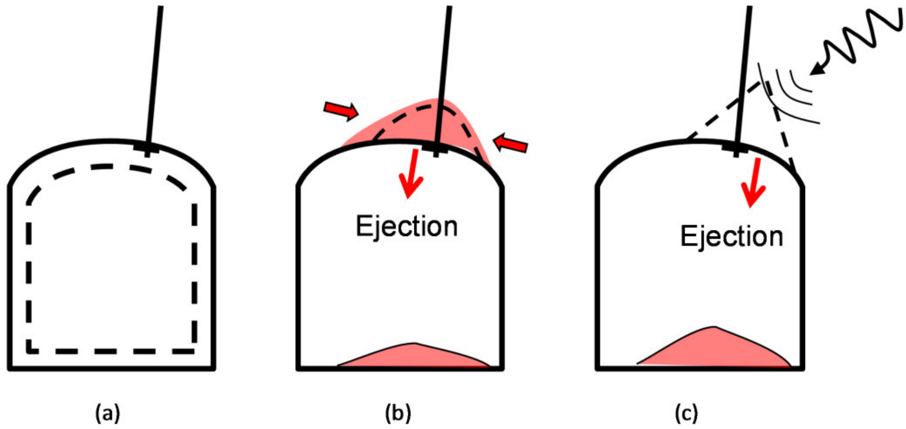

Figure 2. Loading conditions to rockbolts in high stress rock masses. (a) Rock squeezing, (b) strain burst, and (c) fault-slip burst.

3. NATURAL PRESSURE ARCH

Geological exploration drilling was once carried out in a mine drift, excavated 5 years previously, at a depth of 1000 m. The boreholes were drilled in the wall of the drift on the side facing the ore body approximately 150 m from the drift. The fracture logging on the cores provided information on the distribution of the secondary stresses in the rock surrounding the drift. Figure 3 shows the fracture patterns in the cores taken from a horizontal borehole. The fracture intensity in the cores is different at different depths. The cores are small pieces with a low value of RQD (Rock Quality Designation) in the zone from the wall to a depth of 2.1 m (Zone I). The fracture surfaces in this zone are yellow coloured, indicating that they were probably created when the drift was excavated a few years earlier. The cores are disked in the zone from 2.1 m to 8.5 m (Zone II). The fractures in this zone are fresh and perpendicular to the core axis. It can be said with confidence that they were created during the core drilling. Zone II can be further divided into two sub-zones. In Zone IIa the core disking is so severe that the disks are tightly spaced. The disk thicknesses are obviously larger in Zone IIb than in Zone IIa. Zone III is from a depth of 8.5 m to the end of the borehole. The discontinuities in this zone are believed to be mainly of geological origin. The RQD of the cores in Zone III is significantly higher than the other two zones, which implies that Zone III is out of the disturbance distance of the drift. On the basis of the variation of the fracture intensity, it is inferred that Zone I is the failure zone, where the rock fails either in shear or in tension and the tangential stress is partially reduced, while the tangential stress in Zone II is elevated after excavation of the drift but the rock has not yet fractured in situ. Zone II is the position of the natural pressure arch that bears the ground pressure and functions as a protection shield over the drift.

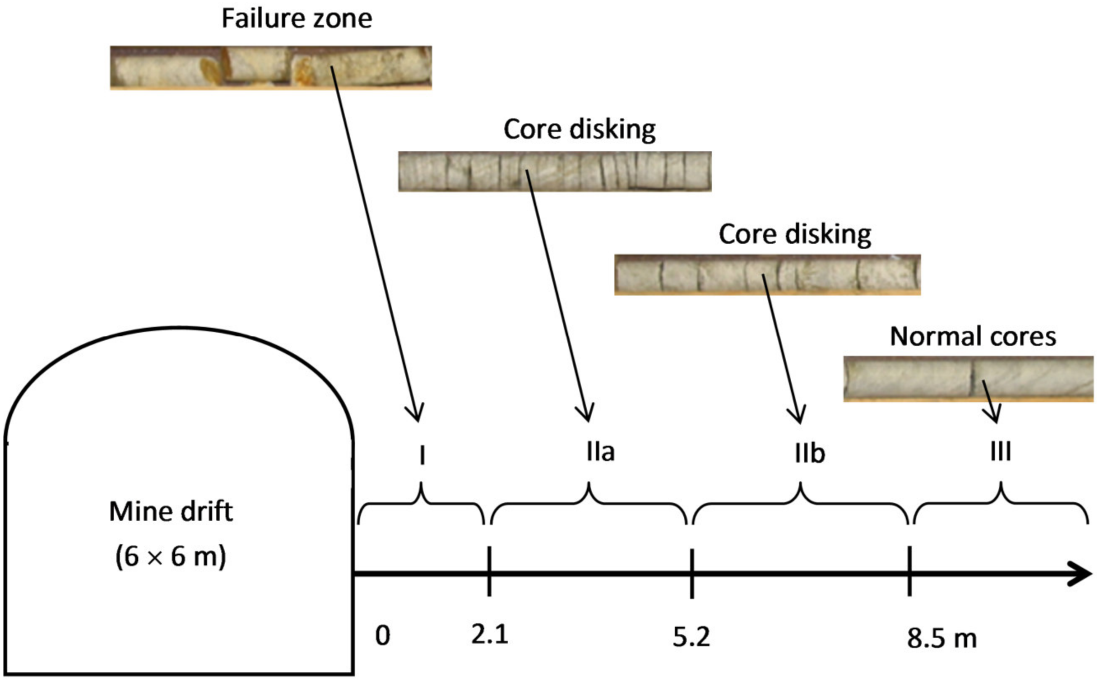

Figure 3. Cores drilled in the wall of a mine drift at a depth of 1000 m [1].

To illustrate the failure zone surrounding an underground opening, numerical modelling was conducted for a tunnel of 6 m in width and 6 m in height, excavated in a rock mass subjected to hydrostatic in situ stresses. The in situ stresses are assumed to be $\sigma_1 = \sigma_2 = \sigma_3 = 30$ MPa in the simulation and the rock mass obeys the Mohr-Coulomb failure criterion with c = 5 MPa and ϕ = 35 degrees. The constitutive model of the material is elastic and perfectly plastic, that is, the residual strength of the material is equal to the peak strength. Figure 4 shows the distribution of the major principal stresses that are oriented approximately parallel with the tunnel walls and roof, that is, in the tangential directions, after excavation. The immediate surrounding rock, approximately 2 m deep in the walls is failed. Beyond that depth, the rock is still intact but the tangential stress has been elevated somewhat, depending on the distance to the tunnel wall. It reaches its maximum at a depth of about 3 m and then gradually drops to the in situ stress level (30 MPa) when the distance is long enough. The rock portion within which the tangential stress is significantly elevated carries the majority of the ground pressure and forms a protection shield, that is, a pressure ring, around the circular opening.

Based on the core logging shown in Figure 3 and the numerical modelling shown in Figure 4, it can be deduced that a pressure arch (or ring) exists at a certain depth of the rock surrounding an underground opening, where the tangential stresses are significantly elevated. This is the so-called natural pressure arch, sketched in Figure 5. The concept of the natural pressure arch was used for rock support design by, among others, Wright [2], Krauland [3] and Li [4].

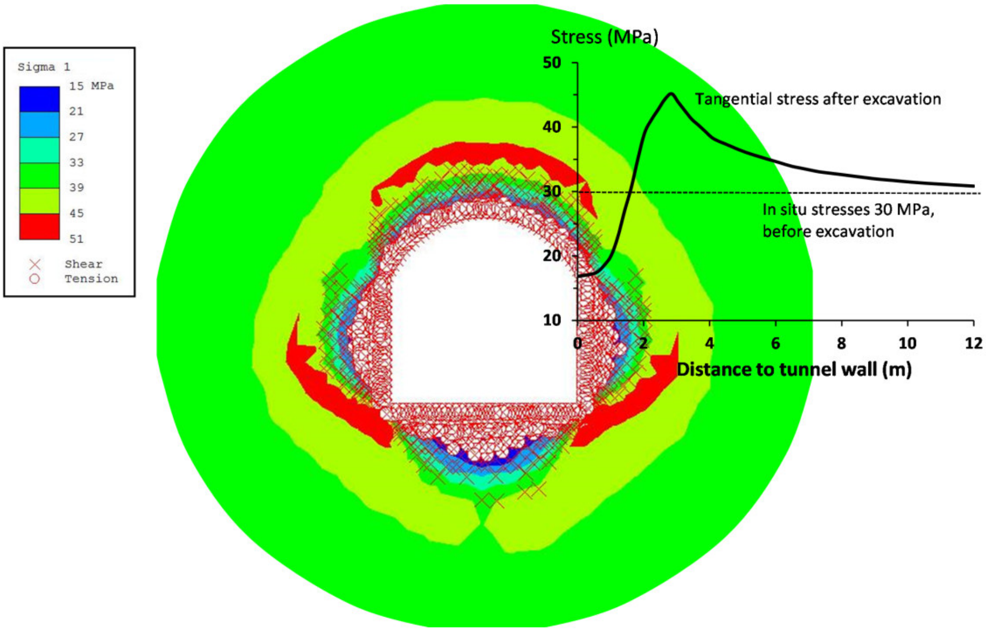

Figure 4. Distribution of the major principal stresses in the rock surrounding a tunnel. The crosses and circles mark the zone of rock failure.

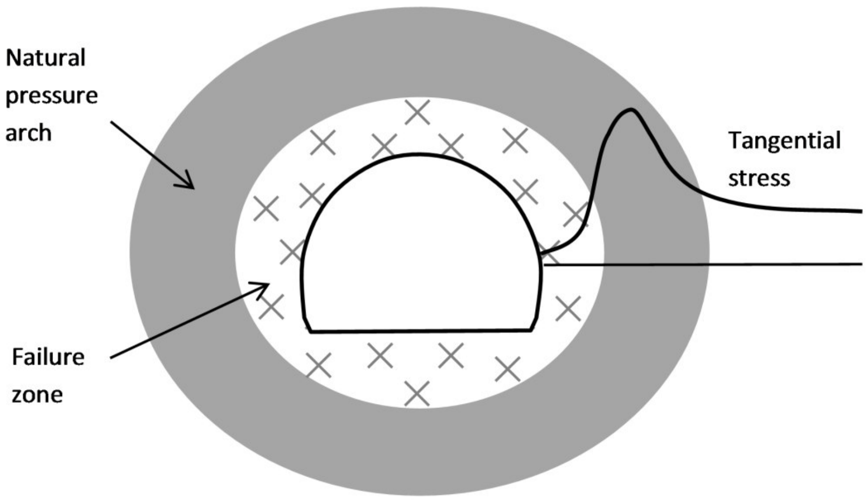

Figure 5. A Sketch illustrating the natural pressure arch surrounding an underground opening.

4. METHODOLOGIES

Rock support refers in general to any measure aiming to stabilise rock masses by using support elements. Support elements may be rockbolts, cables, meshes, straps, lacing, shotcrete (i.e. sprayed concrete), thin liners, steel sets, shotcrete arches and cast concrete lining. A support system provides three primary functions: reinforcement, holding and retention [5]. Reinforcement refers to strengthening of the rock mass; holding to the suspension of potentially loosened blocks; and retention to confinement of the exposed rock surfaces. Each support element may perform one or more of the three primary functions. Reinforcement is usually achieved by installing rockbolts systematically. The increase in the rock strength due to bolting is very limited. Assume that the load capacity of a rockbolt is 200 kN and rockbolts are installed with a pattern of 1 m × 1 m. The maximum confining pressure the rockbolts can provide is 0.2 MPa. The increase in the rock strength by this confining pressure, according to the Mohr-Coulomb criterion, may be in the range of 1 – 2 MPa, which is significantly lower than the inherent strength of the rock mass. The essential function of bolting is to keep the fractured rock together to form a pressure arch around the opened space. In other words, the bolts help the rock to strengthen and support itself. Rockbolts also provide a holding function to loosened blocks and fractured rock. In the case of a large failure zone, rockbolts may be entirely located within the failure zone. The use of long cablebolts is an option to provide an effective holding function. Retaining is mainly achieved by using shotcrete, mesh or other types of thin liners laid on the rock surface in mines. In civil tunnels, allowable rock deformation is much smaller than in mines. Therefore, heavy external support structures such as steel sets, concrete arches and even cast concrete lining are applied to restrain wall deformation. These structures are set up on rock surfaces, but they are similar to the cables installed within the rock mass in the sense that they provide a holding function. A rock support system may be composed of one, or more than one, of the following support layers, depending on the loading condition and the extent of rock failure [6]:

Layer 1 - Bolting: Rockbolts are installed sporadically or systematically.

Layer 2 - Surface retaining: Retaining elements like meshes, straps, lacing, thin liners, shotcrete and cast concrete lining are installed on the rock surface.

Layer 3 - Cable bolting: Single- or multi-strand cables are installed into the competent rock behind the failure zone.

Layer 4 - External support: Structure elements, including steel sets, concrete arches, invert, cast concrete lining and thick shotcrete liners, are set up in tunnels. External support elements may be classified to two classes 4A and 4B, the former referring to moderately strong support elements and the latter to strong and stiff support elements.

In shallow tunnels where the in situ rock stresses are low, rock failure in the surrounding rock is limited to a small depth (Figure 6A). The task of the rock support in this case is to prevent the loosened or failed rock blocks in the failure zone from falling. Support layer 1, that is, spot bolting or sparsely spaced pattern bolting, may be enough to stabilize the rock mass. The bolts should be installed into the natural pressure arch behind the failure zone.

In poor rocks, the failure zone may extend to a depth beyond the length of rock bolts (Figure 6B). In this case, both rockbolts (layer 1) and surface retaining elements (layer 2) are needed. Rockbolts must be tightly installed so that they, together with the surface support elements, help to establish an artificial pressure arch in the failure zone. The artificial pressure arch forms a protection shield over the opening. Either cablebolts (layer 3) or moderately strong external support elements (layer 4) or both are added to restrict the movement of the protection shield. The 1-2-3-layer support system is often used in deep mines and underground caverns of large span, such as underground machine halls in hydropower plants. This support system is characterised by its flexibility in adapting to the rock condition. In civil tunnels, cable bolting is less used than in mines. Instead, a 1-2-4A-layer support system is more than often preferred. External support elements (layer 4A) for this purpose are often steel sets and concrete lining.

In extremely poor rock, the failure zone may become so vast that cablebolts cannot reach the natural pressure arch behind the failure zone. In the worst case, such as in a time-dependent squeezing rock mass, it takes a long time for the failure zone to stop extension. In this case, use of strong and stiff external support elements (layer 4B) is more effective than use of cablebolts to restrict the dilation in the failure zone (Figure 6C). Dilation would slow down under the strong resistance of the external support elements and the extension of the failure zone would finally stop so that the surrounding rock mass becomes stabilized.

Pattern bolting plays a crucial role in a support system. Tightly spaced bolts constrain the failed rock so that an artificial pressure arch is established in the failure zone. The load-bearing capacity of an artificial pressure arch was visually demonstrated by Lang [7]in the 1960s and also recently by Hoek [8]. Li [4]reported an example of applying the concept of an artificial pressure arch for rock support design.

5. FACTOR OF SAFETY

5.1. Factor of safety for gravitational rock falls

Wedge blocks in the roof may become loosened in shallow tunnels where in situ rock stresses are low. The loosened blocks tend to fall under gravity. The load exerted on the support system is equal to the deadweight of the falling blocks. In this load-controlled condition, the factor of safety (FS) of the rock support is defined as

$$\mathrm{FS} = \frac{\text{Load capacity of the support system}}{\text{Total load on the support system}} \tag{1}$$

In this case, a safe rock support requires that the load on the support system is less than the strength of the loaded portion of the support system, that is, FS > 1.

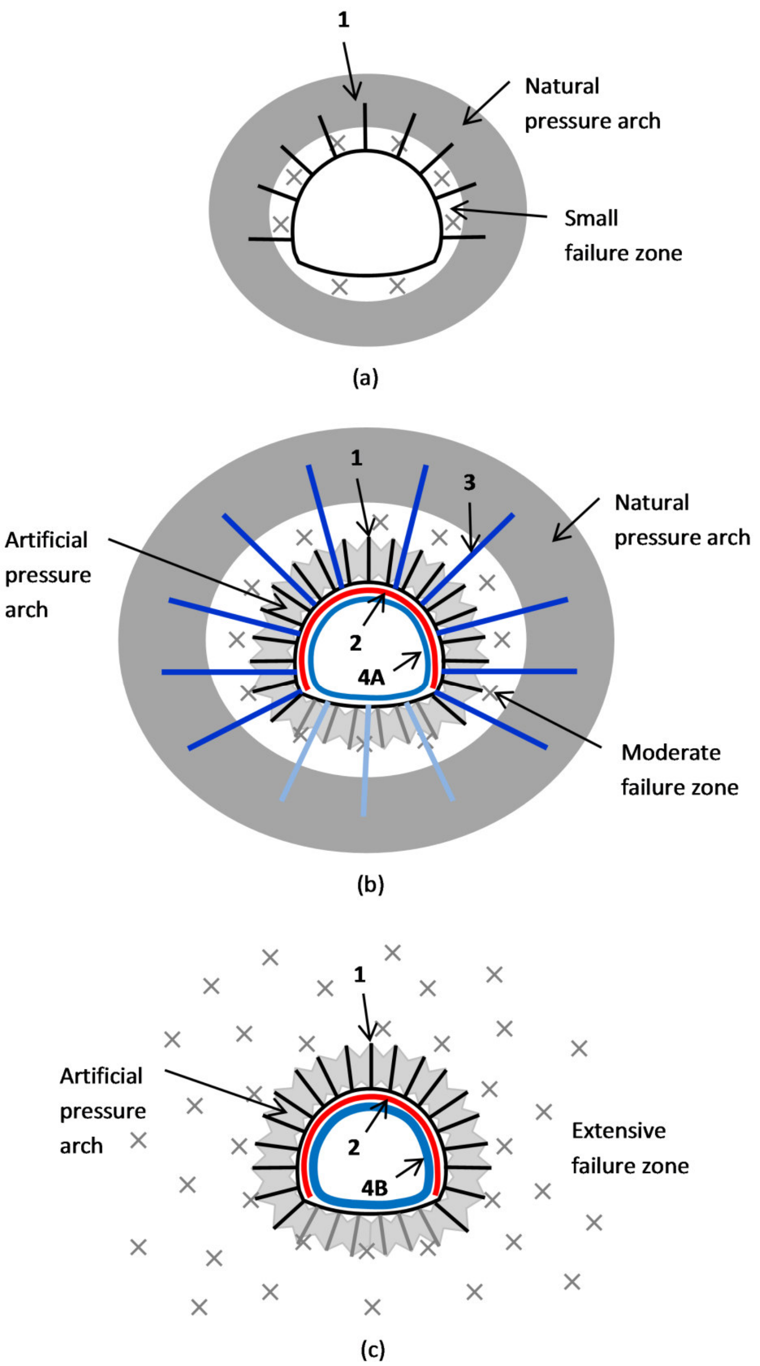

Figure 6. Methodologies of ground support in different degrees of rock failure. The failure zone is (a) small, (b) moderate and (c) extensive.

5.2. Factor of safety in squeezing rock

Rock deformation can be significantly large in tunnels excavated in highly stressed soft and weak rock because of vast rock failure. The essential driving power for the rock deformation is the strain energy released from the rock mass after excavation. A good portion of the released strain energy is dissipated for rock fracturing in weak rocks, which in turn brings about rock deformation. In extremely poor rock conditions, the large rock deformation may lead to rock collapse. The response of the rock mass to excavation is described by the Ground Response Curve (GRC), as sketched in Figure 7. Yield support systems work better than stiff ones in squeezing rock. The support elements of the system tend to retard the rock deformation by providing a support load, but at the same time deform together with the rock mass when the support load is beyond a limit. The rock mass finally becomes stable after a certain amount of displacement. The dashed line in Figure 7 represents the GRC of the rock mass after reinforcement by a yield support system. The load on the rock support is not constant under the squeezing condition because the support load and the displacement are correlated. It is thus not possible to use Eq. (1) to find out the factor of safety. In squeezing rock, it is more relevant to define the factor of safety by displacements rather than load and strength. It is required, from the point of view of stability, that the displacement of the tunnel wall at equilibrium, u_{eq} , has to be smaller than the critical displacement, u_c , beyond which uncontrollable rock collapse would occur, that is,

$$\frac{u_c}{u_{eq}} > 1 \tag{2}$$

The critical displacement uc is indeed difficult to be quantified. In engineering practice, there usually exists a maximum allowable displacement, denoted as umax, from the point of view of operation. For example, the radial displacement of a TBM tunnel usually is not allowed to be larger than 150 mm in order to avoid clogging of the TBM cutter head. In other words, the ratio of the umax to the displacement at equilibrium, u_{eq} , must be larger than 1, that is,

$$\frac{u_{\max}}{u_{eq}} > 1 \tag{3}$$

Furthermore, it is required that the displacement u_{eq} must be smaller than the ultimate displacement u_{ult} of the support system in order to avoid failure of the support system, that is,

$$\frac{u_{ult}}{u_{eq}} > 1 \tag{4}$$

To the end, the value of the factor of safety in squeezing rock is the minimum one among the above three ratios, which is required to be larger than 1, that is,

$$FS = \min\left(\frac{u_c}{u_{eq}}, \frac{u_{\max}}{u_{eq}}, \frac{u_{ult}}{u_{eq}}\right) > 1 \tag{5}$$

5.3. Factor of safety in burs-prone rock

In highly stressed rock masses, a portion of the strain energy stored in the rock mass may be released suddenly, leading to a rockburst event. Use of energy-absorbing support elements, such as strong yieldable rockbolts, is an effective means to stabilise burst-prone rock masses. The support principle in this case is that the energy absorption capacity of the support elements must be higher than the kinetic energy of the ejected rock blocks. Let E_{ab} represent the total energy absorption of the support elements and E_{ej} is the kinetic energy of the ejected rock, which is expressed by:

$$E_{ej} = \frac{1}{2} m V^2 \tag{6}$$

wherem is the mass of the ejected rock and V is the ejection velocity. The ratio of E_{ab} to E_{ej} has to be larger than 1 in order to avoid rock ejection, that is,

$$\frac{E_{ab}}{E_{ej}} > 1 \tag{7}$$

With a competent support system, the ejected rock will stop moving after a displacement u_{eq} (Figure 8). The displacement u_{eq} has to be smaller than the maximum allowable displacement, $u_{\max}$, in order to avoid operational difficulties, and it also should not be larger than the ultimate displacement of the support elements, that is, ($u_{\max}/u_{eq}$) > 1 and (u_{ult}/u_{eq}) > 1. The value of the factor of safety in burst-prone rock is the minimum one among the above three ratios, which is required to be larger than 1, that is,

$$FS = \min\left(\frac{E_{ab}}{E_{ej}}, \frac{u_{\max}}{u_{eq}}, \frac{u_{ult}}{u_{eq}}\right) > 1 \tag{8}$$

The factor of safety for burst-prone rock is determined either by the energy ratio, or one of the displacement ratios.

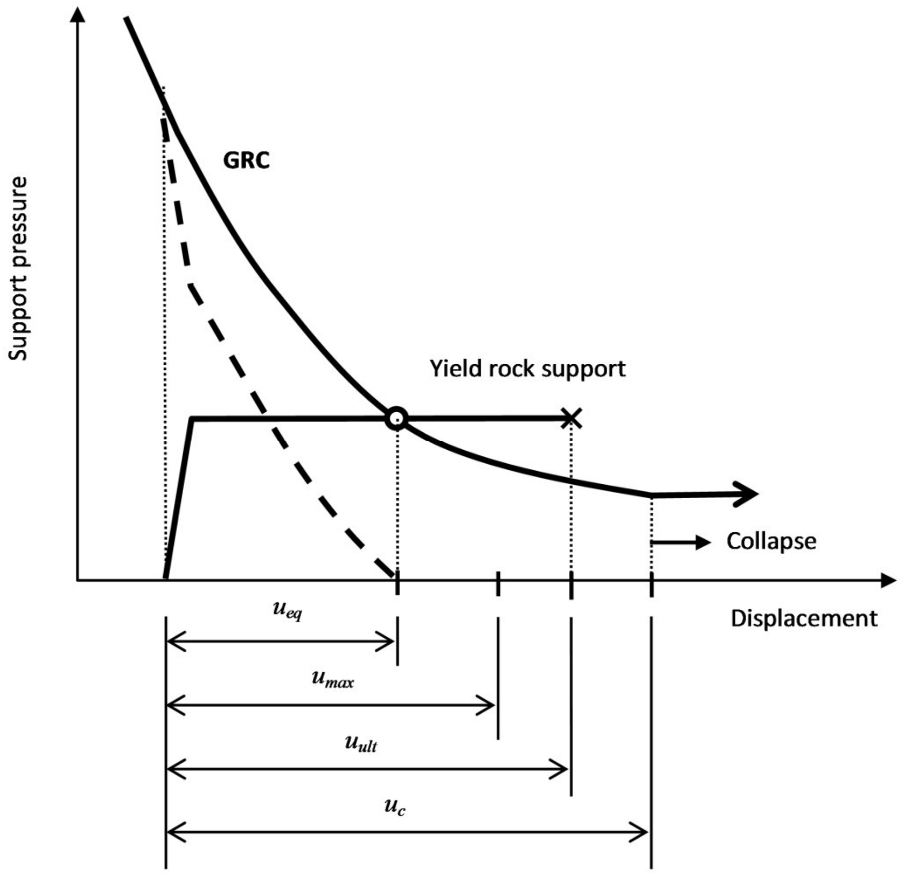

Figure 7. The Ground Response Curve (GRC) and the support characteristic line of yield rockbolts.

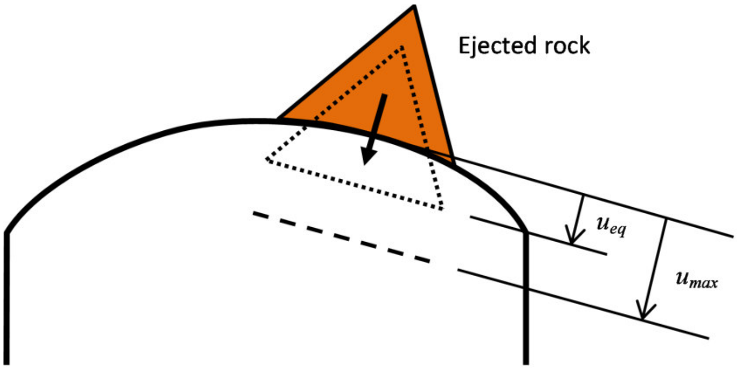

Figure 8. The equilibrium displacement u_{eq} and the maximum allowable displacement $u_{\max}$ related to a rockburst event.

6. COMPATIBILITY BETWEEN SUPPORT ELEMENTS

The current methodology of rock support in civil tunnels is to install fully encapsulated stiff rockbolts in the rock and to apply shotcrete or cast-in concrete lining on the rock surface. Yield support elements may be imbedded in the lining in squeezing rock conditions [6,9]. Rock support systems in civil tunnels are in principle composed of stiff internal elements (fully encapsulated rebar bolts) and yield external elements (deformation-compensated concrete lining), which are conceptually sketched in Figure 9a. In such a support system, the stiff internal elements (rockbolts) may fail after a small deformation, but the external elements (the concrete lining) can accommodate relatively large rock deformation because of the embedded yield elements. The internal and external elements in the system are thus not compatible in deformation. In underground mining, people are used to employing yield rockbolts and meshes to deal with excessive rock deformation. The support load is mainly carried by the rockbolts and the mesh is mainly to restrain the dilation of the rock surface. Figure 9b is a conceptual sketch of this type of support system. In such a support system, it seems that the internal elements (rockbolts) and the external elements (meshes) are compatible in deformation, but the load-bearing capacity of the meshes is very low in the system.

In a satisfactory rock support system, both internal and external elements should be both strong and deformable. In other words, they should be compatible both in load and deformation capacities in order to achieve the optimum reinforcement effect. The behaviour of the internal and external support elements in such a system should be as sketched in Figure 10.

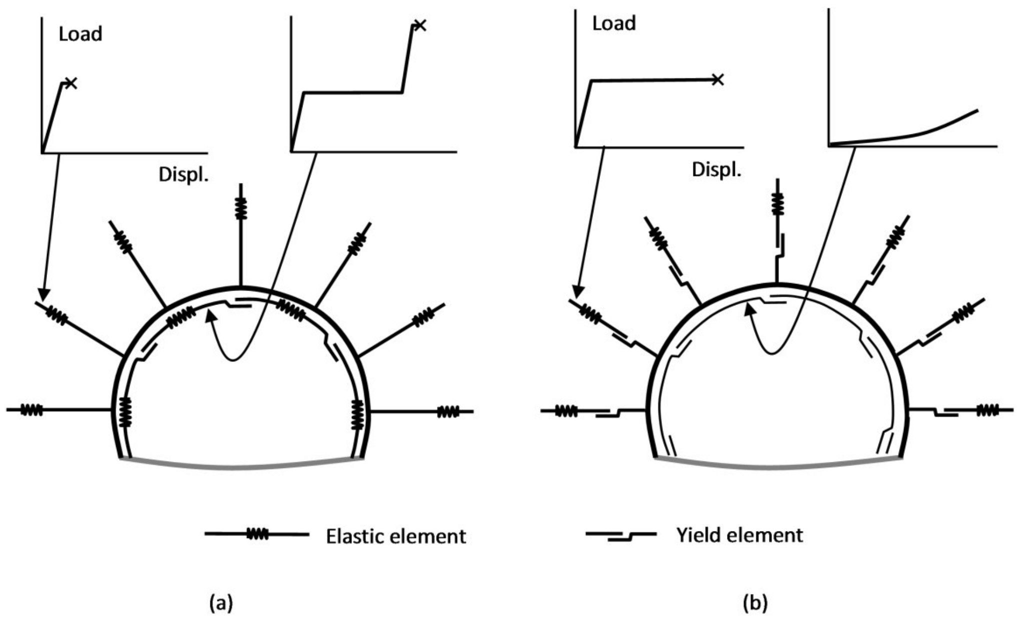

Figure 9. Sketches illustrating incompatible rock support systems: (a) in civil tunnelling and (b) in mining.

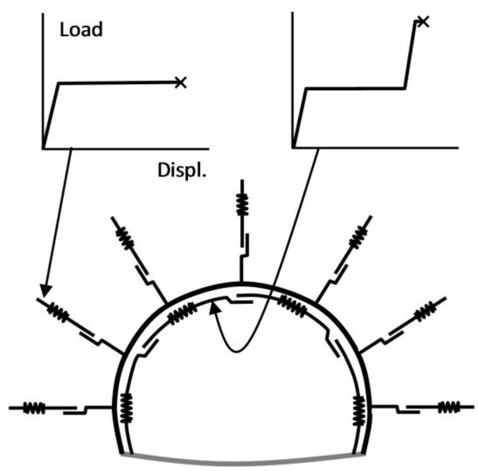

Figure 10. A sketch illustrating the concept of a compatible rock support system.

ISSN 2303-8403 Broj 3, 2017

7. CONCLUDING REMARKS

There exists a natural pressure arch immediately outside of the failure zone in the rock surrounding an underground opening. The rock in the pressure arch is not failed but the tangential stresses in the rock are elevated. The pressure arch forms a protection shield over the underground opening. The major goal of the ground support is to secure the failed or damaged rocks under the natural pressure arch. In the case of a small failure zone around the opening, the rockbolts should be long enough to reach the pressure arch. In the case of a vast failure zone, an artificial pressure arch should be established by rockbolts systematically installed in the failure zone. Cables and strong external support elements are used to secure the artificial pressure arch.

The ground support design can be based on the factor of safety defined by the strength of the support system and the load of falling blocks in low stress rock masses. In squeezing rock, the design has to take into the displacements of the rock mass and the support elements instead of load and strength. In burst-prone rock, the kinetic energy of the ejected rock and the energy absorption of the support elements also need to be taken into account.

The support elements in a support system should be compatible each other in terms of displacement and energy absorption capacities.

8. REFERENCES

[1] Li CC: Evaluation of the state of stress in the vicinity of a mine drift through core logging. 4th Asian Rock Mechanics Symposium, 2006, Singapore. World Scientific, New Jersey. 8p.

[2] Wright FD: Roof control through beam action and arching. In: SME mining engineering handbook, 1973, vol. 1, Ch. 13. New York: Society of Mining Engineers; 80–96.

[3] Krauland N: Rockbolting and economy. In: Rockbolting—theory and applications in mining and underground construction, 1983, O. Stephansson (eds). Rotterdam: Balkema. 499–507.

[4] Li CC: Rock support design based on the concept of pressure arch. Int. J. Rock Mech. Min. Sci., 2006, 43(7): 1083-1090.

[5] Kaiser PK, Tannant DD, McCreath DR: Canadian Rock Burst Support Handbook, 1996. Geomechnaics Research Center, Sudbury, Ontario. 385p.

[6] Li CC: Design principles of rock support for underground excavations. EUROCK 2012, Stockholm, Sweden. 18p.

[7] Lang TA: Theory and practice of rockbolting. Trans. of Amer. Inst. of Mining, Metall. and Petroleum Engineers, 1961, 220: 333-348.

[8] Hoek E: Model to demonstrate how bolts work. In: Practical Rock Engineering. https://www.rocscience.com, 2007.

[9] Schubert W: Recent experience with squeezing rock in Alpine tunnels. In: CUC – Rock Support in Medium to Poor Rock Conditions, 2001, C. Allan (eds), Proc. Int. Symp. Sargans.