Journal of Rock Mechanics and Geotechnical Engineering 9 (2017) 396–414

Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full Length Article

Principles of rockbolting design

Charlie C. Li

Department of Geoscience and Petroleum, The Norwegian University of Science and Technology, Norway

A R T I C L E I N F O

Article history:
Received 17 January 2017
Received in revised form
3 April 2017
Accepted 4 April 2017
Available online 3 May 2017

Keywords:
Rockbolting design
Pressure arch
Bolt length
Bolt spacing
Factor of safety

A B S T R A C T

This article introduces the principles of underground rockbolting design. The items discussed include underground loading conditions, natural pressure zone around an underground opening, design methodologies, selection of rockbolt types, determination of bolt length and spacing, factor of safety, and compatibility between support elements. Different types of rockbolting used in engineering practise are also presented. The traditional principle of selecting strong rockbolts is valid only in conditions of low in situ stresses in the rock mass. Energy-absorbing rockbolts are preferred in the case of high in situ stresses. A natural pressure arch is formed in the rock at a certain distance behind the tunnel wall. Rockbolts should be long enough to reach the natural pressure arch when the failure zone is small. The bolt length should be at least 1 m beyond the failure zone. In the case of a vast failure zone, tightly spaced short rockbolts are installed to establish an artificial pressure arch within the failure zone and long cables are anchored on the natural pressure arch. In this case, the rockbolts are usually less than 3 m long in mine drifts, but can be up to 7 m in large-scale rock caverns. Bolt spacing is more important than bolt length in the case of establishing an artificial pressure arch. In addition to the factor of safety, the maximum allowable displacement in the tunnel and the ultimate displacement capacity of rockbolts must be also taken into account in the design. Finally, rockbolts should be compatible with other support elements in the same support system in terms of displacement and energy absorption capacities.

1. Introduction

Rockbolt is the most widely used support element in support systems in underground mines and civil tunnels. Rockbolting design is indeed mainly based on experience and it appears that rockbolting design is simply a business of selection of rockbolt types and the determination of bolt length and spacing, but, one essentially uses, either explicitly or implicitly, a methodology in a specific rockbolting design. Attempts are made in this article to summarise the design principles and methodologies hidden in rockbolting practise, which include the relationship between the in situ stress state and rockbolt types, the concept of pressure arch, design methodologies, determination of bolt length and spacing, factor of safety, compatibility between support elements and different types of rockbolts.

2. Underground loading conditions

2.1. Low in situ stress conditions

Rock blocks in the roof of an underground opening are prevented to fall as long as a high enough horizontal stress exists in the rock mass. However, they would fall under gravity in low in situ stress conditions. In locations close to the ground surface, the rock mass often contains well-developed rock joint sets. The rock joints sometimes are open, which is an indication that the in situ rock stresses are low in the rock mass. The task of rock support in low stress rock masses is to prevent rock 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 (Fig. 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 force of the block (i.e. the load), is appropriate for rock support design in a load-controlled condition.

E-mail address: Charlie.c.li@ntnu.no.

Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

http://dx.doi.org/10.1016/j.jrmge.2017.04.002

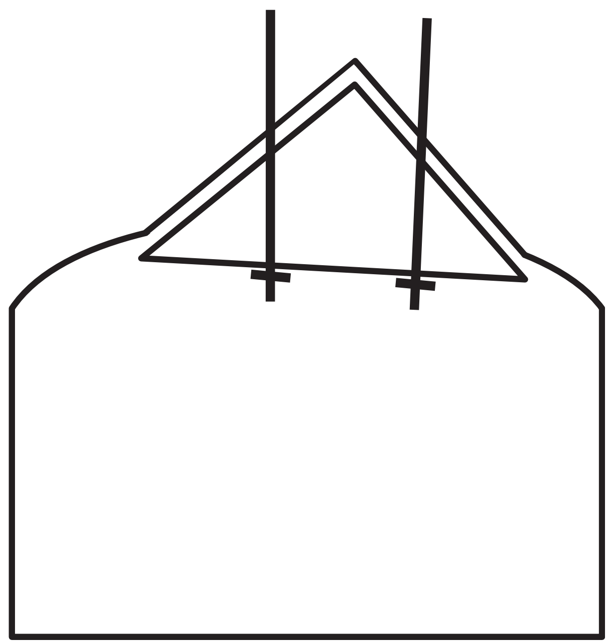

Fig. 1. Gravitational load on rockbolts in low stress condition.

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, i.e. the strength-to-load ratio that is called the factor of safety, should be larger than 1. This principle is valid for ground constructions where the total load on the construction structures 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 force of loosened rock blocks.

2.2. High in situ stress conditions

The author observed in a deep metal mine that the number of geological discontinuities in the rock mass became less and the discontinuities were less opened in depth. For instance, at a depth of 1000 m, it was observed that all of the few discontinuities exposed on an excavation face were completely closed. Therefore, it can be said that the rock mass quality is improved at 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 fall 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 (Fig. 2). It was observed in some metal mines in Sweden that strain burst usually occurred below a depth of 600 m and became 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 force of loosened blocks but to prevent the failed rock from disintegration. In high stress rock masses, the support system must be not only strong but also deformable in order to deal with either stress-induced rock squeezing in soft and weak rock or rockburst in hard and strong rock.

2.3. Suitable rockbolt types

The suitable types of rockbolts for a given rock mass are associated with the loading condition in the rock mass. In the case of a load-controlled condition as shown in Fig. 1, the strength of the rockbolts is the most important parameter for the selection of rockbolt type. The basic requirement is that the strength of the rockbolts must be higher than the load on the bolts. The appropriate types of rockbolts under load-controlled conditions are fully encapsulated rebar bolts, threadbar bolts and cablebolts.

In overstressed weak and soft rock, the excessive deformation needs to be accommodated. The traditional approach to deal with rock squeezing is to use ductile rockbolts in conjunction with other types of ductile surface retaining elements such as mesh. Split set is the typical rockbolt for this purpose in the mining industry. Split set can displace significantly, but it cannot much restrain the rock deformation because of its low load-bearing capacity. Its main function is to avoid disintegration of the fractured rock mass. An active measure to stabilise squeezing rock is to provide a high support resistance to restrain the rock deformation on the one hand, while the support elements in the support system must be deformable on the other hand. Use of energy-absorbing rockbolts can achieve this goal.

Rockburst is an instability issue in overstressed hard and strong rock. The goal of rock support in such conditions is to absorb the kinetic energy of the ejected rock. Energy-absorbing rockbolts should be used in burst-prone rock masses. The higher the load-bearing capacity of the energy-absorbing rockbolt is, the less the ejected rock displaces.

3. Design principles

3.1. 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 mine drift was parallel with the strike of the tabular ore body and the boreholes were drilled in the wall of the drift on the side of the ore body that was approximately 150 m apart from the drift. The fracture logging on the cores provided information on the distribution of the secondary stresses in the rock surrounding the drift. Fig. 3 shows the fracture patterns in the cores taken from a horizontal borehole. The fracture intensity in the cores varies along the borehole. The cores are small pieces with a low value of rock quality designation (RQD) 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 core drilling. Zone II can be further divided into two subzones. 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 8.5 m to the end of the borehole at the depth of approximately 180 m. 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 was the failure zone, where the rock failed either in shear or in tension and the tangential stress was partially reduced, while the tangential stress in Zone II was elevated but the rock had not yet fractured after excavation of the drift. Zone II was the position of the natural pressure arch that carried the ground pressure and functioned as a protection shield over the drift.

To illustrate the failure zone surrounding an underground opening, numerical modelling was conducted for a horseshoe-shaped 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

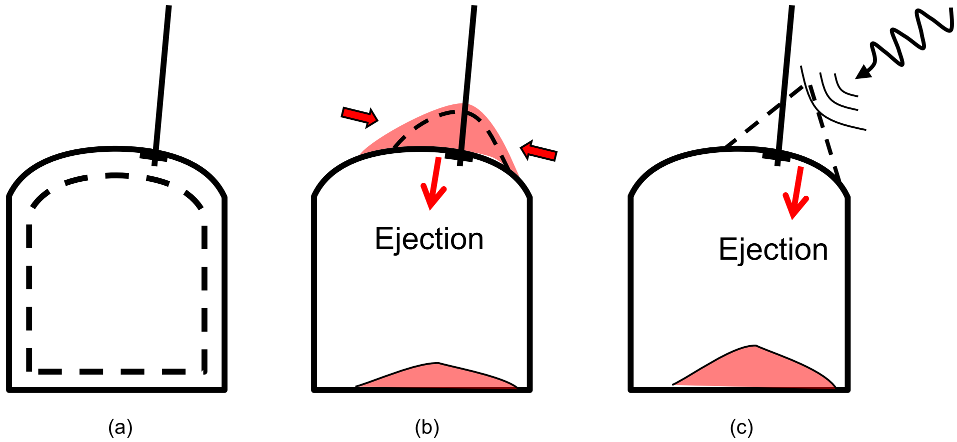

Fig. 2. Loading conditions to rockbolts in high stress rock masses. (a) Rock squeezing, (b) strain burst, and (c) fault-slip burst.

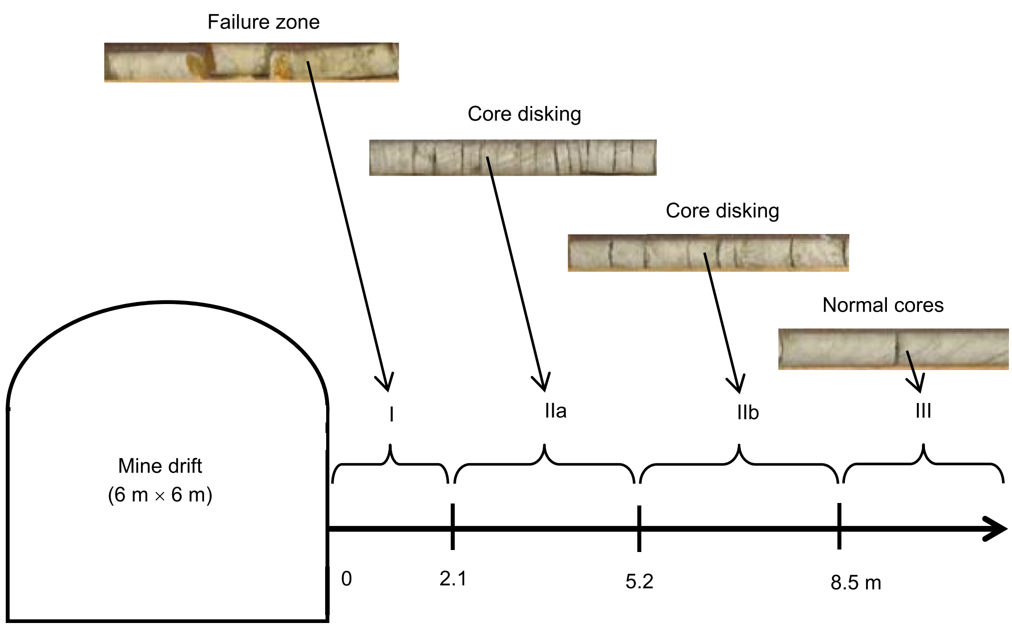

Fig. 3. Cores drilled in the wall of a mine drift at a depth of 1000 m (Li, 2006a).

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 cohesion $c = 5$ MPa and internal friction angle $\phi = 35°$. The constitutive model of the material is elastic and perfectly plastic, i.e. the residual strength of the material is equal to the peak strength. Fig. 4 shows the distribution of the major principal stresses that are oriented approximately parallel with the tunnel walls and roof, i.e. in the tangential directions, after excavation. The immediate surrounding rock, approximately 2 m deep in the walls, fails. 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) in locations far away from the tunnel. The rock portion within which the tangential stress is significantly elevated carries the majority of the ground pressure and forms a protection shield, i.e. a pressure arch, around the tunnel.

Based on the core logging shown in Fig. 3 and the numerical modelling shown in Fig. 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 Fig. 5. The concept of the natural pressure arch was used for rock support design by, among others, Wright (1973), Krauland (1983) and Li (2006b).

3.2. Design methodology

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 (Kaiser et al., 1996). 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.

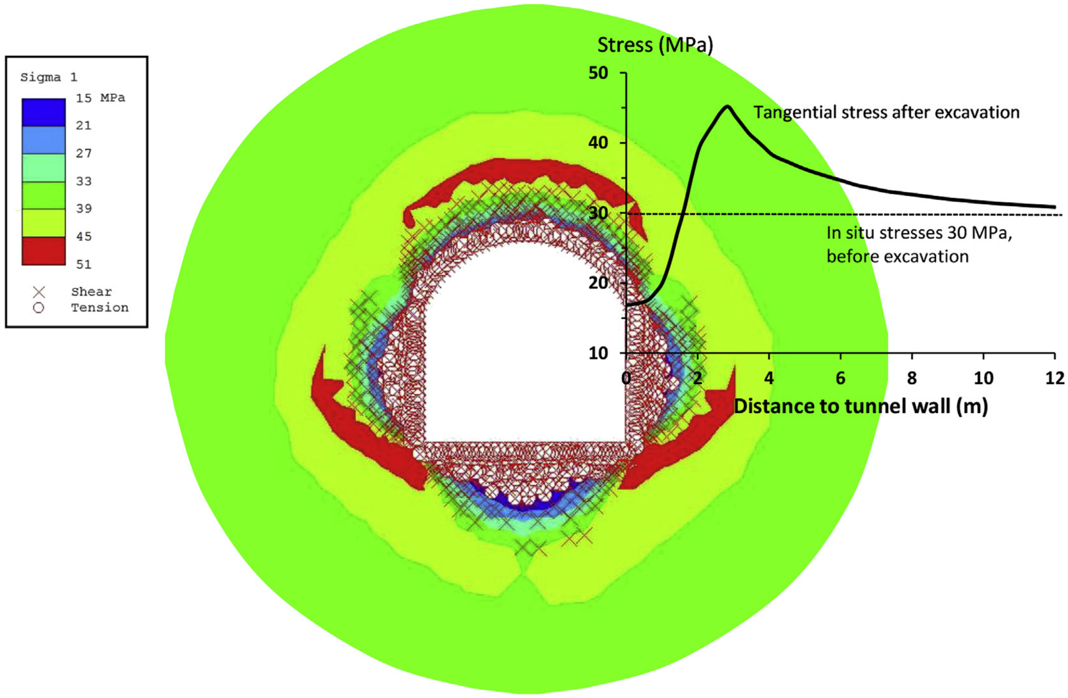

Fig. 4. Distribution of the major principal stresses in the rock surrounding a tunnel. The crosses and circles mark the zone of rock failure.

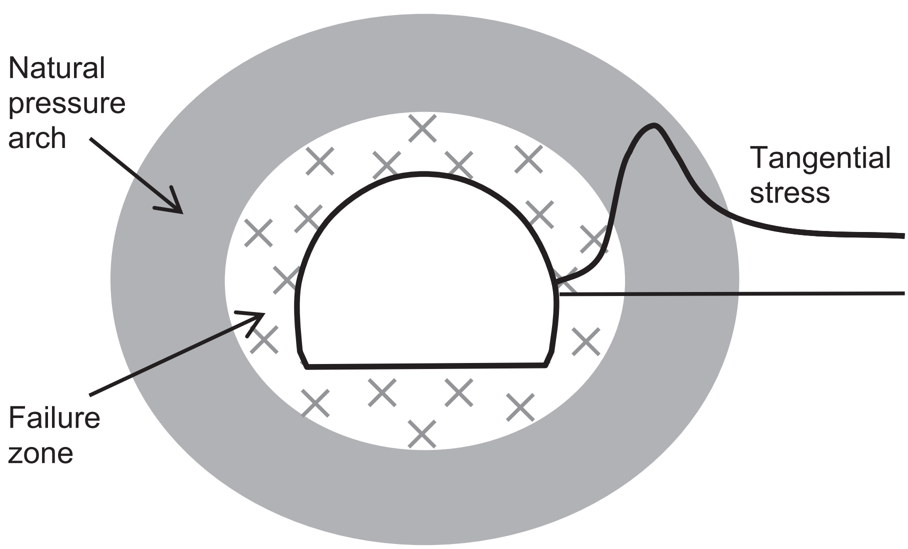

Fig. 5. A sketch illustrating the natural pressure arch surrounding an underground opening.

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 that 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. Surface retaining is mainly achieved by shotcrete, mesh or other types of thin liners laid on the rock surface in mines. In civil tunnels, the allowable rock deformation is much smaller than that in mines. Therefore, heavy external support structures such as steel sets, concrete arches and even cast concrete lining are applied to restrain wall deformation. Those structures are set up on rock surfaces, but they are similar to 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 (Li, 2012):

(1) Layer 1 – Bolting: Rockbolts are installed sporadically or systematically.

(2) Layer 2 – Surface retaining: Retaining elements like meshes, straps, lacing, thin liners, shotcrete and cast concrete lining are installed on the rock surface.
(3) Layer 3 – Cable bolting: Single- or multi-strand cables are installed into the competent rock behind the failure zone.
(4) Layer 4 – External support: Structure elements, including steel sets, concrete arches, invert, cast concrete lining and thick shotcrete liners, are set up in tunnels.

In shallow tunnels, or in the case of a small failure zone, the task of the rock support is to hold the loosened or failed rock blocks. Support layer 1, i.e. spot bolting or sparsely spaced pattern bolting, may be good enough to stabilise the rock mass (Fig. 6a). The bolts should be installed into the natural pressure arch behind the failure zone. In poor rocks, the failure zone may be so extensive that rockbolts cannot reach the natural pressure arch. A support system composed of layers 1 and 2 is then needed (Fig. 6b). Layer 1, in this case, must be tightly spaced pattern bolting, which builds up an artificial pressure arch in the failure zone. The artificial pressure arch forms a protection shield over the opening. In extremely poor rock, support layer 3, cable bolting, needs to be added to the system. 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 order to adapt to the prevailing rock condition. In civil tunnels, cable bolting is less used than in mining excavations. Instead, external support structures (layer 4), such as steel sets and concrete lining, are employed to provide the holding function. A 1-2-4-layer support system is preferred in civil tunnels.

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 (1961) in the 1960s and also recently by Hoek (2007). Li (2006b) reported an example of applying the concept of an artificial pressure arch for rock support design.

3.2.1. Australian methodology

In squeezing rock, the methodology of rockbolting in Australian mines is to reinforce the failure zone of the rock using tightly spaced ductile rockbolts “Split sets” or point-anchored resin bolts of 2.4–3 m in length. The bolt-reinforced zone is then nailed to the competent strata behind the failure zone with cables (see Fig. 7). The tightly spaced rockbolts help the fractured rock to build up an artificial load-bearing arch and the cablebolts integrate the arch with the deeply located stable strata. The rock surface is retained with mesh, straps and mesh shotcrete. The Agnew gold mine is located in Western Australia. Rock squeezing is the major instability problem in the mine below the depth of 500 m because of the weak footwall rocks, ultramafic conglomerates, chlorite and talc. The wall–wall convergence of a 5 m × 5 m mine drive reached 400 mm in 5 months. The rockbolts used in the mine were either Split sets or point-anchored resin Posimix bolts, 2.4–3 m in length and 1 m in spacing. The twin-strand cablebolts used were 6 m in length and 2 m in spacing.

In burst-prone rock, the support system is composed of 2.4-m or 3-m long energy-absorbing rockbolts and meshes or fibre/mesh-reinforced shotcrete. It is claimed that such a system can sustain an ejection velocity of 5 m/s and absorb a dynamic energy of 35 kJ/m^2 (Slade and Ascott, 2007).

3.2.2. Canadian and Scandinavian methodology

The methodology of rockbolting in Canada and Scandinavian countries is to integrate the failed rock with short bolts in conjunction with meshes or/and fibre/mesh-reinforced shotcrete (Fig. 8).

In Canadian metal mines, the types of rockbolts are 2.4-m or 2.1-m long fully resin-encapsulated rebar and Split sets. Energy-absorbing rockbolts are added in case of seismic rock conditions. The Craig mine, in the region of Sudbury, Ontario, is characterised by the faults going through the ore body, which creates a number of fault-slip seismic events in the mining operation areas below the depth of 1600 m. In 5-m span drives, the 1.8–2.4 m long fully resin-encapsulated rebar bolts were used together with surface retaining support elements (meshes, straps, shotcrete, etc.). In burst-prone areas, rebar bolts and modified cone bolts were installed plus 6-m long cables.

In Swedish mines, fully encapsulated rebar bolts with cementitious grout are most often used. Split sets are seldom used in Scandinavian mines, but energy-absorbing rockbolts, such as D-

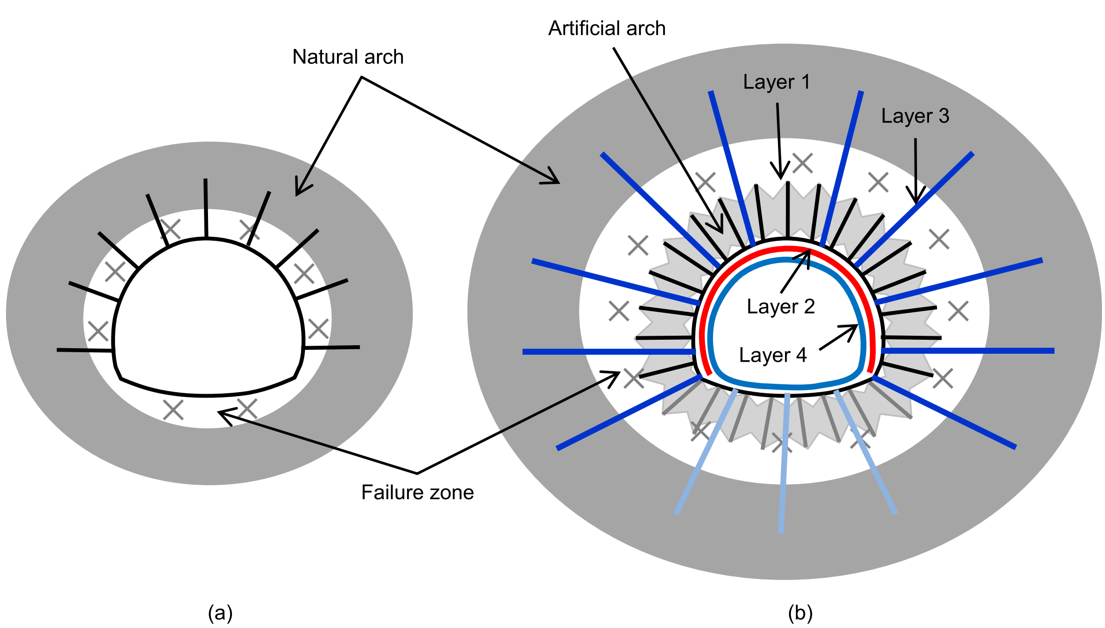

Fig. 6. Principles of rockbolting in different rock conditions of rock failure: (a) for a limited failure zone, and (b) for a vast failure zone.

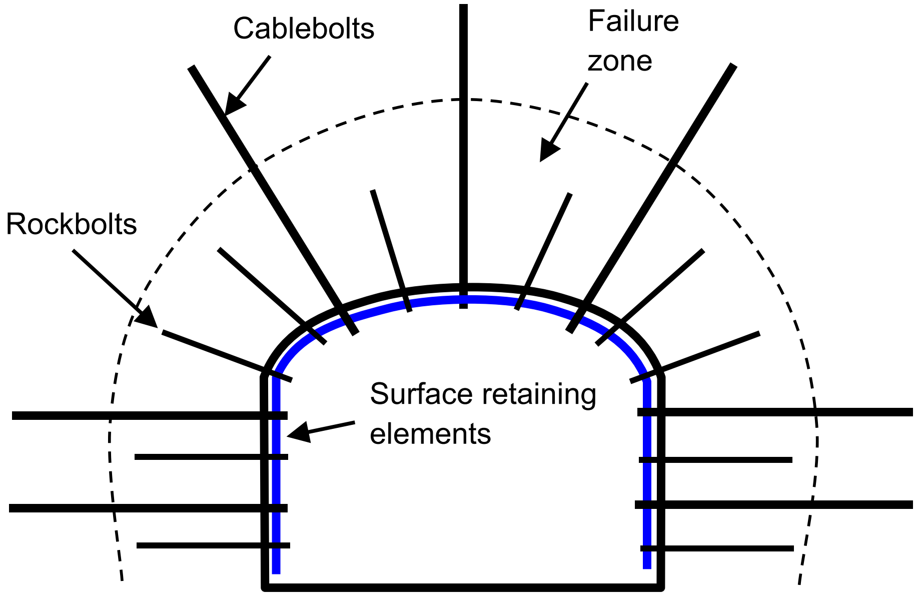

Fig. 7. The Australian methodology of rockbolting in deep metal mines.

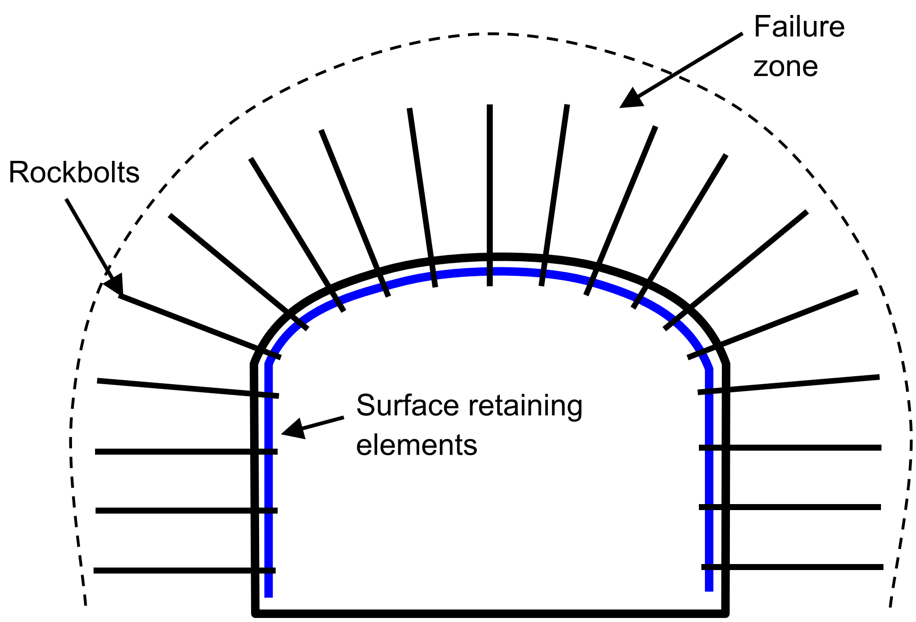

Fig. 8. The Canadian and Scandinavian methodology of rockbolting in deep metal mines.

bolts, have been used for dynamic rock support, for instance, in the iron ore mines in Sweden. The surface retaining liners are usually steel-fibre-reinforced shotcrete, but in burst-prone conditions, wire meshes are laid on the top of the steel-fibre-reinforced shotcrete. The bolt length is typically 2.7–3 m and the bolt spacing is 1–1.5 m. The Kristineberg metal mine in Sweden operates the mining activity in depths below 1000 m at present. The mine is subjected to rock squeezing in zinc-and-lead ore bodies owing to the chlorite/talk-rich rocks, but was subjected to strain burst in the hard quartzite of the gold ore bodies. The rockbolts used in the mine have been 2.7-m long fully cement-grouted rebars with a bolting pattern of 1.2 m × 1.2 m.

3.2.3. *South African methodology*

In burst-prone deep mines in South Africa, the methodology of rockbolting is to dissipate the released kinetic energy with energy-absorbing rockbolts and surface retaining elements. It is thought that the kinetic energy in a rockburst event is partially absorbed by the rockbolts and partially dissipated by fragmentation of the rock contained by the surface retaining elements. Mesh and lacing are often used in South African dynamic support systems.

In South Africa, mine drifts excavated in high stress rock are typically supported with energy-absorbing rockbolts (e.g. cone bolts and Durabar) and ductile bolts (e.g. split set and cables). The primary bolting method is a ring of 1.2-m long bolts and the secondary bolting is a ring of 2.4-m long bolts plus lacing, meshing and 50 mm thick steel or polyester fibre shotcrete (Fig. 9). The bolting pattern is typically 1 m × 1 m. The Mponeng gold mine in South Africa operates its mining activity at depths below 2500 m. The dominant rock is quartzite that is burst-prone under high ground pressure. Rockbolt is only one of a number of ground support elements used in the mine. Split sets, 1.2 m long, are used

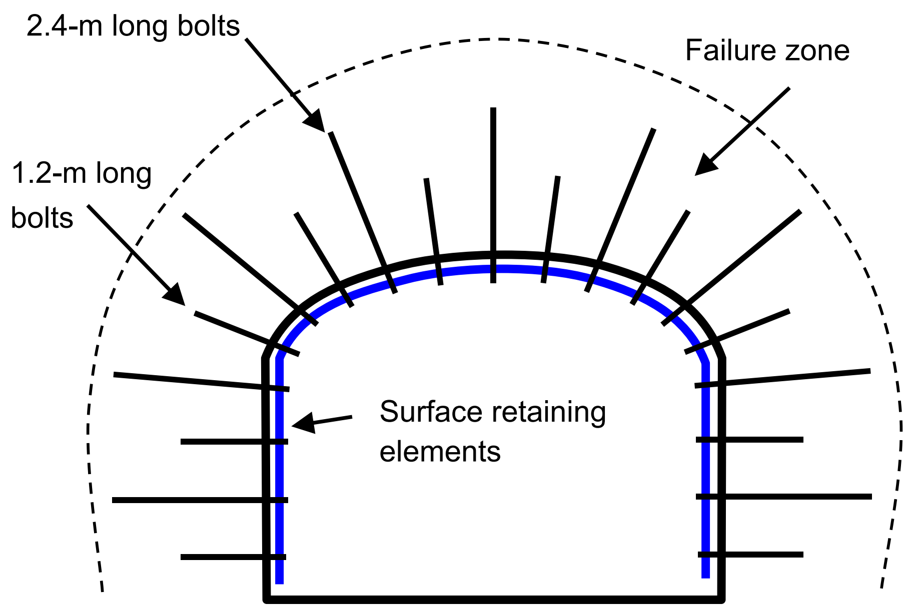

Fig. 9. The South African methodology of rockbolting in deep metal mines.

in mining stopes and Durabars, 2.4–3.6 m long, are used in transport drives.

3.3. Bolt length and spacing

Determination of bolt length and spacing has been a topic for discussion probably since rockbolts were first used for ground support in underground excavations (e.g. Panek, 1964; Coates and Cochrane, 1970; Lang, 1972; Barton et al., 1974; Schach et al., 1979; Farmer and Shelton, 1980; Crawford et al., 1985; Stillborg, 1994). Choquet and Hadjigeorgiou (1993) provided a review on this topic in their paper on the design of ground support. The following presented are the principles for the determination of bolt length and spacing that are used in the practise of rockbolting to date. From the point of view of operation, the bolt length should be less than half of the opening height for roof bolts and half of the span for wall bolts in order to avoid installation difficulties:

$$L_b \leq 0.5H \quad \text{(for roof bolts)} \tag{1a}$$

$$L_b \leq 0.5B \quad \text{(for wall bolts)} \tag{1b}$$

where L_b represents the bolt length, H is the opening height, and B is the opening span. The bolt length is also associated with the bolting principle. In the case that the failure zone is limited to a relatively small depth (Fig. 6a), the bolt length should be at least 1 m longer than the depth of the failure zone, i.e.

$$L_b \geq d_f + 1 \tag{2}$$

where d_f is the depth of the failure zone. In the case of a vast failure zone (Fig. 6b), the bolt length is short, varying from 2 m to 3 m, but its upper limit is governed by Eq. (1).

For tunnels excavated in moderately jointed hard rock masses, the Norwegian Road Authority proposed the following formula to determine the length of un-tensioned bolts in the central section of the tunnel for the purpose of suspending the failure zone on the natural arch (Statens vegvesen, 2000) (see Fig. 10a):

$$L_b = 1.4 + 0.184B \tag{3}$$

In practise, the bolt pattern in systematic bolting is such that the in-row spacing and the distance between rows are equal. The bolt spacing, s, is recommended to be in the range from 1 m to 2.5 m.

However, rock joint spacing should be also taken into account when the bolt spacing is determined. A rule of thumb is to set the bolt spacing equal to 3–4 times the mean joint spacing in the case of a mean joint spacing in the range of 0.3–1 m, i.e.

$$s = (3-4)e \tag{4}$$

where e represents the mean joint spacing.

In the case of a vast failure zone (Fig. 6b), the Norwegian Road Authority recommends the use of relatively short tensioned rockbolts to establish an artificial pressure arch in the failure zone (see Fig. 10b). The bolt length is still estimated using Eq. (3), but the bolt spacing is recommended to be smaller than 3 times the mean joint spacing, i.e.

$$s \leq 3e \tag{5}$$

For rockbolting aiming at the construction of an artificial pressure arch, it is required that the rockbolts interact with each other and an interaction zone is formed in the bolt-reinforced rock party (Fig. 11). Assuming that the reinforcement angle of a single rockbolt is 90°, the thickness of the interaction zone, t, is related to the bolt length (L_b) and spacing (s) as follows:

$$t = L_b - s \tag{6}$$

The bolt length is usually short, 2–3 m, in this type of rockbolting. The thickness of the interaction zone is required to be at least $0.5L_b$ in order that a strong enough artificial arch can be established in the broken rock. This requirement leads to a bolt spacing that should be less than half of the bolt length, i.e.

$$s \leq L_b/2 \tag{7}$$

In the design stage of an underground rock excavation, bolt length and spacing are often determined with the help of empirical methods recommended in various rock mass classification systems. In the Q rock mass classification system (Barton et al., 1974), the bolt length and spacing can be found in a chart based on the Q-value of

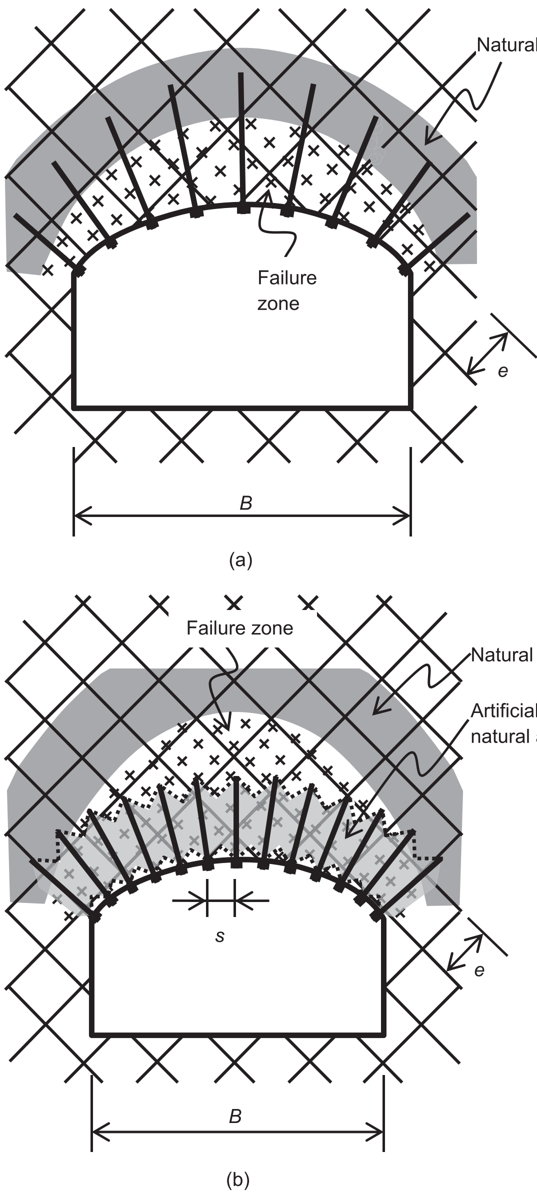

Fig. 10. Rockbolting methods in two different rock conditions: (a) suspension of the failure zone to the natural arch and (b) establishment of an artificial arch within the failure zone. Modified after Stillborg (1994).

the rock mass and a geometrical parameter called the equivalent dimension (Barton and Grimstad, 2014). The equivalent dimension is defined by the span of the excavation and a coefficient describing the intended use of the excavation (road tunnel, underground station, etc.). In the rock mass rating (RMR) system by Bieniawski (1989), bolt length and spacing, as well as other types of support measures, are empirically recommended in a table for five classes of rock mass quality.

3.4. Factor of safety

3.4.1. Factor of safety for gravitational rock falls

As mentioned in Section 2, rock 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 rockbolts is equal to the deadweight force of the falling blocks. In this load-controlled condition, the factor of safety (*FS*) for rockbolting is defined as

$$FS = \frac{\text{Load capacity of the bolts}}{\text{Total load on the bolts}} \tag{8}$$

In this case, a safe rock support requires that the load on the bolt is less than the strength of the bolt, i.e. $FS > 1$. It is required that the factor of safety is in the range of 1.5–3 in rockbolting design.

3.4.2. 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. The greater part of the released strain energy is dissipated in rock fracturing, 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 during excavation is described by the ground response curve (GRC) (see Fig. 12). Yield rockbolts work better than stiff ones in squeezing rock. Yield rockbolts deform together with the rock mass until the rock mass becomes stable after a certain amount of displacement. The dashed line in Fig. 12 represents the GRC of the rock mass after being reinforced with yield rockbolts. In squeezing rock conditions, one cannot find a constant load on the rock support since the support load and the displacement are correlated. It is thus not possible to use Eq. (8) to calculate a factor of safety. In squeezing rock, it is more relevant to define the factor of safety with 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. The factor of safety for the rock support, *FS*, is thus defined as

$$FS = \frac{u_c}{u_{eq}} \tag{9}$$

It must be pointed out that the critical displacement u_c is difficult to be quantified even with the help of numerical modelling. Beyond displacement u_c, the rock mass becomes unstable and calculation iterations would become non-convergent in numerical modelling. In engineering practise, there usually exists a maximum allowable displacement from the point of view of operation. For example, the radial displacement of a TBM (tunnel boring machine) tunnel usually is not allowed to be larger than 150 mm in order to avoid jamming of the TBM head. Thus, there is another factor of safety for the operation, denoted as FS_{op}, which is calculated as

$$FS_{op} = \frac{u_{max}}{u_{eq}} \tag{10}$$

where u_{max} is the maximum allowable displacement. In addition to the factors of safety for stability and operation, it is also required that the rock displacement at equilibrium, u_{eq}, must be smaller than the ultimate displacement u_{ult} of the rockbolt to avoid premature failure of the rockbolt. To the end, the items in the criteria for a rock support system using rockbolts are as follows:

$$\left.\begin{aligned} FS &= \frac{u_c}{u_{eq}} > 1 \\ FS_{op} &= \frac{u_{max}}{u_{eq}} > 1 \\ u_{eq} &< u_{ult} \end{aligned}\right\} \tag{11}$$

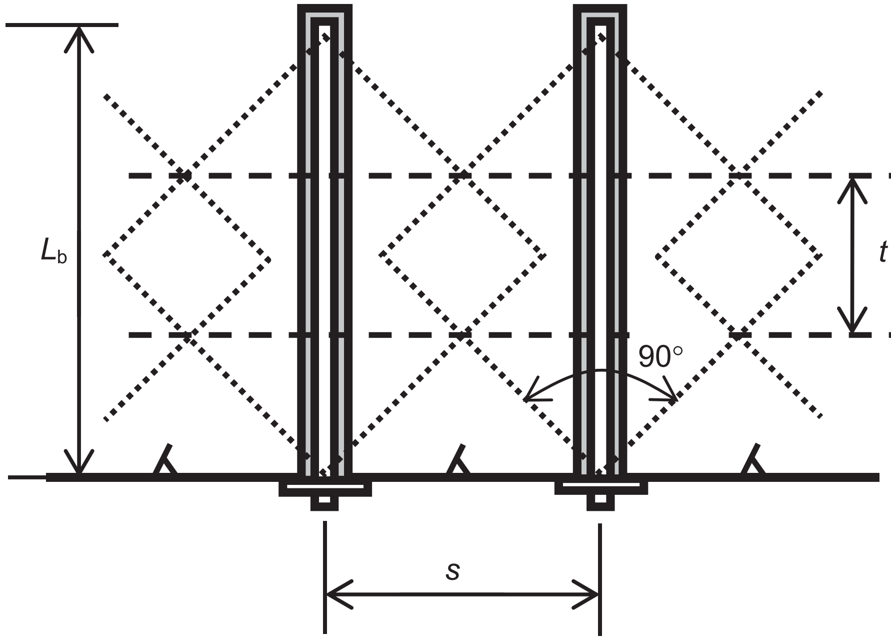

Fig. 11. Reinforcement interaction between rockbolts.

3.4.3. Factor of safety in burst-prone rock

In highly stressed rock masses, a portion of the strain energy stored in the rock mass may be released suddenly, leading to rockburst events. Use of energy-absorbing rockbolts is one of the most effective means to support burst-prone rock masses. The support principle in this case is that the energy absorption capacity of the rockbolts must be higher than the kinetic energy of the ejected rock. The factor of safety for dynamic rock support needs to be calculated with the energy absorption capacity of the rockbolts and the energy released in the rockburst event:

$$FS = \frac{nE_{ab}}{E_{ej}} \tag{12}$$

where n is the number of rockbolts; E_{ab} is the energy absorption of each bolt; and E_{ej} is the kinetic energy of the ejected rock, which is expressed as

$$E_{ej} = \frac{1}{2}mV^2 \tag{13}$$

where m is the mass of the ejected rock, and V is the ejection velocity. The ejection velocity may be estimated according to the horizontally dislodged distance of the ejected rock in case that the burst occurs in walls (Kaiser et al., 1996). Rockburst, however, often occurs in tunnel roof and also in floor (Zhang et al., 2012). In such cases it is not possible to estimate the ejection velocity by this means since the horizontally dislodged distance is zero. A high ejection velocity may elevate the broken degree of the rock pile. Thus, one may empirically estimate the ejection velocity based on the fragmentation of the ejected rock in the case of roof rockburst. With a competent support system, the ejected rock will stop moving after a displacement of u_{eq} (Fig. 13). As mentioned above, the displacement u_{eq} has to be smaller than the maximum allowable displacement, u_{max}, in order to avoid operational difficulties. The criteria for dynamic rockbolting design are

$$\left.\begin{aligned} FS &= \frac{nE_{ab}}{E_{ej}} > 1 \\ FS_{op} &= \frac{u_{max}}{u_{eq}} > 1 \\ u_{eq} &< u_{ult} \end{aligned}\right\} \tag{14}$$

3.5. 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 (Schubert, 2001; Li, 2012). 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 Fig. 14a. 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, yield rockbolts and meshes usually are used to deal with excessive rock deformation. The support load is mainly carried by the rockbolts while the mesh restrains the dilation of the rock surface. Fig. 14b is a conceptual sketch of this type of support system. In such a support system, 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 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 behaviours of the internal and external support elements in such a system are sketched in Fig. 15.

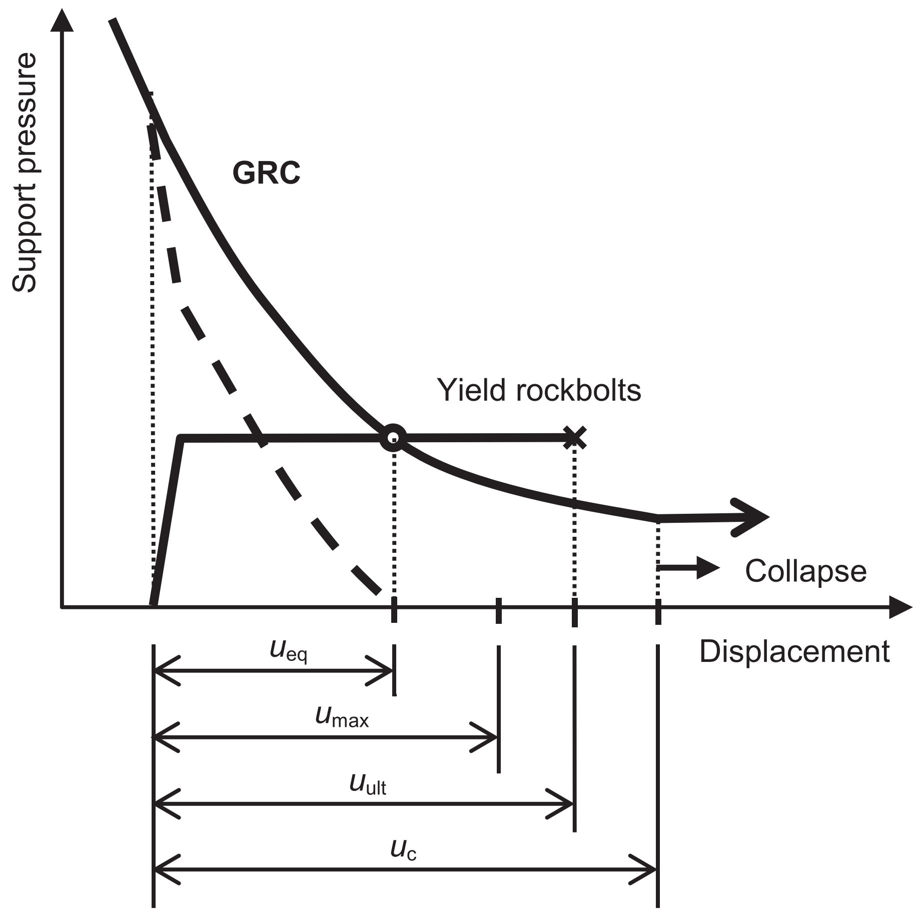

Fig. 12. The ground response curve (GRC) and the support characteristic curve of yield rockbolts.

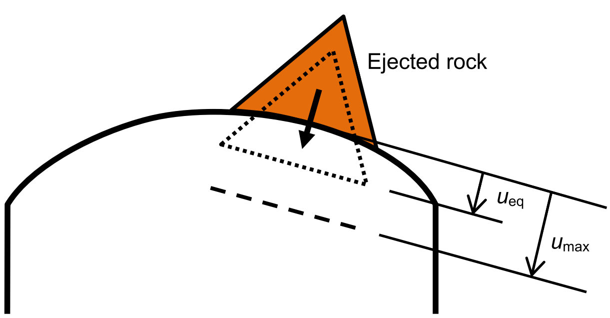

Fig. 13. The equilibrium displacement u_{eq} and the maximum allowable displacement u_{max} related to a rockburst event.

4. Types of rockbolting

4.1. Wedge block in roof

4.1.1. Stability analysis

Consider a horizontal overhanging rock face intersected by three planar discontinuities. A tetrahedral wedge block is formed by the three discontinuities and the rock face, as shown in Fig. 16. This block is kinematically feasible and tends to fall under gravity. The horizontal in situ rock stress is helpful in stabilising the block. Slippage along a discontinuity is prohibited when the dip angle of the discontinuity plane to the hanging rock face is larger than a critical dip angle equal to (90°−ϕ_a) when the weight of the block is neglected, where ϕ_a is the apparent friction angle of the discontinuity. However, the gravity of the roof wedge block does play a role in the stability of the block. The stability of a symmetric wedge in the roof of a tunnel has been studied by many researchers, for instance, Crawford and Bray (1983), Shi and Goodman (1983), Sofianos (1986), and Nomikos et al. (2002, 2006). In the following, a simple case without considering the stiffness of rock joints is considered to demonstrate the influence of the size of the block on the critical dip angle when the gravity is taken into account. Consider a longitudinal wedge block, which is formed by two discontinuities with the strikes parallel to the tunnel axis as well as the roof surface, as an example to understand the influences of the size of the block and the horizontally oriented tangential rock stress σ_θ (Fig. 17). The loads on the wedge block are the tangential rock stress σ_θ, the weight force of the block and the frictional resistance on the sides of the block. The block tends to fall under gravity, but the friction on the sides tends to prohibit the fall. The requirement for stabilising the block is that the frictional resistance on the discontinuity planes is higher than the downward shear force on the planes. Taking into account all the forces exerted on the two discontinuity planes, the stability condition for the block is obtained as follows:

$$\frac{2\sigma_\theta - \rho gh}{2\sigma_\theta + \rho gh}\tan\phi_a > \sin(2\alpha) + \cos(2\alpha)\tan\phi \quad (15)$$

where ρ is the density of the rock, g is the gravitational acceleration, h is the block height, and α is the dip angle of the discontinuities. The critical dip angle can be found by letting the two sides of Eq. (15)

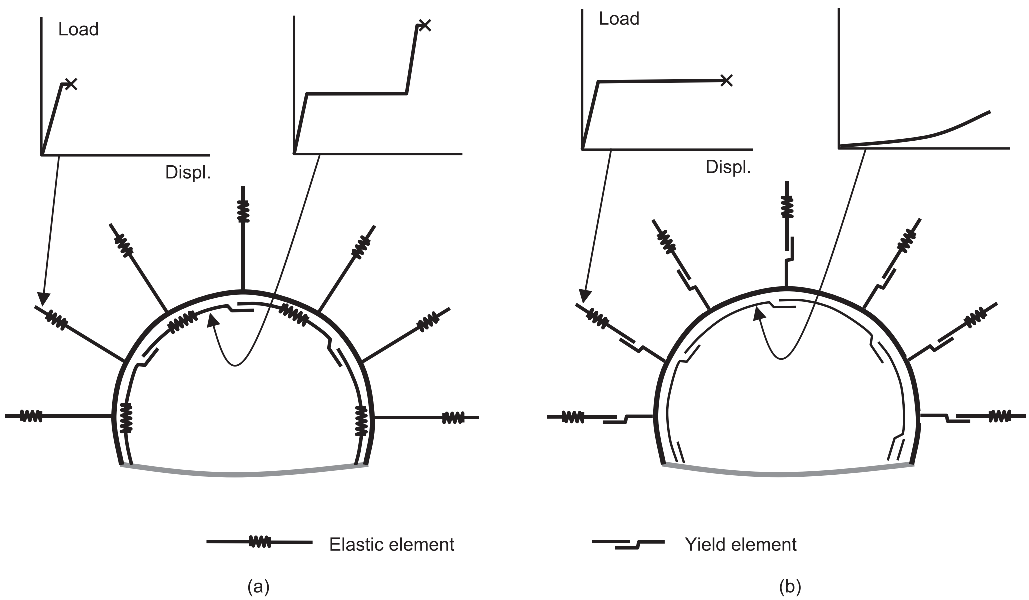

Fig. 14. Sketches illustrating incompatible rock support systems: (a) in civil tunnelling and (b) in mining.

be equal. Given $\rho = 2700$ kg/m^3, $g = 10$ m/s^2 and $\phi = 35°$, the critical dip angle is related to the tangential rock stress σ_θ and the block height h, as shown in Fig. 18. The critical dip angle decreases with an increase in the tangential rock stress and approaches 55°, which is the critical dip angle without consideration of gravity. The critical dip angle increases with a decrease in the tangential rock stress, implying that only steeply dipped wedge blocks could be stabilised by the friction on the block sides in low stress conditions. The critical dip angle changes abruptly with a small change in the tangential rock stress below 1 MPa, but it is not very sensitive to the rock stress above 2 MPa. As shown by the three curves corresponding to different block heights, the critical dip angle also increases with the block size, particularly when the tangential rock stress is lower than 1 MPa. The critical dip angle increases slightly with the block size when the rock stress is higher than 2 MPa. This implies that the critical dip angle is not sensitive to block size when the tangential rock stress is high.

4.1.2. Bolting

The load on rockbolts that are used to stabilise a rock block in the roof is the deadweight force of the block (see Fig. 1). The number of bolts needed, N_{bolt}, is approximately calculated as

$$N_{\text{bolt}} = FS\frac{Wg}{P_{\text{ult}}} \tag{16}$$

where W is the deadweight force of the block, and P_{ult} is the ultimate load of the rockbolt. In the case of fully grouted rockbolts, the embedment length of the bolts in the stable formation must be at least 1 m.

4.2. Wedge block in wall

The principle of stabilising a wedge block in the wall was presented, for instance, by Hoek and Brown (1980) and Harrison and Hudson (2000). It is demonstrated in this section through the example illustrated in Fig. 19a. Assume that a wedge block is formed in the wall and it tends to slide along the lower discontinuity under gravity. The block is stabilised with bolts which are installed with an angle of α to the discontinuity plane. The total reinforcement force contributed by the bolts is $T = \Sigma t$ where t is the force in a single rockbolt. All forces exerted on the block are illustrated in Fig. 19b. They are the gravitational force acting on the block, Wg, the total bolt force, T, the normal reaction force on the sliding plane, N, and the shear resistant force on the sliding plane, R. All forces are in equilibrium in all directions in the critical state at which shear failure occurs along the sliding plane. By equilibrating the forces, it is obtained that the normal force is expressed as

$$N = Wg\cos\psi + T\sin\alpha \tag{17}$$

where ψ is the dip angle of the sliding plane. The shear resistant force is expressed, with an assumption that the Mohr–Coulomb criterion prevails along the sliding plane, as

$$R = cA + (Wg\cos\psi + T\sin\alpha)\tan\phi_{\text{a}} \tag{18}$$

where A is the base area of the sliding plane. The driving shear force, D, is obtained as

$$D = Wg\sin\psi - T\cos\alpha \tag{19}$$

The factor of safety of the wedge block is defined as the ratio of the shear resistant force to the driving shear force, i.e.

$$FS = \frac{R}{D} = \frac{cA + (Wg\cos\psi + T\sin\alpha)\tan\phi_{\text{a}}}{Wg\sin\psi - T\cos\alpha} \tag{20}$$

A factor of safety less than 1, i.e. $FS \le 1$, means that sliding occurs, while the block is stable when $FS > 1$. A factor of safety in the range of 1.5–2 is usually used for rockbolting design.

The bolt force T contributes to an increase in the normal force and a component of the shear force. The increase in the normal force is always positive in enhancing the frictional resistance of the sliding plane, but the contribution of the bolt force to the shear force is either positive or negative, depending on the installation angle, α. There exists a theoretical critical installation angle, denoted as α_{c}, at which the bolts most effectively reinforce the block. Let $FS = 1$ in Eq. (20), representing the equilibrium state when shear failure is initiated along the sliding plane. The bolt force is expressed at this moment as

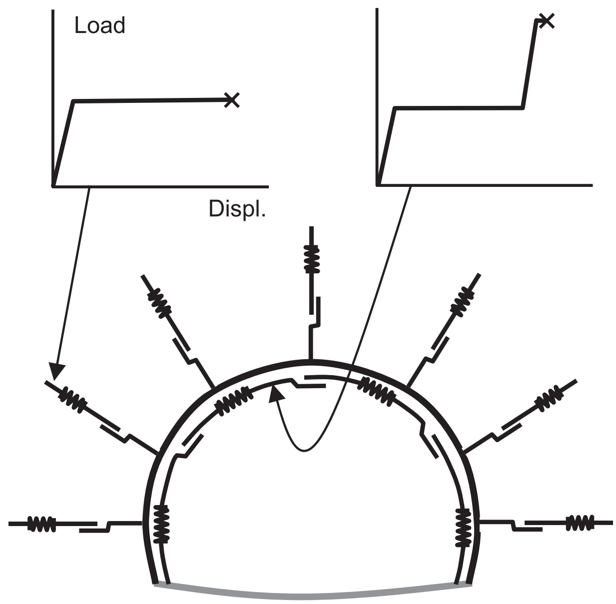

Fig. 15. A sketch illustrating the concept of a compatible rock support system.

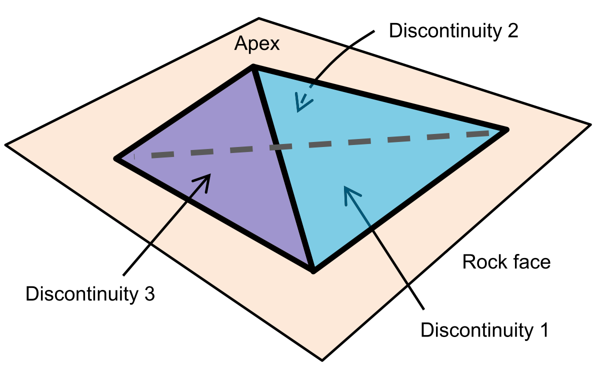

Fig. 16. A kinematic wedge bounded by three planar geological discontinuities and a horizontal overhanging rock face.

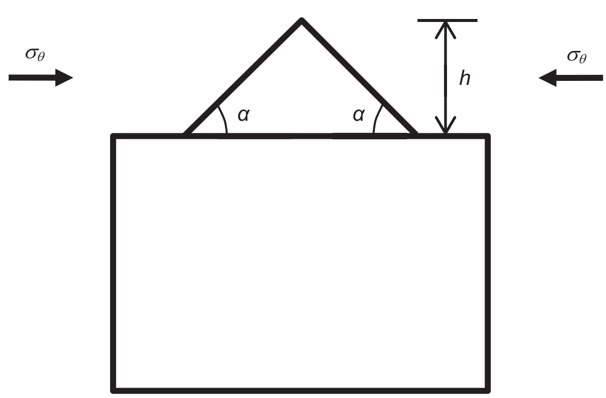

Fig. 17. A wedge block formed in the tunnel roof.

$$T = Wg\frac{\sin\psi - \cos\psi\tan\phi_a}{\cos\alpha + \sin\alpha\tan\phi_a} \tag{21}$$

The bolt force needed to equilibrate the other forces on the block at this moment is minimum when $\partial T/\partial\alpha = 0$. Differentiating the expression above with respect to α and letting the differentiation be equal to zero, the critical installation angle is obtained as

$$\alpha_c = \phi_a \tag{22}$$

In other words, the reinforcement effect of the rockbolts is optimum when they are installed at an angle of $\alpha = \phi_a$.

4.3. Arching bolting

The concept of a natural pressure arch is further explained through the arching of two blocks, as illustrated in Fig. 20. Assume that the ceiling of an opening excavated in a laminated rock mass is composed of two blocks formed by three transverse fractures in the ceiling stratum. The downward movement of the two blocks at the abutments is prohibited owing to the friction on the fracture planes. The two blocks are then forced to rotate under gravity and press each other at the upper part of the middle fracture plane and the lower parts at the abutments. A pressure arch is thus formed within the two blocks and the blocks are stabilised.

The natural pressure arch is located far from the ceiling of the underground opening in the case of a vast failure zone around the opening after excavation. In this case, one can consider constructing an artificial pressure arch within the failure zone to prevent the failed rock from falling. As demonstrated in physical models by Lang (1961) and also by Hoek (2007), an artificial pressure arch may be formed in the interaction zone of systematically installed rockbolts (see Fig. 21). The load-bearing capacity of such a pressure arch can be estimated by (Krauland, 1983; Sinha, 1989):

$$\sigma_{max} = k\sigma_c\left(\frac{t}{B}\right)^2 \tag{23}$$

where σ_{max} is the maximum ground pressure that the pressure arch can bear, and σ_c is the uniaxial compressive strength (UCS) of the bolt-reinforced rock party. The coefficient k is proportional to the moment arm length in the pressure arch. Wright (1973) found that k is approximately 0.9 based on back-calculations of his experimental data.

4.4. Tieback bolting

Tieback bolting is usually used to reinforce rock pillars. With tieback bolting, the bolts, usually equally spaced, go through the entire width of the pillar and are pre-tensioned with a relatively high load (see Fig. 22). It should be noted that the purpose of tieback bolting is not to enhance the peak strength of the pillar to avoid failure but to prevent the pillar from disintegration in the post-failure stage. In accordance with the Mohr–Coulomb criterion, the increase in the strength of the rock by the confining pressure σ_3 is expressed as

$$\Delta\sigma_1 = \sigma_3\tan^2\left(45^\circ + \frac{\phi}{2}\right) \tag{24}$$

Take as an example a pillar reinforced by systematic tieback bolting with a bolt spacing of 1 m. The ultimate load of the bolt is 200 kN and thus the maximum confining stress to the rock by the bolts is $\sigma_3 = 0.2$ MPa. Assume that the peak and residual internal friction angles are 40° and 30°, respectively. It is obtained from Eq. (24) that the increased peak strength is 0.9 MPa and the increased residual strength is 0.6 MPa. The UCS of the rock is usually higher than 50 MPa. An increase of 0.9 MPa is negligible compared to the inherent strength of the rock. However, the unconfined residual strength of the rock is low in most types of rocks. Therefore, an increase

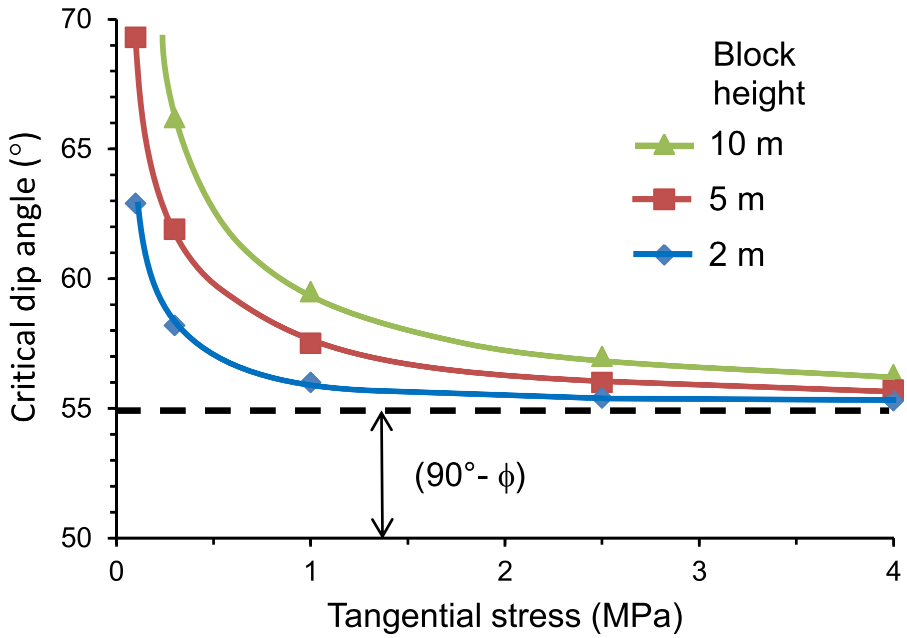

Fig. 18. The critical dip angle versus the tangential rock stress in the rock for three different block sizes.

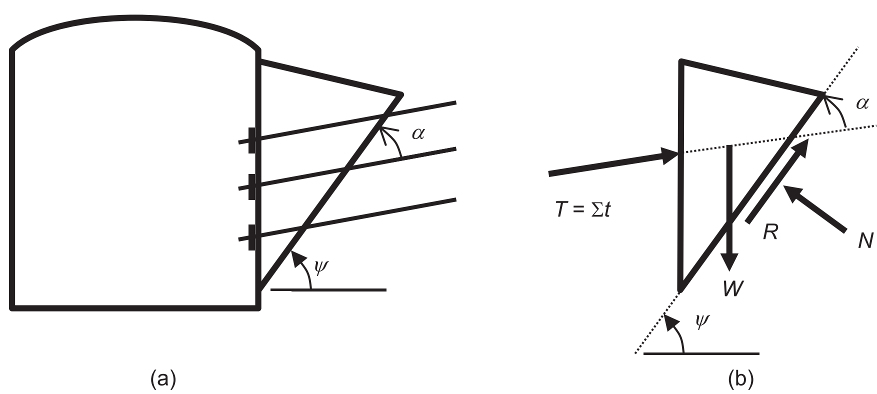

Fig. 19. Use of rockbolts to stabilise a wedge block that tends to slide along the lower discontinuity plane. (a) The block and the rockbolts, and (b) the forces on the wedge block.

of 0.6 MPa in the residual strength could significantly improve the post-failure behaviour of the pillar.

Fig. 23 shows the tieback bolting of a 6 m × 8 m (width × depth) pillar between two niches excavated in the wall of a hydropower cavern. The pillar was subjected to extension fracture during excavation. Cablebolts with a load capacity of 1 MN were installed across the pillar with a spacing of 2 m. They were pre-tensioned to 400 kN after installation. A concrete lining of 300 mm in thickness was cast on both sides of the pillar to improve the load transfer from the cablebolts to the pillar. Strong square plates of 200 mm × 200 mm were attached to the cablebolts. The maximum confining pressure that the cablebolts can provide is approximately 1 MN/(2 m × 2 m) = 0.25 MPa. The increased strength of the pillar is estimated to be 0.75 MPa, corresponding to an increase of the load capacity of 36 MN for the 6 m × 8 m pillar (assuming an internal friction angle of 30°).

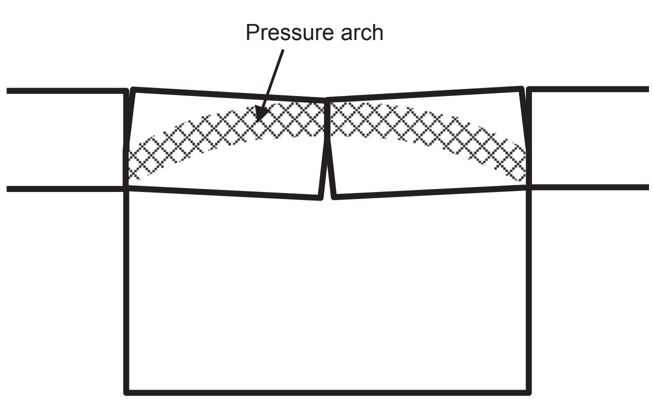

Fig. 20. Pressure arch formed in two ceiling blocks.

4.5. Suspension bolting

In some cases, often in coal mines, a weak layer of formation is exposed on the roof of mine drifts (Krauland, 1983). This layer can be secured by hanging it to the stable stratum behind with rockbolts (Fig. 24). The weak layer is loaded by its own weight so that the bolting design is simply based on the thickness of the layer and

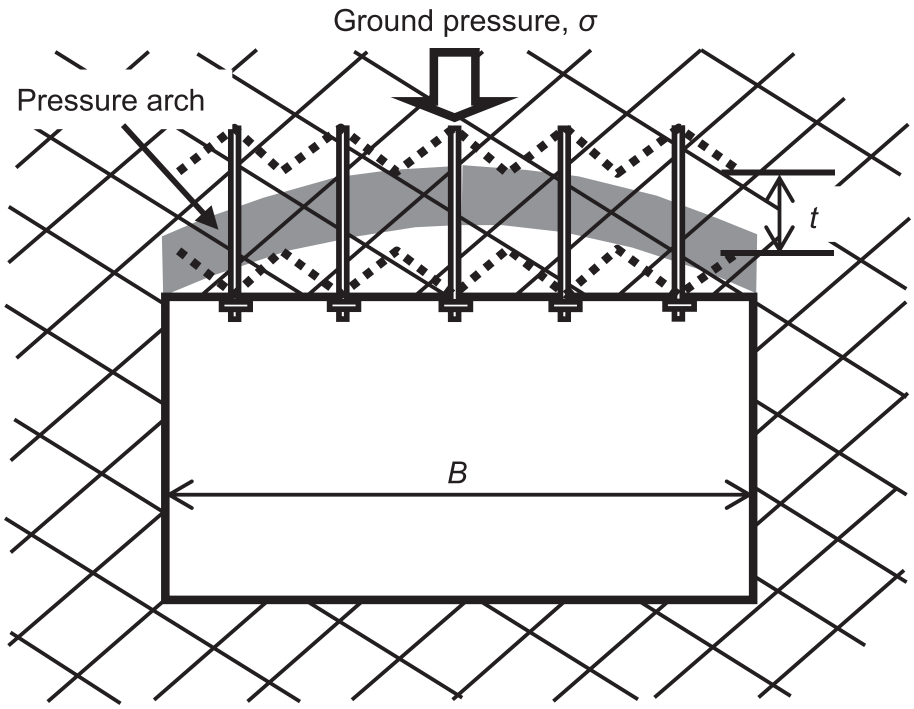

Fig. 21. Pressure arch formed in a bolt-reinforced roof.

the bolt spacing. The required ultimate load capacity of the rockbolt is

$$P_{\mathrm{ult}} = FS(lsC\rho g) \tag{25}$$

where l is the thickness of the weak layer, and C is the bolt spacing between rows. The minimum bolt length, $L_{\min}$, is

$$L_{\min} = l + \text{anchorage length} \tag{26}$$

The anchorage length in the stable stratum should be at least 1 m in the case of fully encapsulated bolting. The principle is that it must be longer than the critical embedment length with a factor of safety of 2–4.

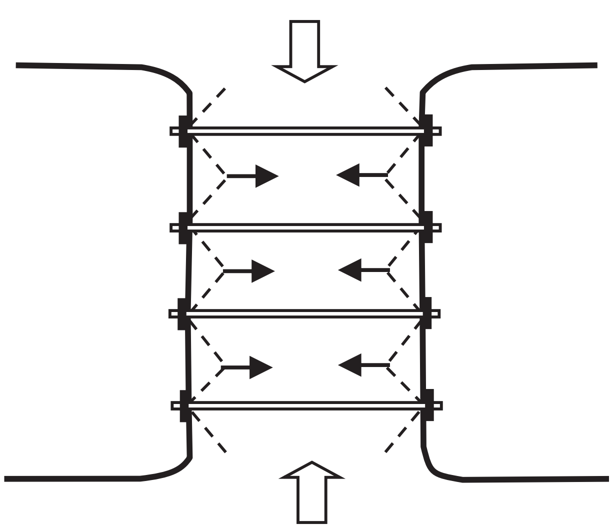

Fig. 22. Tieback bolting.

4.6. Rockbolting in large-scale caverns

Rock failure may extend to a significant depth after excavation of a large-scale underground cavern. Rockbolts and cablebolts play crucial roles in the support system in this case because other support elements, such as lining, shotcrete and mesh, can only passively respond to the rock deformation and provide very limited effective support to the rock mass. When the rock mass quality is poor and the in situ rock stresses are high, the size of the failure zone could be so vast that it is beyond the bolt-reinforced zone and the reinforced rock party continues to move toward the opening. The principle of rockbolting in this situation is to reinforce the rock with tightly spaced and fully grouted rockbolts to a relatively shallow depth (3–7 m) in combination with long cablebolts (10–25 m). The short rockbolts aim to build up a bolt-rock 'shield' surrounding the opening and the long cablebolts suspend the 'shield' to competent and stable rock formations

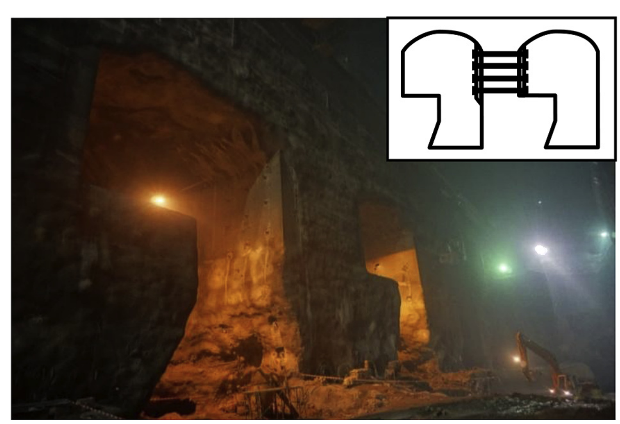

Fig. 23. Tieback bolting in a hydropower cavern. Some of the pillar-through cablebolts are shown in the picture (Photo by J. Mierzejewski).

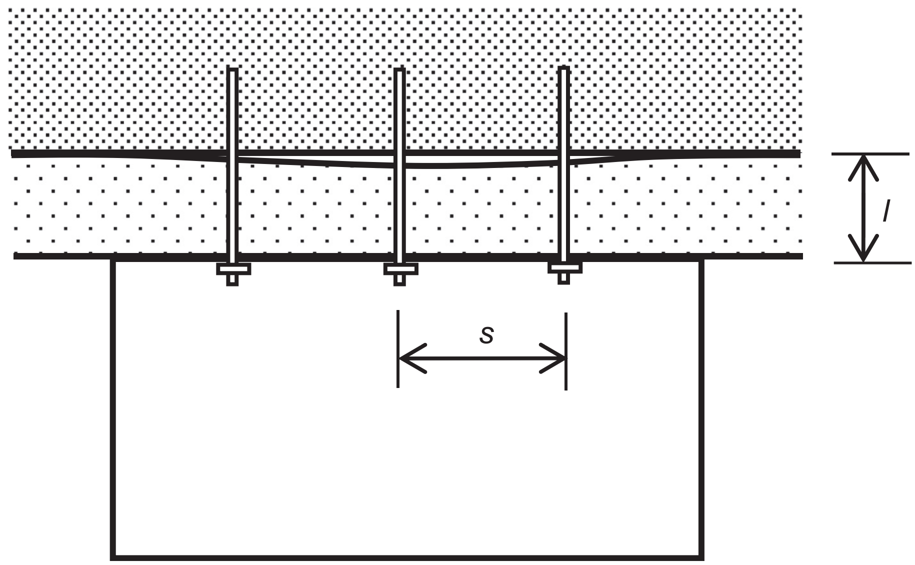

Fig. 24. Suspension bolting.

behind the failure zone. It is required that the cablebolts must be able to tolerate a significant displacement in order that they function properly without premature failure. A common practise to enhance the deformability of a cablebolt is to de-bond its middle portion from the grout with PVC pipes or other types of soft materials.

Fig. 25 shows the rockbolting design for a hydropower cavern located at a depth of approximately 400 m. The cavern is 25 m in span, 45 m in height and 120 m long. The main rock types are sandstone (UCS of 80–150 MPa), siltstone (UCS of 40–60 MPa) and mudstone (UCS of 20–50 MPa). The bedding planes approximately dip toward the upstream with an angle of 30°. The in situ horizontal stress in the rock mass is slightly higher than the vertical stress. The final wall–wall convergence of the cavern is expected to be 200–300 mm. The 7-m long short rockbolts, with load capacity of 300 kN, are fully grouted in the boreholes. The short bolts are spaced 1 m × 1 m in the crown and 1.3 m × 1.3 m in the walls. The load capacity of every cablebolt is 1 MN. In every bolting profile, the cablebolts are spaced 4 m × 4 m in the crown with the three in the middle being 15 m long and the rest 10 m long. The cablebolts in the walls are 15 m long and spaced 5 m × 5 m. Three 20-m cablebolts are installed in

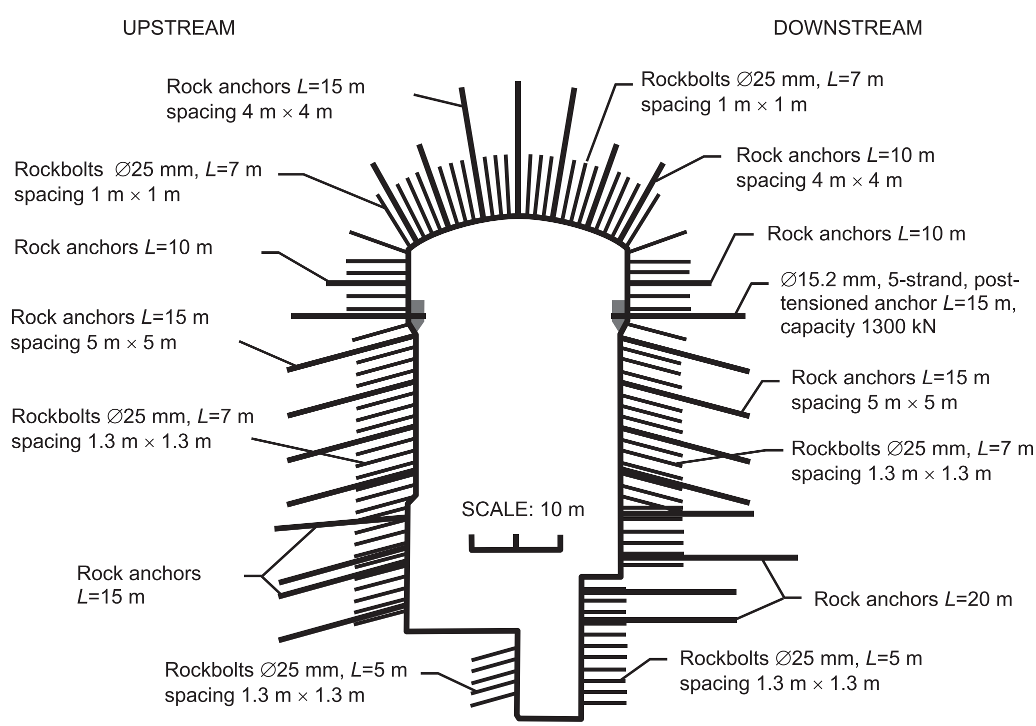

Fig. 25. An example of rockbolting and cablebolting in a large-scale underground hydropower cavern.

the lower portion of the wall on the downstream side because of the risk that the rock mass there may be weakened by excavations nearby.

4.7. Forepoling

In unstable ground, the crown of a tunnel can be pre-supported by driving poles, pipes and planks into the crown ahead of the excavation face with a small inclination angle upward. This is the so-called forepoling reinforcement (see Fig. 26). The pole types range from rebar spiles, self-drilling spiles, pipe spiles and planks.

Rebar spiles consist of smooth or ribbed solid steel bars. They are installed in pre-drilled stable boreholes with cementitious grout in blocky and jointed rock mass against falling blocks, as well as rammed into soft, homogenous soil to prevent loosening of the soil.

Pipe spiles consist of steel tubes. They are installed in pre-drilled stable boreholes in blocky or jointed rock mass as well as rammed into soft, homogenous foundation soil. The hollow hole of the pipes and the gap between the pipes and the ground can be grouted to achieve a better load transfer.

Self-drilling spiles are simply the hollow drill rods that are left in the strata after hole drilling. The drill bit is either left in the strata or retrieved from the inner hole of the rod. Self-drilling spiles are suitable for extremely poor rock masses and consolidated but weak soils. They are installed with conventional drill booms using rotary percussive drilling.

Forepoling planks are particularly suitable for unstable, non-cohesive soil. One product of such a plank is 1.25–3 m in length and approximately 220 mm in width and is made of steel plate of 3–6 mm in thickness (DSI, 2015). The plank is rammed into the soil using hydraulic drifters.

Forepoling with solid bars is usually applied after each excavation round with a length seldom beyond 6 m. Forepoling with pipes, or a so-called pipe umbrella, is installed to support several consecutive excavation rounds. The pipe length is usually 12 m or 15 m (Volkmann and Schubert, 2009). For pre-reinforcement of a 12-m long tunnel excavated in a weak rock mass, Hoek (2015) proposed the following solution: the grouted pipe forepoles would be 12 m long and 114 mm in diameter at a spacing of 0.3–0.6 m; the pipe forepoles would be installed every 8 m to provide a minimum of 4 m overlap between successive umbrellas. The specification of the most commonly used forepoles is given in Table 1.

Forepoles are loaded laterally by the loosened ground materials above. Three things must be done in order that the forepoles work as desired (see Fig. 27). Firstly, the toe of the poles must be in a few metres ahead of the excavation face; secondly, the near-end of the poles must be supported by rockbolts, arches or both; and thirdly, the poles must be overlapped. The poles must have at least two support positions, one being the face rock and the other being the rockbolts and arches at the near end. Additional support arches are needed when the pole length is much longer than the advance length. Support arches can be lattice girders, steel sets, shotcrete, etc.

A long forepole umbrella with several support arches underneath can be divided into two areas, A and B, according to the

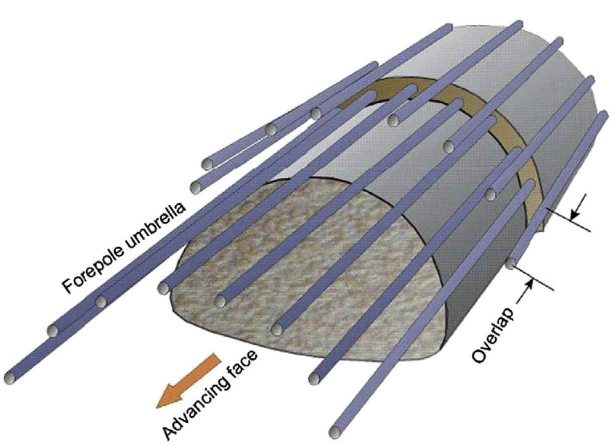

Fig. 26. Sketch of tunnelling under the protection of a forepole umbrella (Aksoy and Onargan, 2010; Hoek, 2015).

Table 1
Specification of forepoles (Bang, 1984; Ocak, 2008; Volkmann and Schubert, 2009; DSI, 2015; Hoek, 2015).

Type of forepoles	Diameter (mm)	Length (m)	Spacing (m)	Angle (°)
Solid rebar spiles	20–50	4.6–6.1	0.5–1.5	6–10
Steel pipes	38–200	9–15	0.3–0.6	6–10
Self-drilling spiles	32–51	–	–	6–10

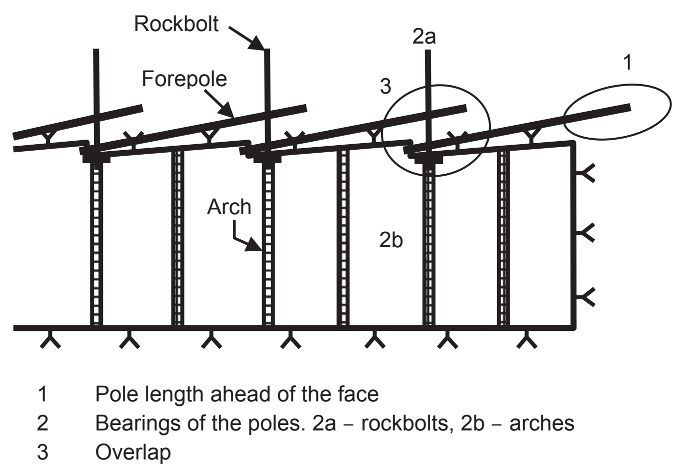

Fig. 27. Longitudinal profile of forepoling.

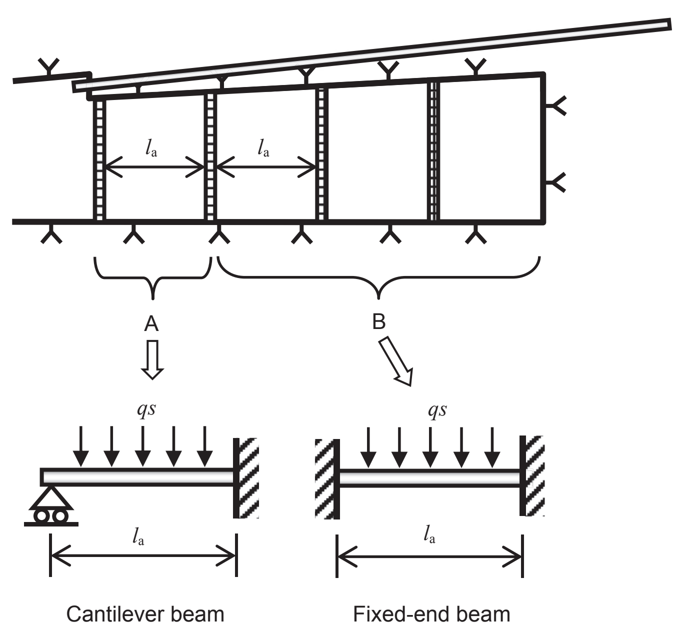

Fig. 28. Beam models for pole sections between supporting arches.

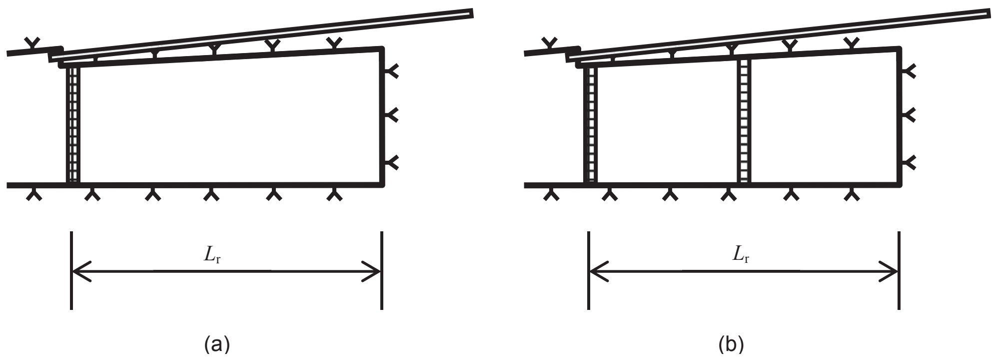

Fig. 29. Forepole umbrellas. (a) Without supporting arch in between and (b) with a supporting arch in the middle.

bending nature of the poles (Fig. 28). The uniformly distributed load along a pole is expressed by qs, where q is the ground pressure and s is the spacing between poles. The pole section in area A can be simplified to a cantilever beam loaded by the uniformly distributed load qs. The maximum deflection, $\delta_{A,m}$, of this pole section is expressed, according to beam theory, as

$$\delta_{A,m} = 5.4\frac{qs}{EI}l_a^4 \times 10^{-3} \tag{27}$$

where l_a is the arch spacing, E is the Young's modulus of the pole material, and I is the bending moment of the pole. Every pole section between two adjacent arches in area B can be simplified by an end-fixed beam. The maximum deflection, $\delta_{B,m}$, of the fixed-end beam is expressed as

$$\delta_{B,m} = 2.6\frac{qs}{EI}l_a^4 \times 10^{-3} \tag{28}$$

It is seen by comparing the two expressions above that the maximum deflection of the pole in area B is approximately one half of the deflection of the pole section in area A. The pole is thus most deflectable in the section close to the near end of the pole. If only one arch is set up at the near end of the poles (Fig. 29a), every pole functions as a cantilever beam. The maximum deflection of the poles is expressed, according to Eq. (27), as

$$\delta_{A,m0} = 5.4\frac{qs}{EI}L_r^4 \times 10^{-3} \tag{29}$$

where L_r represents the round length. If there is an additional supporting arch in the middle of distance L_r, the span between the arches becomes $L_r/2$. The maximum deflection of the pole section close to the near end of the pole becomes

$$\delta_{A,m1} = \frac{1}{16}\delta_{A,m0} \tag{30}$$

In other words, the deflection of the pole with an additional arch in the middle of the round is reduced to one sixteenth of the deflection without the additional arch. This means that the middle arch is effective in making the poles stiffer.

The following is an example of spiling adopted for rock support in squeezing rock in a metal mine. The spiles are either rebars or self-drilling rockbolts depending on the rock conditions. The rockbolts are at least 6 m long for an advance length of 4 m and fully grouted with cement in the boreholes. In the case of using self-drilling bolts, the bolt must be drilled at least 1 m beyond the advance face. No matter what type of spiles is used, they must be longer than the advance length of blasting.

Installation of the spiles is depicted in Fig. 30. Spiling holes are located approximately 1 m above the contour. They are drilled 10°–15° upward with a spacing of 0.3 m.

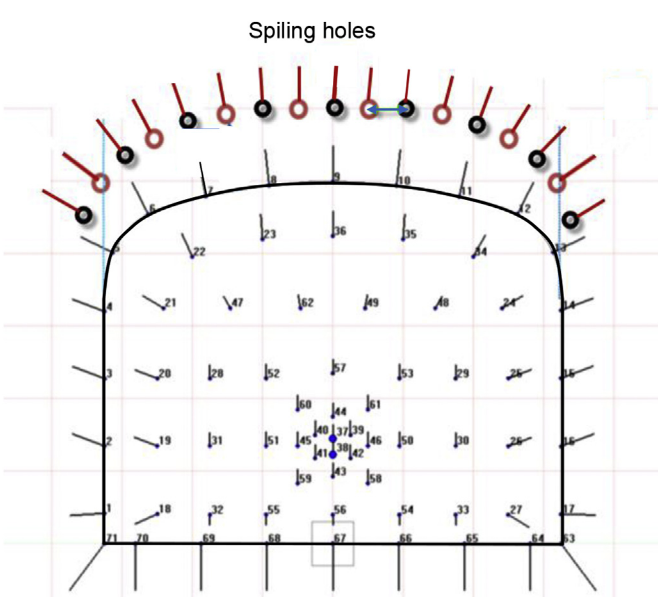

Fig. 30. Spiling in a mine drift.

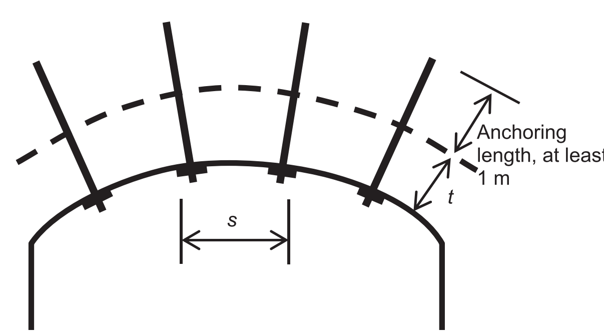

Fig. 31. Systematic rockbolting in burst-prone rock conditions.

4.8. Rockbolting in burst-prone rock

Energy-absorbing rockbolts, such as the D-bolt and cone bolt, should be installed in burst-prone areas in order to achieve a satisfactory reinforcement effect. The occurrence of rockburst is random both in position and time. Thus rockbolts are usually systematically installed, for example, with a spacing pattern $s \times s$ (see Fig. 31). Assuming that the expected ejection depth is t, the bolt length should be at least 1 m longer than the ejection depth. The bolt spacing s is then required according to Eq. (14) to be

$$s^2 = \frac{1}{FS}\frac{2E_{ab}}{t\rho V^2} \tag{31}$$

It is crucial that the rockbolts have a strong link with surface retaining elements such as the mesh and straps so that the load on the surface support elements is transferred to the rockbolts.

5. Conclusions

The strength of rockbolts is the key parameter for rockbolting design in low stress rock masses. Rockbolts should be deformable in addition to the requirement of high strength in high stress rock masses. In other words, rockbolts should be energy absorbent in squeezing and burst-prone rock conditions.

There exists a natural pressure arch immediately outside of the failure zone in the rock surrounding an underground excavation. In the case of a shallow failure zone, the rockbolts should be long enough to reach the pressure arch. In the case of a vast failure zone, short rockbolts are tightly installed to establish an artificial pressure arch within the failure zone and long cables are anchored into the natural pressure arch.

Determination of the bolt length and spacing is associated with the methodology of rockbolting. In the case of the anchorage of rockbolts in the natural pressure arch, the bolt length should be at least 1 m beyond the failure zone. In the case of establishing an artificial pressure arch, appropriate bolt lengths are approximately 3 m in mine drifts and up to 7 m in large-scale hydropower caverns. Bolt spacing is more important than bolt length in this case. The principle is that the bolt spacing guarantees that the rockbolts interact with each other. The appropriate bolt spacing is 1 m for 3-m long bolts and less than 1.5 m for 7-m long bolts.

The rockbolting design is based on the deadweight force of falling blocks and the strength of the rockbolt in low rock stress locations. For high rock stresses, one should take into account the portion of the rock-released energy that needs to be taken care by the rockbolts. The maximum allowable displacement and the ultimate displacement capacity of the rockbolt should also be taken into account.

The rockbolts in a rock support system should be compatible with other support elements with respect to displacement and energy absorption capacities.

Conflict of interest

The author wishes to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

References

Aksoy CO, Onargan T. The role of umbrella arch and face bolt as deformation preventing support system in preventing building damages. Tunnelling and Underground Space Technology 2010;25(5):553–9.

Bang S. Limit analysis of spiling reinforcement system in soft ground tunneling. Tunnel 1984;3:140–6.

Barton N, Grimstad E. Tunnel and cavern support selection in Norway, based on rock mass classification with the Q-system. Publication No. 23. Norwegian Tunnelling Society; 2014. p. 45–77.

Barton N, Lien R, Lunde J. Engineering classification of rock masses for the design of tunnel support. Rock Mechanics 1974;6(4):189–239.

Bieniawski ZT. Engineering rock mass classifications. New York: Wiley; 1989.

Choquet P, Hadjigeorgiou J. The design of support for underground excavations. In: Excavation, support and monitoring. Comprehensive rock engineering – principles, practice and projects, vol. 4. Pergamon Press; 1993. p. 313–48.

Coates DF, Cochrane TS. Development of design specifications for rock bolting from research in Canadian mines. Research Report R224. Mining Research Centre, Energy, Mines and Resources Canada; 1970.

Crawford AM, Bray JW. Influence of the in situ stress field and joint stiffness on rock wedge stability in underground openings. Canadian Geotechnical Journal 1983;20(2):276–87.

Crawford AM, Ng L, Lau KC. The spacing and length of rock bolts for underground openings in jointed rock. In: Einsenstein Z, editor. Proceedings of the 5th international conference on numerical methods in geomechanics. A.A. Balkema; 1985. p. 1293–300.

DSI. Spiles and forepoling boards. Product catalog. DWIDAG-System International; 2015.

Farmer IW, Shelton PD. Factors that affect underground rockbolt reinforcement systems design. Transactions of the Institutions of Mining and Metallurgy 1980;89:68–83.

Harrison P, Hudson J. Engineering rock mechanics. Part 2: illustrative worked examples. Pergamon; 2000.

Hoek E, Brown ET. Underground excavations in rock. London: Institution of Mining and Metallurgy; 1980.

Hoek E. Model to demonstrate how bolts work. In: Practical rock engineering; 2007.

Hoek E. Numerical modelling for shallow tunnels in weak rock. 2015. https://www.rocscience.com/documents/pdfs/rocnews/Spring2003/ShallowTunnels.pdf.

Kaiser PK, Tannant DD, McCreath DR. Canadian rock burst support handbook. Sudbury, Canada: Geomechanics Research Center; 1996.

Krauland N. Rockbolting and economy. In: Stephansson O, editor. Rockbolting – theory and applications in mining and underground construction. Rotterdam: A.A. Balkema; 1983. p. 499–507.

Lang T. Rock reinforcement. Bulletin of the International Association of Engineering Geology 1972;9:215–39.

Lang TA. Theory and practice of rockbolting. Transactions of American Institute of Mining, Metallurgical, and Petroleum Engineers 1961;220:333–48.

Li CC. Design principles of rock support for underground excavations. In: Eurock 2012. Stockholm, Sweden; 2012.

Li CC. Evaluation of the state of stress in the vicinity of a mine drift through core logging. In: Proceedings of the 4th Asian rock mechanics symposium. New Jersey, USA: World Scientific; 2006a.

Li CC. Rock support design based on the concept of pressure arch. International Journal of Rock Mechanics and Mining Sciences 2006b;43(7):1083–90.

Nomikos PP, Sofianos AI, Tsoutrelis CE. Symmetric wedge in the roof of a tunnel excavated in an inclined stress field. International Journal of Rock Mechanics and Mining Sciences 2002;39(1):59–67.

Nomikos PP, Yiouta-Mitra PV, Sofianos AI. Stability of asymmetric roof wedge under non-symmetric loading. Rock Mechanics and Rock Engineering 2006;39(2):121–9.

Ocak I. Control of surface settlements with umbrella arch method in second stage excavations of Istanbul Metro. Tunnelling and Underground Space Technology 2008;23(6):674–81.

Panek LA. Design for bolting stratified roof. Transactions of the Society of Mining Engineers 1964;229:113–9.

Schach R, Garschol K, Heltzen AM. Rock bolting: a practical handbook. Oxford: Pergamon; 1979.

Schubert W. Recent experience with squeezing rock in Alpine tunnels. In: CUC – rock support in medium to poor rock conditions; 2001.

Shi G, Goodman RE. Key block bolting. In: Rockbolting –- theory and applications in mining and underground construction. Rotterdam: A.A. Balkema; 1983. p. 143–67.

Sinha RS. Rock reinforcement. In: Underground structures – design and instrumentation. Amsterdam: Elsevier; 1989. p. 129–58.

Slade J, Ascott B. Impact of rockburst damage upon a narrow vein gold deposit in the Easter Goldfields, West Australia. In: Challenges in deep and high stress mining. Australian centre for geomechanics; 2007. p. 247–56.

Sofianos AI. Stability of wedges in tunnel roofs. International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts 1986;23(2):119–30.

Statens vegvesen. Fjellbolting. Håndbok 215. Oslo: Trykkpartner AS; 2000 (in Norwegian).

Stillborg B. Professional users handbook for rockbolting. Clausthal-Zellerfeld: Trans Tech Publications; 1994.

Volkmann GM, Schubert W. Effects of pipe umbrella systems on the stability of the working area in weak ground tunneling. In: Sinorock – international symposium on rock characterisation, modelling and engineering design methods. International Society for Rock Mechanics; 2009.

Wright FD. Roof control through beam action and arching. In: SME Mining Engineering Handbook, vol. 1. New York: Society of Mining Engineers; 1973. p. 80–96. Chapter 13.

Zhang C, Feng XT, Zhou H, Qiu S, Wu W. Case histories of four extremely intense rockbursts in deep tunnels. Rock Mechanics and Rock Engineering 2012;45(3):275–88.

Dr. Charlie C. Li is professor of rock mechanics for civil and mining engineering at the Norwegian University of Science and Technology (NTNU) in Norway. Li received his BSc degree in 1981 and MSc degree in 1984, both in geological engineering, in Central South Institute of Mining and Metallurgy (at present Central South University), and his PhD in mining rock mechanics at Lulea University of Technology (LUT), Sweden, in 1993. After that, he was employed as a research associate and then associate professor at LUT until 2000. He worked then in the Kristineberg mine of Boliden Mineral Ltd., Sweden, as a mining engineer for 4 years. He has been the professor of rock mechanics at NTNU since 2004, in charge of the teaching and research program in the subject of rock mechanics as well as the rock mechanics laboratory. He is a member of the Norwegian Academy of Technological Sciences (NTVA). He is the European Vice-President of the International Society for Rock Mechanics (ISRM) for the term of office 2015–2019. Prof. Li's research interests are in rock failure, stability analysis of underground spaces, ground support and application of rock mechanic principles for underground space design. He carried out thorough studies of rock behaviour under compression in laboratory. He made numerous field observations of rock failure and responses of rock support elements in mines and other types of underground excavations. After a thorough study of the performances of rockbolts, he proposed analytical models for the rockbolts currently used in rock engineering practise, which have been acknowledged in the circle of rock mechanics. Based on the models, Li identified the shortcomings of the conventional rockbolts and pointed out that rockbolts, as well as other support elements, must be not only strong and but also deformable, i.e. energy-absorbing, in high in situ rock conditions. He invented a new type of energy-absorbing rockbolt, called D-Bolt in 2006. The D-Bolt is as strong as a fully encapsulated rebar bolt but its deformation capacity is significantly higher than that of the rebar bolt. The D-Bolt is particularly powerful in combating stress-induced rockburst and squeezing. The bolt has been used worldwide in many deep mines and also in hydropower projects, for instance in Sweden, Canada, USA, Chile, Australia and South Africa, to combat instability problems of rockburst. Dr. Li has practical expertise in ground support in difficult rock conditions (for instance, rock squeezing and rockburst), stability analysis of underground caverns and in situ measurements and interpretation. His current research interests are on understanding of rockburst and theories and practise of dynamic rock support.