Full Length Article

Principles of rockbolting design

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ABSTRACT

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 practice 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.

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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.
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 or 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 rock 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 diskling 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
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