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Analytical model for the load transmission law of rock bolt subjected to open and sliding joint displacements

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ABSTRACT

The present study focuses on the load transmission law along the rock bolt and the efficacy for different anchored conditions. Shear displacement along the joint face is considered on the basis of traditional rock bolt stress calculation method with regard to joint open displacement. Combining the elastic foundation beam model and the rock bolt pull-out model, an analytical solution for the stress distribution along the rock bolt through a single joint face is derived. The performance of the proposed analytical solution is compared with an experimental result and a previous study. In addition, joint displacement, joint location, and dip angle of the joint face are set as variables to further explore the corresponding anchored effect. The results indicate that larger joint displacement along the joint face tend to cause a distinct increase of the shear and compressive stress along the rock bolt. When the joint face moves inward, the interaction force between the rock bolt and the surrounding rock mass decreases, suggesting that the strength of the rock bolt is not taken full use. Moreover, for a certain joint displacement and joint location, larger anchored angle between the rock bolt and the joint face contributes to better reinforcement effect.

1. Introduction

Rock bolting is a widely adopted technique to stabilize rock masses around tunnels, underground mining galleries and other civil engineering projects.[1,2] Normally, the reinforcement of rock bolts on the rock mass reduces the yield zones around the tunneling boundary by restricting the deformation of the unstable rock mass.[3] In previous investigations, the main attention was focused on the stress distribution along the bolt subjected to axial load. In recent years, the lateral interaction between rock bolts and the surrounding rock mass have been paid considerable attention due to the progress that have been made in rock bolting axial load transfer mechanism.[4]

The shear behavior of rock bolts and their contribution to joint strength is affected by a series of factors such as the strength of host material,[5] applied axial pretension load ,[6,7] grout conditions of the bolt,[8] joint roughness coefficient (JRC)[9,10] and so on. Plenty of theoretical studies had been conducted previously. Farmer 11 studied the axial behavior of rock bolts under tensile loads. He derived an analytical equation about distribution of the shear stress along the rock bolt, showing that the shear stress decreases from the loading point to the far end of the rock bolt until the bolt-rock interface decoupling occurs. Kamal et al. 12 pointed out that axial load distribution along a fully grouted passive rock bolts shows that a neutral point exists on the bolt rod, in which the shear stress at the interface between the bolt and grouted material vanishes. Indratatna and Kaiser 13 established an analytical model for the design of bolt grout interactions around a circle tunnel in accordance with the elasto-plastic law. By taking advantage of finite difference method (FDM), Brady and Lorig 14 numerically analyzed the interactions between bolt grout and Mohr-Coloumb media, indicating that the radial displacement and yield region decreased due to the installation of grouted bolts around a circular tunnel.

Furthermore, a number of experimental researches had also been done. Grasselli 15 conducted a direct shear test on rock-like material containing two pre-existing parallel joints. The results indicated that the larger intersection angle between the rock bolt and the joint, the smaller shear displacement of the joint face. Jalalifar et al. 16 used five different kinds of rock bolt to conduct double shear experiment, finding that failure occurs due to the induced axial and shear stresses acting between the hinge point distances in the vicinity of the shear joint plane. Cao et al. 17 discussed the failure modes of cable bolting using a bond strength model as well as an iterative method. They introduced the interfacial shear stress model for ribbed bar and a closed form solution is using a tri-line stress strain relationship. Hyett et al. 18 carried out a series of laboratory and field pullout tests to investigate the major

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factors influencing the bond capacity of grouted cable bolts. Aziz and Webb 19 indicated that cable bolt capacity most critically depends on the cement properties, embedment length and radial confinement. This redistribution of forces along the bolt is the result of movement in the rock mass, which transfers the load to the bolt via shear resistance in the grout.

However, the majority of the explorations mentioned above focus mostly on the stress distribution conditions along the rock bolt caused by the joint opening displacement, or the anchored effect considering different joint forms. Yet the mechanism about how the load on the rock bolt is transmitted, due to open and sliding joint displacements, still needs to be studied further. In this investigation, combining the displacement calculating method proposed by Kamal et al. 12 and the infinitesimal analytical method applied by Ma et al. 20 the stress distribution rule along the rock bolt that caused by the shear displacement of the joint face is obtained. In the meanwhile, the bolt drawing model is applied to take the constitutive behavior of the rock mass into account, which makes the analytical results more approximate to the actual state of the studied object.

From the introductions above, it can be seen that previous studies mainly focussed on the analytical stress distribution calculation model along the rock bolt and the anchored effect under different anchored patterns. Thus, combining and extending the theoretical models proposed by previous scholars, analytical stress distribution models of rock bolt for open and sliding joint displacements are derived and verified by the experimental results of the bolts under pullout loads. The experimental investigations and corresponding discussions in this article are divided into three parts. In the first place, joint displacements are set as variables to explore the load transmitting mechanism on the rock bolt. Then location of the joint face is changed to study the variation of the load distribution along the rock bolt on the two sides of the joint face. Finally, a series of joint dip angles are selected to compare the corresponding anchored effect. Hence, the rock bolt anchoring efficacy for different working conditions can be found, which can provide some references for the practical engineering applications.

2. Model for the displacement of rock bolt and rock mass induced by sliding movement of joint face

During the pull-out process of a rock bolt, displacements and deformations of the rock bolt and the surrounding rock mass appear simultaneously. Due to the differences of the strength properties between the two kinds of materials, the concrete variation value may differ distinctively. For instance, an interruption of the deformation exists around the joint face for the rock mass. Whereas for the rock bolt, the deformation is always continuous for the whole length. As a result, interaction force on the rock bolt-rock mass inter face is produced. To obtain the specific interaction force value, strains of rock bolt and rock mass under certain joint displacements are calculated separately.

2.1. Displacement field of the surrounding rock mass

The calculating model is shown in Fig. 1, in which the rock bolt is

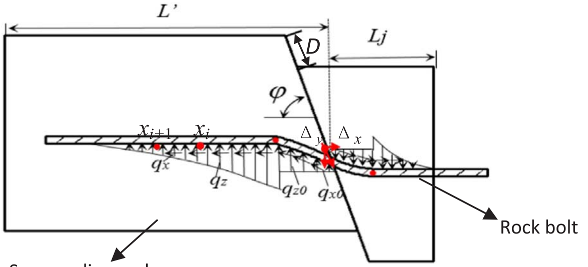

Fig. 1. Simplified model of a rock bolt subjected to displacement of the joint face.

divided into several infinitesimals from the right side of the model to the left end. The dip angle of the joint face is φ, thus the joint displacement D can be expressed by the horizontal displacement Δ_x and vertical displacement Δ_y. In general, the rock displacement decreases monotonically with radial distance x, which is measured from the excavation boundary. Whereas due to the effect of the joint face, a sudden variation of the displacement field is produced. Hence the Heaviside function $H(L_j - x)$ is involved (see Eq. (1)). Therefore, the horizontal and vertical displacement components can be expressed respectively as Eqs. (2) and (3).[12] It should be noted that when calculating the displacement of the rock mass on the outer side of the joint face, due to the effect of H function, an extra open and shear displacement components Δ_x and Δ_y is attached. Whereas for the inner side, the displacement field takes on a normal negative exponential distribution as x increases. Consequently, a step appears at the joint face for the rock mass displacement field. In addition, the meanings of the symbols in this article are as follows. The far field stress is expressed as P, and $n \in (0, 1)$ is a parameter controlling elastic deformation of the rock mass. L_j is the length of the bolt up to at the bolt intersect to the joint plane from the right frontier of the model. r_0 stands for the tunnel radius. Furthermore, Poisson's ratio and elasticity modulus of the rock mass are ν_r and E_r respectively. λ describes the uniformity coefficient of the initial surrounding rock stress. $G_r = \frac{E_r}{2(1+\nu_r)}$ represents the shear modulus of the rock mass.

$$H(L_j - x) = \begin{cases} 1 \ \text{if}(L_j - x) > 0 \\ 0 \text{if}(L_j - x) \le 0 \end{cases} \tag{1}$$

$$U_r = U_{r0} e^{-nx} + \Delta_x H(L_j - x) \tag{2}$$

$$U_\theta = U_{\theta 0} e^{-nx} + \Delta_y H(L_j - x) \tag{3}$$

$$U_{r0} = \frac{P r_0 (1+\nu_r)}{4G_r}[(1+\lambda) + (1-\lambda)(3-4\nu_r)] \tag{4}$$

$$U_{\theta 0} = \frac{P r_0}{4G_r}(1-\lambda)(3-4\nu_r) \tag{5}$$

where U_r and U_θ are the horizontal and vertical displacement of the rock mass in case of a single joint plane. U_{r0} and $U_{\theta 0}$ defined in Eqs. (4), (5) are the total deformation of the excavation wall, which will change depending on the far field stress and rock properties.[25] It should be noted that when calculating the displacement of the rock mass on the outer side of the joint face, due to the effect of H function, an extra open and shear displacement components Δ_x and Δ_y is attached. Whereas for the inner side, the displacement field take on a normal negative exponential distribution as x increases. Consequently, a step appears at the joint face for the rock mass displacement field.

2.2. Displacement field of the rock bolt

The displacement of the rock bolt is also divided into U_b (in the horizontal direction) and U_t (in the vertical direction). Fig. 2(a) presents the stress condition for an elementary of the rock bolt-rock mass structure. $\sigma_b(x)$ is the rock bolt axial stress, $d\sigma_b(x)$ is the axial stress increment, $F(x)$ is the shear stress on the interface between the rock mass and rock bolt. The relationship of the parameters above can be expressed as

$$(\sigma_b(x) + d\sigma_b(x) - \sigma_b(x))A_b = -F(x) \cdot dx \tag{6a}$$

$$\frac{d\sigma_b(x)}{dx} = -\frac{F(x)}{A_b} \tag{6b}$$

where A_b is the cross-sectional area of the rock bolt. In this study, the rock bolt is assumed to be in the elastic deformation state, and its stress-strain relationship is

$$\sigma_b(x) = E_b \cdot \varepsilon_b = E_b \cdot \frac{dU_b(x)}{dx} \tag{7}$$