Journal of Rock Mechanics and Geotechnical Engineering 8 (2016) 164–169

Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full length article

Periodical zonal character of damage near the openings in highly-stressed rock mass conditions

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ARTICLE INFO

Article history:
Received 2 July 2015
Received in revised form
8 September 2015
Accepted 28 September 2015
Available online 1 December 2015

Keywords:
Rock failure
Openings
Zones
Modeling

ABSTRACT

Rock mass damage at great depths near underground openings is often of a zonal character. However, the classical elastoplastic theory fails to explain sufficiently all properties of zonal failure structures. A new non-Euclidean mathematical model for highly-stressed rock mass was developed based on the principles of mechanics of defected material and non-equilibrium thermodynamics. Methods were developed to determine model parameters that provide satisfactory correspondence between the experimental findings concerning faulted zonal structures near openings at great depths and mathematical calculations. The mechanism of this phenomenon was discovered which consisted in a periodical character of stresses in the surrounding rock mass and development of tensile macrocracks at zones of maximal tangential stresses. Main relationships between the cracking zone width and rock mass strength were established.

1. Introduction

Failure conditions can be observed in boundary areas of openings in highly-stressed rock mass during mining and drilling operations. In some cases the failure is of zonal character with zones of tensile macrocracks alternating with relatively monolithic rock mass (Adams and Jager, 1980; Shemyakin et al., 1986).

Many attempts were made to describe the zonal character of rock mass failure near openings based on classical mechanics (Shemyakin et al., 1987; Odintsev, 1994, 1996; Reva and Tropp, 1995; Metlov et al., 2002; Li et al., 2006a,b; Pan et al., 2007; Wu et al., 2009a,b; Jia and Zhu, 2015). However, none of the theories could explain all properties of the zonal failure structures without introducing new assumptions in every new case.

There were efforts to describe the main relationships between zonal disintegration within the dislocation model and the theory of the strain energy density factor (Zhou et al., 2008) based on the principle of stress superposition and Chebyshev polynomials expansion of the pseudo-traction with numerical calculations (Zhou et al., 2009), numerical model of growth and coalescence of cracks within rock mass with the weak element adaptation (Qian et al., 2009), gradient theories of elastoplastic solids (Qi et al., 2011; Wang et al., 2012), and numerical models of defected rocks around an excavation under the action of a slowly unloaded P-wave (Zhu et al., 2014). However, they all failed to quantify zonal failure structures.

A new gage theory was recently applied to solids to describe the whirl fields of plasticity under high-energy conditions (Kadic and Edelen, 1983; Panin, 1990; Panin et al., 1990). The main principle of the gage theory is the incompatibility of deformations in damaged solids. We were the first to apply this principle to the zonal failure phenomenon in rock mass near openings (Guzev and Paroshin, 2001). The approach involving the non-Euclidean model was used in some work later to study the problem of zonal disintegration (Qian and Zhou, 2011; Zhou et al., 2011, 2012, 2013), but no methods for calculating model parameters were proposed and no correlations between theoretical and experimental findings were demonstrated. In this paper, we demonstrate an example to describe the phenomenon using the non-Euclidean mathematical model. The description is based on the concept of rock mass hierarchical block system (Sadovsky, 1979; Makarov, 2004; Xu, 2009) with the rock sample taken as the first level and the rock mass on the scale of the opening as the second level of the hierarchical system.

2. Mathematical model

Rock at great depths is modeled by a faulted structure, which is far from the state of thermodynamical equilibrium due to the damage accumulation. In addition, it is subjected to prolonged

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Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.
1674-7755
http://dx.doi.org/10.1016/j.jrmge.2015.09.010

compressive stresses at infinity. The boundary-value problem is formulated by determining the stress state of a weightless solid plane with damage. The stresses prescribed at infinity model a gravity field. The plate contains a round hole that models an unsupported underground opening (Fig. 1). Due to the polar symmetry of the problem, the equilibrium equations are written as follows:

$$\frac{\partial\sigma_{rr}}{\partial r}+\frac{1}{r}\left(\sigma_{rr}-\sigma_{\varphi\varphi}\right) = 0,\ \sigma_{r\varphi} = 0 \quad (r_0 \le r < \infty) \tag{1}$$

where r is the distance from the center of the opening to the selected point in the rock mass, σ_{rr} is the normal radial stress, $\sigma_{\varphi\varphi}$ is the normal tangential stress, and $\sigma_{r\varphi}$ is the shear stress.

At the boundary of the opening ($r = r_0$) and at infinity, the following stresses are applied:

$$\sigma_{rr} = 0\ (r = r_0); \quad \sigma_{rr}, \sigma_{\varphi\varphi} \to \sigma_\infty\ (r \to \infty) \tag{2}$$

where $\sigma_\infty = \gamma_r H$, γ_r is the unit weight of rock (kN/m^3), and H is the opening depth (m).

The rock mass at great depth is modeled by the material where the conditions of deformation compatibility ε_{ij} are not met commonly:

$$R = \frac{\partial^2\varepsilon_{11}}{\partial x_2^2}-2\frac{\partial^2\varepsilon_{12}}{\partial x_1\partial x_2}+\frac{\partial^2\varepsilon_{22}}{\partial x_{11}^2}\neq 0 \tag{3}$$

The damage parameter R is expressed by the following equation (Guzev and Paroshin, 2001):

$$\Delta^2 R-\gamma^2 R = 0 \tag{4}$$

where Δ is the Laplace operator and γ is the model parameter.

As the problem is plane- and axi-symmetrical, Eq. (4) in the polar coordinates can be rewritten as

$$\left(\frac{\partial^2}{\partial r^2}+\frac{1}{r}\frac{\partial}{\partial r}\right)^2 R = \gamma^2 R \tag{5}$$

The solution to Eq. (5) decreasing at $r \to \infty$ is

$$R(r) = aJ_0(\sqrt{\gamma}r)+bN_0(\sqrt{\gamma}r)+cK_0(\sqrt{\gamma}r) \tag{6}$$

where J_0, N_0, and K_0 are the Bessel, Neumann, and MacDonald functions of zero order, respectively.

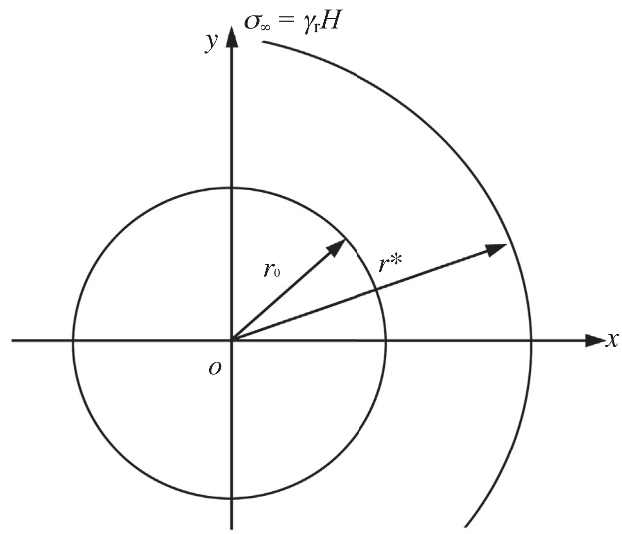

Fig. 1. Design diagram of an unlined opening.

3. Non-classical boundary conditions and problem solution

At the opening boundary, the rock mass undergoes considerable damage; therefore, the damage parameter R should be not equal to zero. Assuming that all failure zones of rock are equivalent and are of the same origin, we introduce the extremum of the function at the boundary $R(r)$ and the following failure zones. Therefore, the boundary conditions for the function $R(r)$ are

$$R'(r)\big|_{r=r_0} = 0,\ R'(r)\big|_{r=r^*} = 0 \tag{7}$$

where r^* is the distance from the opening contour to the middle point of the first failure zone, which has been obtained from experimental data.

The equation for the first invariant of stresses $\sigma = \sigma_{zz}+\sigma_{rr}+\sigma_{\varphi\varphi}$ is written as follows with the determined function R:

$$\Delta\sigma = \frac{E}{2(1-\nu)}R,\ \sigma \to 2(1+\nu)\sigma_\infty, \quad r \to \infty \tag{8}$$

where E is the modulus of elasticity and ν is the Poisson's ratio.

The solution to Eq. (8) gives the equations of stress components:

$$\left.\begin{aligned}
\sigma_{rr} &= \sigma_\infty\left(1-\frac{r_0^2}{r^2}\right)-\frac{E}{2(1-\nu^2)\gamma^{3/2}}\frac{1}{r}[aJ_1(\sqrt{\gamma}r)+bN_1(\sqrt{\gamma}r)+cK_1(\sqrt{\gamma}r)]\\
\sigma_{\varphi\varphi} &= \sigma_\infty\left(1+\frac{r_0^2}{r^2}\right)-\frac{E}{2(1-\nu^2)\gamma}[aJ_0(\sqrt{\gamma}r)+bN_0(\sqrt{\gamma}r)-cK_0(\sqrt{\gamma}r)]+\frac{E}{2(1-\nu^2)\gamma^{3/2}}\frac{1}{r}[aJ_1(\sqrt{\gamma}r)-bN_1(\sqrt{\gamma}r)+cK_1(\sqrt{\gamma}r)]\\
a &= \frac{-c[K_1(\sqrt{\gamma}r_0)Y_1(\sqrt{\gamma}b_1)-Y_1(\sqrt{\gamma}r_0)K_1(\sqrt{\gamma}b_1)]}{J_1(\sqrt{\gamma}r_0)Y_1(\sqrt{\gamma}b_1)-Y_1(\sqrt{\gamma}r_0)J_1(\sqrt{\gamma}b_1)}\\
b &= \frac{-c[J_1(\sqrt{\gamma}r_0)K_1(\sqrt{\gamma}b_1)-K_1(\sqrt{\gamma}r_0)J_1(\sqrt{\gamma}b_1)]}{J_1(\sqrt{\gamma}r_0)Y_1(\sqrt{\gamma}b_1)-Y_1(\sqrt{\gamma}r_0)J_1(\sqrt{\gamma}b_1)}
\end{aligned}\right\} \tag{9}$$