Application of the convergence-confinement method to excavation response of roadway in rock mass considering large deformation

K Guan, W C Zhu and Q Y Zhang

Center for Rock Instability and Seismicity Research, School of Resource and Civil Engineering, Northeastern University, Shenyang, 110819, China
Corresponding Author ORCID: 0000-0001-9436-3504

Abstract. For mining operations in a rock mass with high deformability and low strength, large deformation is one of the primary manifestations of roadway failure, which easily occurs when the construction techniques and support measures cannot accommodate the geological environment. Therefore, the ground response to excavation disturbance and control technology of squeezing behavior should be analyzed based on large deformation theory. This study presents an elasto-plastic theoretical model for a circular roadway in the finite strain framework using a hypoeplastic-plastic theory and convergence-confinement method to provide a rigorous approach for large deformation prediction and weak rock-support interaction analysis. A novel numerical procedure for the strain-softening rock mass subjected to large deformation is proposed based on an existing brittle-plastic analytical formula, which is simple but precise. Furthermore, suggestions on analysis of weak rock-support interaction based on large deformation method are presented, which may be beneficial to understanding deformation prediction and support design more rigorously.

1. Introduction

For deeper mines, hard rock in the shallow ground may present the characteristics of weak rock with large deformation and hard support under the comprehensive action of high ground and complex structural stresses. The contradiction between high stress and low strength is more prominent. A large amount of deformation energy is released after excavation disturbance, causing stress redistribution and local stress concentration. Then, it leads to large deformation disasters, such as crushing expansion, high squeezing, and strong rheology of weak rock. Additionally, different confining pressure and rock temperature levels induce different post-peak characteristics of the rock mass. Under the conditions of high confining pressure and high rock temperature in a deep mine, the deformation behavior of rock mass shows a transition phenomenon from brittleness to ductility, presenting significant permanent plastic deformation during the rock failure process. The convergence-confinement method (CCM) is widely accepted for estimating circular cavity deformation and support capacity. Significant studies based on small strain theory have been presented. These studies focused on the unloading response of a circular cavity by considering different constitutive relationships[1-7]. However, the small strain solutions are perfectly appropriate for good quality rock without significant deformation; it may lead to erroneous and sometimes even absurd results for a high convergence ratio[8, 9]. Therefore, in problems involving high squeezing potential and
big roadway convergence, the large deformation model considering geometric nonlinearity should be highlighted\[10-13\].

The heavily squeezing phenomena in practical engineering can be accurately depicted using finite strain elasto-plasticity theory\[14\]. Thus, the overall elasto-plastic analysis is a Lagrangian method that focuses on the motion of each material point and is well suited for problems involving large deformation. Recently, Vrakas and Anagnostou\[15\] established a large deformation theoretical model within the framework of hypoelastic-plastic theory, in which the rock deformation can be additively decomposed into the elastic and plastic ones, thus leading to a simple but practical analysis method for the ground response curve (GRC) in weak rock. In this analysis, the equilibrium differential equation was built in the context of the actual deformed configuration instead of the undeformed one. This indicates that the difference between the large deformation result and small strain solution is significant, especially for weak rocks under high overburden. Subsequently, large deformation analysis was applied to establish the quantitative relationship between small strain and large strain convergences\[8, 16, 17\], and estimate the elastic deformation modulus for squeezing ground and deal with tunnel reprofiling works to reestablish the desired clearance under squeezing conditions\[18\].

Zhang et al.\[19\] considered the large strain occurs in both elastic and plastic rock masses, and derived the analytical solution for the squeezing response in a circular opening excavated in an elastic-brittle-plastic rock. Zhang et al.\[20\] proposed a large strain numerical procedure for the strain-softening rock mass with rock strength and elastic modulus degraded based on a fifth-order Runge-Kutta method. Xu and Xia\[21\] presented a large strain solution for tunnel excavated in strain-softening rocks conforming to the Generalized Zhang-Zhu strength criterion.

This study aims at developing a simple but precise strain-softening numerical procedure based on finite strain elasto-plasticity theory to provide a convenient estimation of GRC and lay a foundation for determining the interaction process between the support structure and weak rocks, where the large deformation may exist and the classical small strain elasto-plastic analyses are inappropriate.

2. Numerical computation strategy for the strain-softening behavior based on finite strain

A circular roadway is considered under the plane strain and rotationally symmetric conditions, with uniform support pressure $\sigma_0$, exerted on the wall and in-plane hydrostatic stress $\sigma_0$ imposed through the homogeneous domain. As $\sigma_0$ is reduced, the rock mass behaves elastically initially, and a plastic zone is formed once critical yield stress is attained at the excavation boundary. The strain-softening behavior of the material points is examined by adopting the plastic internal variable $\eta$, which controls the evolution process of the strength parameters and post-peak yield surface, according to plastic deformation. This can be written as\[22\]

$$\eta = \varepsilon^p_t - \varepsilon^p_r,$$  \hspace{1cm} (1)

where $\varepsilon^p_t$ and $\varepsilon^p_r$ denote the tangential and radial plastic strain, respectively.

For simplicity, a bilinear softening law in the plastic zone is defined as follows\[23\]

$$\omega(\eta) = \begin{cases} \omega_p - (\omega_p - \omega_r) \frac{\eta}{\eta_c}, & 0 < \eta < \eta_c \\ \omega_r, & \eta \geq \eta_c \end{cases}$$  \hspace{1cm} (2)

where $\omega$ denotes the strength parameters, including cohesion force $c$, friction angle $\phi$, and dilation angle $\psi$. The subscripts $p$ and $r$ denote the variables in the peak and residual state, respectively. The elastic-perfectly plastic, strain-softening, and elastic-brittle-plastic models can be transformed by changing the value of $\eta_c$ from infinity to zero.

The stresses $\sigma_t$ and $\sigma_r$ in the plastic zone follow the Mohr–Coulomb failure criterion:

$$\sigma_t = m(\eta)\sigma_0 + \sigma_0(\eta)$$  \hspace{1cm} (3)

where $m(\eta) = (1 + \sin \phi(\eta))/(1 - \sin \phi(\eta))$, and $\sigma_0(\eta) = (2c(\eta)\cos \phi(\eta))/(1 - \sin \phi(\eta))$.

To simplify the mathematical derivation in the following parts, the dimensionless operation is conducted as follows:

$$\tilde{\sigma} = \frac{1}{E} \left( \sigma + \frac{\sigma_0(0)}{m(0) - 1} \right)$$  \hspace{1cm} (4)

where $E$ is the Young’s modulus.

Considering Eqs. (3) and (4), the yield surface expression in the plastic region is given by
\[ \bar{\sigma}_t - m(\eta)\bar{\sigma}_r = 0. \]  

(5)

The non-associated flow rule is used, thus, the plastic potential function can be written as

\[ G(\bar{\sigma}_t, \bar{\sigma}_r, \eta) = \bar{\sigma}_t - \beta(\eta)\bar{\sigma}_r \]  

(6)

where \( \beta(\eta) = (1 + \sin \psi(\eta))/(1 - \sin \psi(\eta)) \).

Compared with the small strain formulation, the large deformation analysis refers to the deformed geometry instead of the undeformed state. Therefore, the radial displacement is calculated by

\[ u(r) = r_0 - r \]  

(7)

where \( r_0 \) is the initial position of the material point, and \( r \) is the current location of the deformed geometry after the excavation disturbance.

The logarithmic strains are adopted to describe the large deformation in squeezing rock mass.

\[ \varepsilon_r = \ln \left(1 + \frac{du}{dr}\right), \varepsilon_t = \ln \left(1 + \frac{u}{r}\right) \]  

(8)

where \( \varepsilon_r \) and \( \varepsilon_t \) are the radial and tangential strain corresponding to the current geometry state after excavation.

Figure 1. Hierarchical model in the plastic zone

Our previous studies[24-26] are of particular interest to this study. We have presented an analytical solution and numerical computation strategy for the elastic-brittle-plastic and strain-softening models considering the influence of the axial in-situ stress and axial plastic flow. Therefore, in order to obtain the squeezing response in strain-softening rock mass neglecting the out-of-plane stress which is the case in the classical plane strain model, a hierarchical model[25, 26], where the plastic zone is divided into \( n \) concentric annuli, is adopted (Figure 1). The excavation response in each annulus can be characterized by the elastic-brittle-plastic analytical solution in the case of axial stress being the intermediate principal stress[24]. Accounting that the stresses and strains in the elastic zone can be solved conveniently according to Lame’s solutions[24] after the current radius of the deformed roadway and plastic radius \( R_p \) are obtained, thus only the numerical strategy in the plastic zone of the strain-softening rock mass is introduced here.

For convenience, in the programming and computation of the mathematical derivation, the normalized radius \( \rho \) and normalized radial displacement \( U \) are defined by

\[ \rho = \frac{r}{R_p}, U = \frac{u}{R_p} \]  

(9)

and the logarithmic strains in Eq. (8) are converted into

\[ \varepsilon_r = \ln \left(1 + \frac{du}{d\rho}\right), \varepsilon_t = \ln \left(1 + \frac{U}{\rho}\right), \]  

(10)

where \( \rho \) and \( U \) represent the current (deformed) state after excavation.

The stress and displacement at the elastic-plastic interface (\( \rho_{(j)} = 1 \)) are equal to the initial values of \( \sigma_{(1)} \) \( (\sigma_r(1) = \sigma_{R_p} = (2\sigma_0 - \sigma_{Dp})/(m_p + 1), \sigma_t(1) = m_p\sigma_r(1) + \sigma_{Dp}) \), and \( U_{(1)} \). For the \( j \)th annulus, the outer and inner radius will be denoted by \( \rho_{(j)} \) and \( \rho_{(j+1)} \), respectively. It is worth noting that analogous to the brittle-plastic analysis by Guan et al.[23], the rock behavior over \([\rho_{(j)}, \rho_{(j+1)}]\) can be determined analytically by substituting the outer critical radial stress \( \sigma_{R_p} \) by \( \sigma_{(j)} \) and the internal pressure \( \sigma_a \) by \( \sigma_{(j+1)} \). The constant radial stress increment \( \Delta \sigma_r \) for each annulus is taken as

\[ \Delta \sigma_r = \frac{\sigma_a - \sigma_{R_p}}{n} \]  

(11)
and thus
\[ \sigma_{r(j+1)} = \sigma_{r(j)} + \Delta \sigma_r \]  

If \( n \) is sufficiently large, the radial and tangential stress within the \( j \)th annulus can be given in a way similar to the brittle-plastic analytical expression of Guan et al.\[24\]
\[ \bar{\sigma}_r = \bar{\sigma}_{r(j+1)} \left( \frac{\rho}{\rho_{(j+1)}} \right)^{m_{(j)}-1} \]
\[ \bar{\sigma}_t = m_{(j)} \bar{\sigma}_r = m_{(j)} \bar{\sigma}_{r(j+1)} \left( \frac{\rho}{\rho_{(j+1)}} \right)^{m_{(j)}-1} \]

where \( \bar{\sigma} \) is obtained from dimensionless operation Eq. (4) with strength parameters corresponding to those at \( \rho = \rho_{(j)} \).

When \( \rho = \rho_{(j)} \), Eq. (13) yields the position at \( \rho = \rho_{(j+1)} \):
\[ \rho_{(j+1)} = \rho_{(j)} \left( \frac{\sigma_{r(j)}}{\bar{\sigma}_{r(j+1)}} \right)^{\frac{1}{m_{(j)}-1}} \]

Combined with the non-associated flow rule, generalized Hooke’s law, and stress expression in the plastic zone, we can obtain
\[ \varepsilon_r + \beta_{(j)} \varepsilon_t = \ln (\Omega_1) + \Omega_2 \left( \frac{\rho}{\rho_{(j+1)}} \right)^{m_{(j)}-1} \]

where \( \Omega_1 = \exp \left[ (1 + \mu) (1 - 2\mu) (1 + \beta_{(j)}) \bar{\sigma}_0 \right] \) and \( \Omega_2 = (1 + \mu) (1 - \mu - \mu \beta_{(j)} + m_{(j)} \mu \beta_{(j)} - m_{(j)} \mu \beta_{(j)} - m_{(j)} \mu \bar{\sigma}_0) \).

Substituting Eq. (10) into Eq. (16), the normalized radial displacement \( U_{(j+1)} \) at \( \rho = \rho_{(j+1)} \) can be determined; thus, the tangential strain \( \delta_{(j+1)} \) is obtained using Eq. (10)
\[ U_{(j+1)} = \rho_{(j+1)} \left\{ \left[ C_{12} - \delta_{(j)} \cdot \Omega_1 f (T(\rho_{(j)}), T(\rho_{(j)})) \right]^{\frac{1}{m_{(j)}-1}} - 1 \right\} \]

The radial strain can be obtained by combining Eq. (16) and Eq. (21) for \( \rho = \rho_{(j+1)} \) as follows:
\[ \varepsilon_{r(j+1)} = \ln (\Omega_1) + \Omega_2 - \beta_{(j)} \varepsilon_t(j+1) \]

According to the generalized Hooke’s law, the radial and tangential elastic strain at \( \rho = \rho_{(j+1)} \) can be given by
\[ \varepsilon_{r(j+1)}^e = (1 + \mu) \left[ (1 - \mu - \mu m_{(j)}) \bar{\sigma}_{r(j+1)} + (2\mu - 1) \bar{\sigma}_0 \right] \]
\[ \varepsilon_{t(j+1)}^e = (1 + \mu) \left[ (m_{(j)} - \mu - \mu m_{(j)}) \bar{\sigma}_{r(j+1)} + (2\mu - 1) \bar{\sigma}_0 \right] \]

Decomposing the total strain into plastic and elastic, the strain-softening variable \( \eta \) in Eq. (1) is given as follows:
\[ \eta = (\varepsilon_t - \varepsilon_t^e) - (\varepsilon_r - \varepsilon_r^e) \]

Thus, we obtain the internal variable at \( \rho = \rho_{(j+1)} \) as follows:
\[ \eta_{(j+1)} = \left( \varepsilon_t(j+1) - \varepsilon_t^e(j+1) \right) - \left( \varepsilon_r(j+1) - \varepsilon_r^e(j+1) \right) \].
After obtaining strain-softening variable $\eta_{(j+1)}$, the strength parameters at $\rho = \rho_{(j+1)}$ can be updated by Eq. (2).

When $j = n$, the material point $\rho_{(n+1)}$ is located at the opening wall ($r = a$) due to the pre-set relation for the radial stress increment (Eq. (11)). Considering the radial displacement definition Eq. (7) with $r = a$, we obtain the following expression:

$$\frac{U_{(n+1)} + \rho_{(n+1)}}{\rho_{(n+1)}} = \frac{u_a R_p + a / R_p}{a / R_p} = \frac{a_0}{a}$$

which leads to the current radius of the opening:

$$a = a_0 \frac{U_{(n+1)} + \rho_{(n+1)}}{\rho_{(n+1)}}$$

where $\rho_{(n+1)}$ and $U_{(n+1)}$ are calculated using Eqs. (15) and (20), respectively.

Setting $\rho = \rho_{(n+1)}$ and $r = a$, and considering the normalized expression Eq. (9), the plastic radius can be determined as follows:

$$R_p = \frac{a}{\rho_{(n+1)}}$$

The radial displacement at each material point can be obtained as follows:

$$u_{(j+1)} = R_p U_{(j+1)}.$$ 

This leads to the wall convergence displacement $u_a = U_{(n+1)} = R_p U_{(n+1)}$.

As stated above, the response in the elastic zone can be acquired on the basis of Lame’s solutions[24] once the current radius $a$ and plastic radius $R_p$ are computed. The numerical implementation is similar to our previous study[25], with only one plastic zone considered and the axial in-situ stress being $2\mu \sigma_0$.

3. Validation of the numerical procedure

To confirm the validity of the proposed numerical method for small deformation, the hard rock parameters of Ogawa and Lo[27] are adopted with $a_0 = 1$ m, $\sigma_0 = 1$ MPa, $\sigma_a = 0$, $E = 50$ GPa, $\mu = 0.2$, $c_p = 0.173$ MPa, $c_r = 0.061$ MPa, $\varphi_p = 55^\circ$, $\varphi_r = 52^\circ$, and $\psi = 0$. Figure 2 shows results of the comparison between the proposed method and infinitesimal strain analytical solution[23, 28] when $n = 300$, indicating perfect agreement for the elastic-brittle-plastic and elastic-perfectly plastic models.
To better understand the role of this method in depicting the large deformation problem, the derived numerical solutions are compared to Zhang et al. [21], who proposed a large deformation calculation procedure with a five-order Runge-Kutta finite difference method based on finite strain theory. The parameters are given as follows: a0 = 1.0 m, \( \eta_c = 0.15 \), \( \sigma_0 = 1 \) MPa, \( E = 30 \) MPa, \( \mu = 0.3 \), \( c_p = 0.2 \) MPa, \( \sigma_r = 0.02 \) MPa, \( \theta_p = 40^\circ \), \( \theta_r = 20^\circ \), \( \theta_p = 10^\circ \), and \( \theta_r = 5^\circ \). As shown in Figure 3, GRC in the squeezing rock mass can be well predicted using the proposed numerical method, especially for a high convergence ratio (ua/a0 > 10%). Therefore, identical comparison results confirm the capability of the proposed numerical procedure in characterizing the responses of rock subjected to either a slight deformation or heavy squeezing conditions. Additionally, the ground deformation based on the conventional small strain method remains adequately accurate when ua/a0 < 10%, but it is overestimated for rock mass with high squeezing potential, indicating the importance of adopting the large deformation method in this case.

**Figure 2.** Comparison with small strain analytical solution for hard rock with (a) elasto-brittle-plastic behavior, (b) elastic-perfectly plastic behavior, and (c) strain-softening behavior

**Figure 3.** Comparison with the results by finite strain theory in strain-softening rock mass

**4. Discussion on weak rock-support interaction**

Based on the equivalent continuum mechanics, CCM is the most used method in analyzing the interaction between the surrounding rock and support. This method focuses on the elastic-plastic behavior of rock stratum under the action of the support structure. It considers the spatial effect of roadway face, self-supporting effect of surrounding rock, and support time in the roadway excavation process. CCM consists of GRC, support reaction curve (SRC), and longitudinal displacement profile (LDP). GRC describes the relationship between the increasing convergence of the roadway and the decreasing support pressure under the plane strain state. SRC represents the resistance caused by the deformation of the support structure, and LDP reflects the wall convergence at any longitudinal position of the roadway due to excavation disturbance. Thus, the coupling study of the above three
curves can provide a theoretical basis for designing support structure parameters and analyzing surrounding rock-support interaction.

Alejano[22, 29] and Vlachopoulos[30, 31] pointed out that reasonable construction of LDP directly affects the accurate design of support time and determines the practicability of CCM in engineering. Many scholars have established the LDP expressions under different rock mass conditions. In particular, the LDP proposed by Vlachopoulos et al.[32] is widely used. However, LDPs are generally established by theoretical analysis and numerical simulation based on the small strain method, thus leading to the necessity to reconstruct LDP and fictitious support pressure reflecting the spatial effect of advancing face for weak rock with high deformability and significant geometric nonlinear characteristic[17]. On the other hand, as the roadway face advances, rock strength gradually degrades, fictitious support pressure ρ decreases, and support pressure σa increases (see Figure 4). Therefore, the entire weak rock-support interaction process should be investigated comprehensively, considering the spatial effect of face and strain-softening behavior of rock mass in the deformed excavated structure in the large deformation method framework.

5. Conclusion
This study presented a numerical procedure for strain-softening rock mass with large deformation based on finite strain, accounting for the unloading response in the deformed roadway profile. The behavior of the strain-softening at each material point is computed conveniently using an existing brittle-plastic analytical formula. Thus, the stresses and deformation of the material point can be given in the closed form, enabling the procedure to be implemented exactly and straightforwardly. It is verified that the proposed numerical procedure can characterize the responses of rock subjected to either a slight deformation or heavy squeezing conditions. Finally, some suggestions on analyzing weak rock-support interaction based on the large deformation method are presented.

6. References
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