Deformation Prediction and Effect Evaluation of Various Reinforcing Method of Tunnel by Considering Time Dependency of Rock Mass Strength

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1 Introduction

In Japan, most of road tunnels and railway tunnels are constructed in the natural ground covered by soft rocks. When analyzing the deformatonal behaviors of these rock tunnels, the time-dependent characters of soft rock such as deterioration of strength according to the passage of time have to be considered. In the surrounding ground of a rock tunnel, plastic zones can be generated during excavation and they can expand along with time after the secondary lining is constructed due to the change of the mechanical behaviors of the ground, producing extra pressure and inducing damages like cracking and spalling on lining. Another factor that causes lining damage is regarded to the deterioration of lining material along with time. Reinforcements against the deformation in tunnels are essential when damages and deformation proceed. In the design stage, the deformatonal behaviors of lining and rocks surrounding lining need to be adequately assessed to accurately predict the long-term performance of the tunnel and to choose necessary measures once damages happen. Appropriate mechanical models of natural ground and lining, which can represent the time-dependent features of strength deterioration, named rheology generally, are required to accomplish an adequate assessment.

The rheological characteristics of some types of rock have been comprehensively studied through a great deal of specified creep, relaxation and quasistatic compression tests\textsuperscript{[1-3]}. According to the results from laboratory (or in-site) tests and the experience from engineering practice, many rheological models have been proposed to account for the time-dependent features of rock mass from manifolds. These models can be generally divided into two categories: the classic viscoplastic models and the viscoplastic-damaged models. The constitutive laws in the classic viscoplastic models try to relate the current strain rate to the current stress (and/or stress rate) directly. Particularly, the relationship between the deviatoric strain rate and the deviatoric stress (and/or stress rate) can be schematically represented by a series of spring, dashpot and plastic slider that connected in parallel and/or in series. This category of rheological model is represented by the Burger-MC model, the Bingham model, the power law MC model etc.\textsuperscript{[4, 5]}

Based on the classic Burger-MC model, a Burger-Deterioration model is proposed in this paper. Instead of the complicated damage mechanics, a deterioration threshold and two deterioration ratios are introduced in this model to consider the time-dependent strength deterioration aspect of rock equivalently. The proposed model is implemented in the numerical codes FLAC3D, and is applied to an engineering instance (Ureshino Tunnel Line I, Nagasaki Expressway) to account for the delayed deformations that occurred after its completion since Nov. 1992. The proposed numerical model was then used to assess the effects of various reinforcing methods by comparing their deformatonal behaviors.

2 The Burger-Deterioration Model

2.1 The classic Burger-MC model
Fig. 1. Schematical representation of the deviatoric behavior of Burger-MC rheological model.

The constitutive laws of the classic Burger-MC model are characterized by an elastoplastic volumetric behavior and a viscoplastic deviatoric behavior. The deviatoric behavior can be schematically illustrated in Fig. 1, where a Kelvin unit characterized by its shear modulus \( G^K \) and viscosity \( \eta^K \), a Maxwell unit characterized by its shear modulus \( G^M \) and viscosity \( \eta^M \) and a Mohr-Coulomb plastic unit characterized by its cohesion \( c \), friction angle \( \phi \) and dilation angle \( \psi \) are connected in series and subjected to a certain deviatoric loading jointly. Consequently, the deviatoric strain rate partitioning is formulated as Eq. (1). And the constitutive laws of the deviatoric behavior for these three units are formulated as Eqs. (2)-(4), respectively, while the constitutive laws of the volumetric behavior are formulated as Eq. (5).

\[
\begin{align*}
\dot{\varepsilon}_{ij} &= \dot{\varepsilon}_{ij}^K + \dot{\varepsilon}_{ij}^M + \dot{\varepsilon}_{ij}^P \\
\dot{s}_{ij} &= 2\eta^K \dot{\varepsilon}_{ij}^K + 2G^K \varepsilon_{ij}^K \\
\dot{\varepsilon}_{ij}^M &= \frac{\dot{s}_{ij}}{2G^M} + \frac{s_{ij}}{2\eta^M} \\
\dot{\varepsilon}_{ij}^P &= \lambda \left( \frac{\partial g}{\partial \varepsilon_{ij}} - \frac{\dot{\varepsilon}_{kk}^P}{3} \delta_{ij} \right) \text{ with } \dot{\varepsilon}_{kk}^P = \lambda \left[ \frac{\partial g}{\partial \varepsilon_{11}} + \frac{\partial g}{\partial \varepsilon_{22}} + \frac{\partial g}{\partial \varepsilon_{33}} \right] \\
\dot{\sigma}_{kk} &= 3K (\dot{\varepsilon}_{kk} - \dot{\varepsilon}_{kk}^P)
\end{align*}
\]

where \( \varepsilon_{ij} \) and \( s_{ij} \) are the deviatoric components derived from the strain tensor and the stress tensor, respectively; \( \varepsilon_{kk} \) and \( \sigma_{kk} \) are the volumetric components of the strain and stress tensors. The superscripts \( K, M \) and \( P \) denote the Kelvin, Maxwell and MC plastic components of the corresponding variables. \( g \) is the plastic potential for MC unit, and \( \lambda \) is a multiplier that can be eliminated in the calculation afterwards. \( G \) and \( K \) are the shear modulus and the bulk modulus.

On the other hand, the stress state should be enveloped by a failure criterion. For MC unit, the failure criterion \( f \) and the plastic potential \( g \) generally can be expressed as

\[
\begin{align*}
f &= \sigma_1 - \sigma_3 \left( \frac{1 + \sin \phi}{1 - \sin \phi} + 2c \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} \right) \\
g &= \sigma_1 - \sigma_3 \frac{1 + \sin \psi}{1 - \sin \psi}
\end{align*}
\]

where \( \sigma_1 \) and \( \sigma_3 \) are the major and the minor principal stresses; \( c, \phi \) and \( \psi \) are the cohesion, the friction angle and the dilation angle of MC unit.
2.2 The Burger-Deterioration model

Some types of rock specimens would experience three different phases in some creep compression tests, if the constant stress applied on the specimens is larger than a threshold (the so-called long-term strength). The strain rate will attenuate in primary phase, stabilize in secondary phase and accelerate in tertiary phase, which lead to a delayed failure finally. The viscoplastic-damaged models incorporate the damage mechanics with the viscoplastic mechanics to account for the tertiary creep phase phenomenon. However, there is not a well-recognized one among the researchers, since the damage mechanics would differ from each other due to different types of rock mass (such as brittle rock and ductily rock).

Instead of introducing the complicated damage mechanics, a Burger-Deterioration rheological model is proposed in this paper. Its framework is the same with the Burger-MC model and it just simply assumes that the cohesion \(c\) and the friction angle \(\phi\) will decrease with time, regardless of whether the loss of strength is caused by cycle loading fatigue, by clay mineral hydration or by some other reasons. It is assumed that the loss of strength is controlled by its current stress state; furthermore, there exists a threshold to initiate this kind of strength deterioration and a lower limit to circumscribe the strength deterioration

\[
\frac{dc}{dt} = -\omega_c R \quad (R \geq R_{thr}, \quad c \geq c_{res}) \tag{8-a}
\]

\[
\frac{d\phi}{dt} = -\omega_\phi R \quad (R \geq R_{thr}, \quad \phi \geq \phi_{res}) \tag{8-b}
\]

\[
R = \frac{\sigma_1 - \sigma_3}{2c \cos \phi + (\sigma_1 + \sigma_3) \sin \phi} \tag{8-c}
\]

The parameter \(R\) is named stress coefficient in this paper, which indicates the “distance” from the current stress state to the MC failure envelope. When the stress coefficient is greater than a certain threshold \(R_{thr}\), the rock strength initiates to deteriorate. The multipliers \(\omega_c\) and \(\omega_\phi\) are deterioration ratios that scale the increments of \(c\) and \(\phi\) by some certain proportions. The Eqs (8) that describe the constitutive laws of the Burger-Deterioration model can be implemented in the numerical codes FLAC3D.

3 Delayed deformation analyses

3.1 Outline of the Ureshino tunnel

The proposed Burger-Deterioration model is applied to an engineering instance (Ureshino Tunnel Line I) to account for the delayed deformations that occurred after its completion since Nov. 1992. The construction of Ureshino Tunnel Line I in Nagasaki Expressway began from May 1990 and finished in Nov. 1992. The typical geometrical dimension of cross section is schematically illustrated in Fig.2. The tunnel convergence monitoring positions, mainly including the convergence at the crown \((u_c)\), the springline \((u_s)\) and the invert \((u_i)\), are also denoted by the triangles in this figure.

In the longitudinal direction, the span of each excavation cycle was 1.0m. The shotcrete lining and the rock bolt were installed immediately after each excavation cycle, and the second lining was cast in place after 25 excavation cycles completed. In this paper, a total of 30 excavation cycles (from SAT 170+00.0 to SAT 200+00.0) are simulated step by step. The properties of the rock mass, as listed in Table 1, are referenced to the construction and monitoring reports on Ureshino Tunnel (Japan Highway Society, 2000). Notice that at the excavation stage, the rheological calculation switch is set off, since the elastoplasticity is the overwhelming mechanics rather than the rheology. Thus, the six rheological parameters are unnecessary at this stage.

3.2 Delayed deformation characteristics
Fig. 2. The cross-section dimensions and the convergence monitoring positions.

Table 1. The properties of the rock mass employed in numerical simulations.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho$ (kg/m³)</td>
<td>2300</td>
<td>$c$ (MPa)</td>
<td>0.577</td>
</tr>
<tr>
<td>$K$ (MPa)</td>
<td>833</td>
<td>$\phi$ (°)</td>
<td>30.0</td>
</tr>
<tr>
<td>$G$ (MPa)</td>
<td>N/A</td>
<td>$\psi$ (°)</td>
<td>5.1</td>
</tr>
<tr>
<td>$\eta_u$ (Pa·s)</td>
<td>N/A</td>
<td>$\omega_u$ (MPa/y)</td>
<td>N/A</td>
</tr>
<tr>
<td>$G''$ (MPa)</td>
<td>385</td>
<td>$\omega_p$ (°/y)</td>
<td>N/A</td>
</tr>
<tr>
<td>$\eta''$ (Pa·s)</td>
<td>N/A</td>
<td>$R_{dir}$</td>
<td>N/A</td>
</tr>
<tr>
<td>In-situ $\sigma_{zz}$ (MPa)</td>
<td>8.0</td>
<td>In-situ $\sigma_{xx}$ (MPa)</td>
<td>8.0</td>
</tr>
</tbody>
</table>

The monitoring on the tunnel convergence began from 1990 when Line I commenced working, and lasted out until 2000 when the construction of Ureshino Tunnel Line II was completed. It is found that Ureshino Tunnel Line I experienced a continuous converging even after it was put into use, which was close related to the rheological features of rock mass. Setting the rheological calculation switch on and using the proposed rheological model, the numerical simulations aforementioned proceed to rheological analyses focusing on the period from Nov. 1992 to Nov. 1997.

Firstly, by setting the related rheological parameters to some extreme values (i.e. infinite or zero), the proposed model can be simplified into Kelvin-MC model, Maxwell-MC model and deteriorated MC model, respectively. The deformations calculated by these three models, as schematically illustrated in Figure 3, are featured by exponential, linearly and unstably increases in both $u_c$ and $u_i$, respectively. In fact, the deformation characteristics of the Kelvin section, the Maxwell section and the deteriorated MC section correspond to the first, the second and the third phases in some rock specimens creep tests, where the strain rate attenuates, stabilizes and accelerates respectively. Meanwhile, the deformation characteristics of the Burger-Deterioration model, which combines these three sections jointly, are illustrated in Fig. 4.
The crown settlement increases exponential, which indicates that it is dominated by the viscoplastic mechanics. On the other hand, two comparatively mechanics, the viscoplasticity and the strength deterioration, influence the invert upheaval jointly, which leads to a stair-shaped deformation characteristic. It is the stair-shaped deformation characteristic that agrees with the monitoring data (the dotted-line in Fig. 5) qualitatively, which implies the suitability of the proposed model to account for the delayed deformation occurred in Ureshino Tunnel Line I after its completion.

3.3 Parameter identification

Although the proposed model is able to account the delayed deformation mechanics in Ureshino Tunnel Line I qualitatively, another problem arises that how to evaluate the corresponding parameters involved in this model more realistic, since these rheological parameters usually are not ready for a certain engineering instance. In this study, a total of 40 cases with different rheological properties are studied by the same numerical simulations. The properties are taken in such a random way that they are normally distributed with the expected value and deviation referenced to the lab tests of rock specimens (Japan Highway Society, 2000). Comparing the simulation results with the in-site monitoring data, the errors for the crown settlement and the invert upheaval in each case are defined in Eq. (9). The weight for each case, \( w_j \), and the weighted average of the rheological properties can be obtained via Eq. (10).

\[
e_{c, j} = \frac{u_{c, j} - u_{c, m}}{u_{c, m}} \quad (1 \leq j \leq 40) \quad (a)
\]

\[
e_{u, j} = \frac{u_{u, j} - u_{u, m}}{u_{u, m}} \quad (1 \leq j \leq 40) \quad (b)
\]

\[
w = \left( \frac{2}{e_u + e_v} \right) \sum_{j=1}^{40} \left( \frac{2}{e_{c, j} + e_{u, j}} \right) \quad (1 \leq j \leq 40) \quad (a)
\]

\[
a = \sum_{j=1}^{40} w_j a_j \quad (1 \leq j \leq 40) \quad (b)
\]

where \( u_{c, j} \) (\( u_{j} \)) and \( u_{c, m} \) (\( u_{m} \)) are 60×1 vectors recording the crown settlements (the invert upheavals) computed from jth case and the monitoring data during sixty months from Nov. 1992. \( \| \| \) is the denotation of norm calculation for vectors. As delineated in Fig. 5, the simulation results agree with the monitoring data well and the errors of \( u_c \) and \( u_u \) fall below an acceptable range. The proposed Burger-Deterioration model, as well as its corresponding parameters is proven to be able to account for the delayed deformation mechanics in Ureshino Tunnel Line I after its completion, and can be used to predict the further deformation of rock mass and help the maintenance of tunnel in the future.

4 Effect evaluation of various reinforcing method

4.1 Reinforcing methods for tunnel lining

Based on the aforementioned numerical model, five kinds of reinforcing methods usually adopted in field constructions were numerically studied to assess their reinforcing effects. Those are inner lining method, carbon fiber sheet bonding method, steel plate bonding method, rock bolt method and backfilling method. The former four methods are usually used in the situations that crack or spalling occurred on lining, and the last method is conducted when void spaces (cavity) occurred behind lining. The inner lining method, carbon fiber sheet bonding method and steel plate bonding method were modeled as shell elements on the surface of lining in the numerical simulations as shown in Fig. 6, while the rock bolts were treated as cable element as shown in Fig. 2. An element with a thickness of 5cm was firstly treated as null element to represent the cavity, then it was backfilled by grouting materials in the analyses of backfilling method. The elastic modulus and Poisson’s ratio of these elements are: Inner lining \( E=3.10e4\) (MPa), \( \nu=0.2 \); Carbon fiber sheet \( E=2.30e5\) (MPa), \( \nu=0.3 \); Steel plate \( E=2.10e5\) (MPa), \( \nu=0.3 \); Rock bolt \( E=3.10e4\) (MPa); Backfilling \( E=5.00e3\) (MPa), \( \nu=0.4 \). The reinforcing measures can be constructed at different periods after a tunnel is put into service, which can be several months or several years, and the effects of the reinforcements can be different. In this study, a number of cases were numerically investigated, in which the reinforcing measures were constructed 2 years, 3 years and 4 years after the completion of tunnel. The characteristics of these reinforcing methods are depicted as follows.
(1) Inner lining method

In the Inner lining method, new inner lining is constructed as a substitution of the lining concrete when the lining is remarkably deteriorated or damaged. It is also adopted as water leakage and frost damage measure. The inner lining is usually constructed by using sprayed concrete, however, the engineering practices have shown that the bonding between the spayed concrete and the lining concrete can be the weak points of this method, and spalling happened in some tunnels. To improve the strength of the inner lining structure, recently, the steel fiber reinforced concrete has been extensively used in Japan.

(2) Carbon fiber sheet bonding method

The carbon fiber sheet bonding method reinforces the lining concrete of a tunnel by attaching fiber sheets on the lining. Carbon fiber sheet is very light and thin, by using which, the cross-sectional shape of tunnel can be mostly preserved. For the same reason, the constructability of this method is fairly good and the construction period is short. The strong bending strength of fiber sheets can help prevent the propagation of cracks on lining, while no additional measures are necessary to prevent the rust of the sheet.

(3) Steel plate bonding method

In this method, steel plates are attached on the surface of lining and fixed by anchor bolts to strengthen the lining. Between the steel plates and lining surface, a resin layer of several mms is usually placed as a cushion. The cross-sectional space (area) of a tunnel may be compromised by using this method comparing to the carbon fiber sheet bonding method, but is still small comparing to other methods such as inner lining method.

(4) Rock bolt method

Generally, good reinforcing effects can be achieved in engineering practices when rock bolts are authentically integrated with the surrounding rocks of a tunnel. The integration of rock bolts, lining and rocks can effectively improve the strength of lining as well as decrease the long-term deformation of surrounding ground.

(5) Backfilling method

In some old tunnels constructed before NATM is extensively adopted, cavities widely exist between lining...
and ground, which could induce unevenly distributed stresses acting on the lining, and therefore is unfavorable to the stability of tunnel lining especially the long-term deformation behavior of tunnels. Backfilling is the principal measure to deal with this problem in engineering practices. By conducting backfilling method, the cavities behind lining are blocked up, which can help reduce the extension of plastic zones and prevent the underground water flows in the surrounding ground.
4.2 Comparisons of convergence of various reinforcing methods

Fig. 7 shows the delayed convergences $u_\delta$ measured at the springline of tunnel by applying different reinforcing methods at 2 years, 3 years and 4 years after the tunnel is completed, respectively. Fig. 7(a) shows the delayed convergence of the tunnel reinforced by Inner lining method, in which, the case reinforcement conducted 2 years later decreased the convergence by 27.7% comparing to the case without reinforcement. The results of inner lining method indicate that the reinforcement conducted earlier could achieve much better reinforcing effects, which can also be observed in Fig. 7(b) and Fig. 7(c) with smaller effects. For the rock bolt method, however, the cases conducting reinforcement at different times show almost equivalent values with the no reinforcement case (see Fig. 7(d)). As shown in Table 1, the rocks in the ground of Ureshino tunnel is very weak, the competency factor of ground $S_{ap}$, defined by a ration of the uniaxial compressive strength to the initial ground pressure, is as low as 0.24, and the plastic zones widely extend to the surrounding ground, which have limited the effect of rock bolts. Fig. 7(e) shows that by applying backfilling method, the convergence was decreased by 54.6% compared with the case that no reinforcement was applied. The backfilling method exhibited good integration effect by combining lining with the surrounding ground, which could rectify the unevenly distributed stresses and improve the stability of the lining.

The comparison of the decreased ratios of the delayed convergences at different construction times is shown in Fig. 8. For the model without cavity in the back of lining, the inner lining method decreased the delayed convergence with the largest magnitude, followed by the steel plate bonding method and the carbon fiber sheet bonding method. The rock bolt method had nearly no effect on decreasing the delayed convergence. The backfilling method was proved effective to decrease the delayed convergence when cavities occurred in the back of lining. For all the methods, the reinforcement conducted in earlier stage achieved better reinforcing effect.

5 Conclusions

Based on the classic Burger-MC rheological model, a Burger-Deterioration model consisting of a Kelvin section, a Maxwell section and a deteriorated MC section that are connected in series is proposed in this paper, which is implemented in the numerical codes FLAC$^{3D}$ and applied to an engineering instance, Ureshino Tunnel Line I of Nagasaki Expressway, to account for the delayed deformations that occurred after its completion. The deformation characteristics of the Burger-Deterioration model are conspicuously featured by its stair-typed increasing of convergence at the invert, which qualitatively agrees with the in-site monitoring data better than other rheological models. This model is furthermore used to assess the effects of various reinforcing methods as well as to investigate the influence of construction time after the completion of tunnel on the reinforcing effects. The simulation results have confirmed the different performances of various reinforcing methods and showed that the construction conducted earlier can achieve better reinforcing effects.

References


