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Analytical and numerical approach for tunnel face advance in a viscoplastic rock mass

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ABSTRACT

This paper presents both a numerical and an analytical approach for tunnel excavation and support mounting. The three-dimensional aspect of the tunnel face advance and the lining installation is simulated by a two-dimensional study assuming the hypothesis of gradual decompression of the primary stress on the outline of the tunnel. The numerical approach consists of a tunnel axis in an elasto-viscoplastic rock mass and a concrete elastic lining. This study emphasizes some important factors that influence the tunnel calculation, such as tunnel face, history of excavation phases, timing of the lining mounting, lining stiffness, and depth of the tunnel. The analytical approach is based on the determination and integration of the rock–lining interface differential equation. The author presents the analytical solution for (a) constant and non-constant lining pressure, and (b) taking into account the tunnel face influence. The comparison of the numerical results with the analytical solution is performed, and a good agreement is obtained.

1. Introduction

The supported or unsupported tunneling problem is widely investigated both analytically for simple geometries and/or loadings, e.g. [1–7], but especially numerically for more complex conditions e.g. [7–21]. The use of advanced time-dependent behavior and damage models for rock and soil type materials for tunnel calculation represents a great interest. Critescu [1] provided a constitutive elastoviscoplastic model with energetic damage evolution incorporated, used to determine remarkable analytical results for underground openings, while Pellet et al. [2] presented a constitutive elastoviscoplastic model with anisotropic damage described by a second rank tensor. Also, important analytical results are presented by Wang et al. [3] concerning the sequential excavation problem in viscoelastic rock mass with a detailed parametric analysis of different factors of the problem, by Simionescu [4] for circular and non-circular openings including thermal effects, by Sulem et al. [5] for tunneling problems both by analytically (convergence-confinement method) and numerically approaches.

Important FE implementation of time-dependent models along with case studies are presented also, for instance, just to mention only a few. Barla et al.[9] used two different elastoviscoplastic models to perform a numerical analysis of tunnels in squeezing conditions, along with the Lyon–Turin Base Tunnel taken as a case study. Shalabi [10] assessed ground movement and contact pressure on the lining of Stillwater Tunnel (Utah, USA), investigated by an axisymmetric FE analysis using a power law and hyperbolic creep.

An important related issue is the complex excavation problem, e.g. [12–21]. Concerning the geometry and the loading, the successive phases of the tunnel excavation and support mounting is a three-dimensional problem. An analysis in the tunneling direction implies that in most cases a step-by-step method, which models each excavation step by removal of ground elements at the tunnel face and activation of lining elements behind the face, which can be computationally quite inefficient. There are numerical techniques devoted to override this tridimensional aspect of the problem, e.g. [14].

There are also certain cases when the problem may be simplified assuming the hypothesis that close to the tunnel face, on the tunnel outline, the decompression of the primary stress component is occurring gradually ([5,12]). This hypothesis allows a 2D axisymmetrical approach of the problem, which overcomes the considerable effort of a 3D numerical calculation.

This paper deals with an analytical and a finite element solution for the problem of a circular tunnel excavated in an homogeneous isotropic elasto-viscoplastic rock mass. The numerical model consists of the successive phases of the excavation and support mounting, emphasizing the role of two important factors of the analysis, namely the time and the tunnel face influence, taking into account the 3D aspect of the problem. A similar problem is analytically approached by determining the solution

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of the differential equation of the rock–lining interface displacement. A comparison between the two solutions is presented and good results are obtained.

For the excavation problem, an important factor is the time effect and it is involved at least in two different aspects: the rheological behavior of the rock mass and the excavation history. Moreover, the tunnel support mounting determines a problem of interaction. Another important factor in the analysis of ground–support interaction, during the excavation, is the tunnel face influence. For instance, since the behavior of the rock-mass is viscoplastic, rock pressure on the lining increases in time. On the other hand, closer the lining is installed to the tunnel face, more the pressure at the rock–lining interface increases with the advance of the tunnel face.

The state of stress and strain around a lined tunnel depends explicitly on the mechanical and geometrical characteristic of the rock-mass and the support; the excavation conditions, such as excavation rate, generally the excavation phases; the support mounting conditions, especially the support mounting time after the excavation and the distance between the lining and the tunnel face, see e.g. [12].

In this paper, the numerical calculations are performed with a finite element code called CESAR [22] made in LCPC-Paris in which the viscoplastic module is coded and implemented by the author. The viscoplastic module is presented in [7], [8] and its validation with experimental data and available analytical solution or other numerical codes, as well, in [7,8,12]. The constitutive law used in this approach is proposed by Cristescu [1] and it is briefly presented further.

2. Formulation of the problem

Consider the following boundary problem: the rock mass is an infinite body in which a circular opening is made, assuming then that the underground opening is at a certain depth characterized by a hydrostatic primary (initial) stress, $\boldsymbol{\sigma}^P - P\mathbf{1}$, where $P=\gamma h$, h is the depth at which the tunnel is dug, γ is the specific gravity of the rock and **1** is the unity tensor.

Since the tunnel possesses a circular geometry, the rock-mass and the lining mechanical properties are such that they do not depend on the angular coordinate θ and the far stress field in situ is hydrostatic, the problem is an axisymetrical one in Orz plane (Fig. 1). Consequently, the primary stress components σ_v and σ_h are assumed equal. The boundary conditions are presented in Fig. 1.

Cristescu's elasto-viscoplastic constitutive law [1] is used for the rock-mass and elastic behavior for the lining. We present briefly the constitutive hypothesis and the constitutive model.

1) The rock-mass is considered homogeneous and isotropic. Thus, the constitutive functions will depend only on the invariants of the stress and strain tensors. The stress tensor and the strain tensor will be denoted$\boldsymbol{\sigma}$and$\boldsymbol{\varepsilon}$, respectively (their principal components will be denoted $\sigma_1, \sigma_2, \sigma_3, \varepsilon_1, \varepsilon_2, \varepsilon_3$). Among the stress invariants, those with important physical meaning are: $\sigma = (\sigma_1+\sigma_2+\sigma_3)/3$ mean stress, $\overline{\sigma}^2 = \sigma_1^2+\sigma_2^2+\sigma_3^2-\sigma_1\sigma_2-\sigma_2\sigma_3-\sigma_3\sigma_1$ equivalent stress, or $\tau = \sqrt{2}/3\,\overline{\sigma} = \sqrt{2}/\sqrt{3}\,II_{\sigma'}$ octahedral shear stress, with $II_{\sigma'}$ being the second invariant of the stress deviator.
2) The displacements and rotations are assumed small, so that $\dot{\boldsymbol{\varepsilon}} = \dot{\boldsymbol{\varepsilon}}^E + \dot{\boldsymbol{\varepsilon}}^I$, where $\dot{\boldsymbol{\varepsilon}}^E$ and $\dot{\boldsymbol{\varepsilon}}^{\mathbf{I}}$ are the elastic strain rate and the irreversible strain rate, respectively.
3) The initial yield stress of the material is zero or very close to zero.
4) The applicable domain for the constitutive equation is considered for compressive stresses (positive) and bounded by the failure surface which may be incorporated in the constitutive law.
5) The constitutive equation is

$$\dot{\varepsilon}\left(\frac{1}{3K}-\frac{1}{2K}\right)\dot{\sigma}\mathbf{1}+\frac{1}{2G}\dot{\boldsymbol{\sigma}}+\boldsymbol{\kappa}(\boldsymbol{\sigma},\boldsymbol{\delta})\left\langle 1-\frac{W^I(t)}{H(\overline{\boldsymbol{\sigma}},\boldsymbol{\sigma})}\right\rangle\frac{\partial F}{\partial\boldsymbol{\sigma}}. \quad (1)$$

with K, G being the bulk and shear modulus, respectively, 1 being the unit tensor, k represents the viscosity coefficient that may depend on the stress state and the strain state, and probably on a damage parameter d describing the history of the micro cracking the rock was subjected to, and the bracket $<,>$ represents the positive part of respective function: $\langle A\rangle = (A+|A|)/2 = A^+$.

The quantity $W^I(T) = \int_0^T \boldsymbol{\sigma}(t)\cdot\dot{\boldsymbol{\varepsilon}}^I(t)dt = W_v^I(T)+W_d^I(T)$ represents the irreversible work, being used as a hardening parameter or internal state variable, split into volumetric and deviatoric parts. We introduce the damage parameter d, defined by

$$d(t) = W_v^I(t_{max}) - W_v^I(t) \quad (2)$$

that describes the energy released by micro-cracking during the entire dilatancy period. In Eq. (2), t_{max} represents the time for which W_v^I is maximum. The failure threshold is considered to be the total energy released by micro cracking during the entire dilatancy process and it is characterized by the following parameter (constant):

$$d_f = W_v^I(t_{max}) - W_v^I(t_{failure}) \quad (3)$$

$H(\boldsymbol{\sigma})$ represents the loading function, generally a function of stress tensor $\boldsymbol{\sigma}$ with $H(\sigma,\overline{\sigma}) = W^I(t)$ the creep stabilization boundary equation, function H depending on the two stress invariants noted above. $F(\boldsymbol{\sigma})$ represents viscoplastic potential, that establishes the orientation of $\dot{\boldsymbol{\varepsilon}}^{\mathbf{I}}$.

We use the model describing the Borod coal behavior and the constitutive functions and material constants are as follows. For

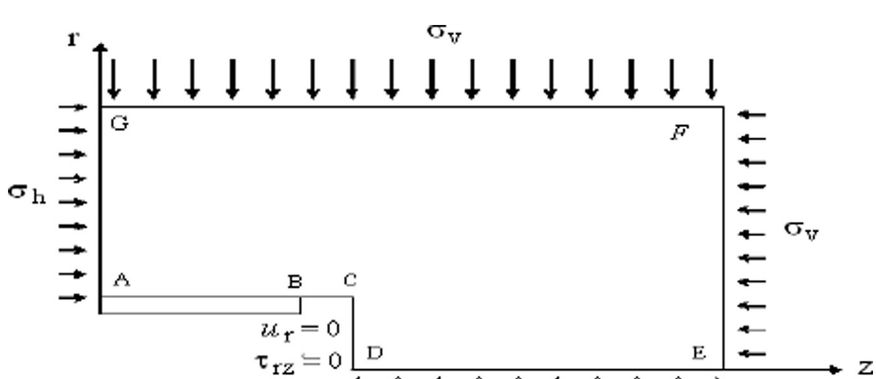

Fig. 1. The domain and boundary conditions for the problem in Orz plane along the tunnel axis.