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ANALYSIS OF THE SUCCESSIVE PHASES OF A TUNNEL EXCAVATION WITH SUPPORT MOUNTING

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This paper analyses the successive phases of a tunnel excavation in a viscoplastic rock mass with an elastic lining mounting. Tunnel face advancing and the support installing yield to a 3D problem, depending on many parameters as time, influence of the tunnel face, history of excavation phases, etc. The aim of this analysis is to reduce the problem to 2D study, as realistically as possible. The study is performed for a circular tunnel and a hydrostatic primary stress state of the rock mass, that determine an axisymmetrical problem. Some theoretical consideration concerning the 3D redistribution of excavation-induceed stresses exploring the near-field stress paths during the progressive face advancement, with a brief presentation of the hypothesis of gradual decompression of the primary stress on the outline of the tunnel, is also performed. The FEM solution is presented by numerous figures and the results are in completely concordance with the practical observations.

Key words: Rock mechanics; Viscoplasticity; Tunnel excavation; Supported tunnelling; FEM.

1. INTRODUCTION

The tunneling problem is one of the most studied one in rock mechanics field in a quite large variety of geometrical or constitutive hypothesis. Concerning the geometry, the problem has been studied in plane stress, plane strain, axisymmetry conditions, in cross section or in the tunnel length axis, with circular or noncircular opening. Regarding the rock-mass behaviour, the constitutive models used are linear elastic or nonlinear [1, 2] – elastoplastic [3–6], viscoelastic [7–9], viscoplastic [7–22], poroelastic [23], chemical-poroelastic[24–26], thermo-elastic [27, 28], thermo-poroelastic, thermo-poro-viscoplastic domain with a numerical [29–34] or analytical approach if possible [35–40].

This paper analyses the problem of a circular tunnel excavated in a homogeneous isotropic and elasto-viscoplastic rock mass by taking into account the 3D aspect of the study. 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.

The time effect is important and it is involved through two different aspects: the rheological behaviour of the rock mass on one hand and the excavation history on the other hand. Moreover, the tunnel support mounting determines a *problem of interaction* between the rock-mass and the lining. For instance, since the behavior of the rock-mass is viscplastic, 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 advancing 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, namely the support mounting time after the excavation and the distance between the lining and the tunnel face.

Regarding the geometry and the loading, the successive phases of the tunnel excavation and support mounting is a *3D problem*. However, there are certain cases when the problem can be simplified by assuming that close to the tunnel face, on the tunnel walls, $r = a$, the decompression of the primary stress component is occurring gradually [5],[20] and[34]. This hypothesis will be discussed in §3 and it is worthwhile as a 3D numerical calculation takes a considerable effort.

In our approach, the numerical calculations are performed with a FEM code called CESAR [41] made in LCPC-Paris. The viscoplastic module is coded and implemented in the finite element code CESAR by the author. The viscoplastic module was presented in [11], [16] and its validation with available analytical solution or other numerical codes, as well, in [16], [21], [29], and [35].

2. FORMULATION OF THE PROBLEM

We consider the following boundary problem: the rock mass is an infinite body in which a circular opening is made, assuming further 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 excavated, γ is the specific gravity of the rock mass and $\mathbf{1}$ 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 can be treated as an *axisymetrical* one in O_{rz} plane (Fig. 2.1).

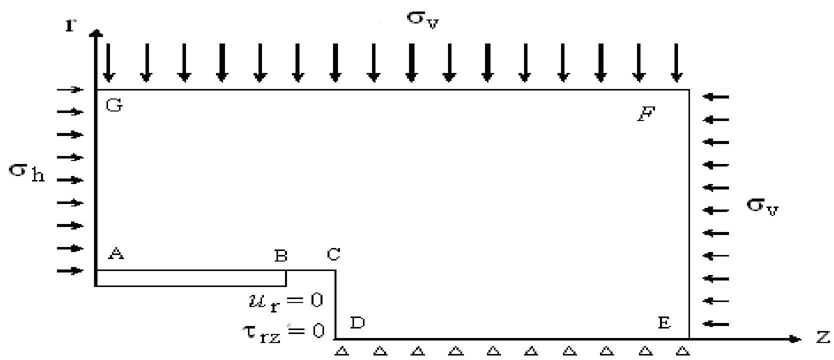

Fig. 2.1 – The domain and the boundary conditions for the study of the problem in O_{rz} plane along the tunnel axis.

Consequently, primary stress components σ_v and σ_h are assumed equal. The boundary conditions are:

On AB i.e. $z \in [z_A, z_B]$ and $r = a$: $\sigma_{rr} = p$ (the lining pressure) and $\sigma_{rz} = 0$.

On BC i.e. $z \in [z_B, z_C]$ and $r = a$: $\sigma_{rr} = 0$ and $\sigma_{rz} = 0$.

On CD i.e. $z = z_C = z_D$ and $r \in [0, a]$: $\sigma_{rr} = 0$ and $\sigma_{rz} = 0$.

On DE i.e. $z \in [z_D, z_E]$ and $r = 0$: $u_r = 0$ and $\tau_{rz} = 0$.

On EF i.e. $z = z_E = z_F$ and $r \in [0, r_F]$: $\sigma_{zz} = \sigma_v$ and $\sigma_{rz} = 0$.

On FG i.e . $z = z_F = z_G$ and $r = r_F$: $\sigma_{rr} = \sigma_v$ and $\sigma_{rz} = 0$.

On GA i.e. $z = z_A$ and $r \in [r_A, r_G]$: $\sigma_{zz} = \sigma_v$ and $\sigma_{rz} = 0$.

Cristescu's elasto-viscoplastic constitutive law is used for the rock-mass and elastic behaviour for the lining. We will briefly summarize the constitutive hypothesis and the main feature of the constitutive model.

The constitutive equation will be formulated following the hypothesis of Cristescu [7]:

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:

– the mean stress:

$$\sigma = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3), \tag{1}$$

– the equivalent stress

$$\bar{\sigma}^2 = \sigma_1^2 + \sigma_2^2 + \sigma_3^2 - \sigma_1\sigma_2 - \sigma_2\sigma_3 - \sigma_3\sigma_1 , \tag{2}$$

or octahedral shear stress $\tau = \frac{\sqrt{2}}{3}\bar{\sigma} = \frac{\sqrt{2}}{\sqrt{3}} II_{\boldsymbol{\sigma}'}$ with $II_{\boldsymbol{\sigma}'}$ being second invariant of the stress deviator.

2) The displacements and the rotations are assumed small, so that:

$$\dot{\boldsymbol{\varepsilon}} = \dot{\boldsymbol{\varepsilon}}^E + \dot{\boldsymbol{\varepsilon}}^I , \tag{3}$$

where $\dot{\boldsymbol{\varepsilon}}^E$ and $\dot{\boldsymbol{\varepsilon}}^I$ being the elastic strain rate and the irreversible strain rate respectively.

3) The component of the elastic strain rate satisfies the Hooke law:

$$\dot{\boldsymbol{\varepsilon}}^E = \left(\frac{1}{3K} - \frac{1}{2G}\right)\dot{\sigma}\mathbf{1} + \frac{1}{2G}\dot{\boldsymbol{\sigma}} , \tag{4}$$

with K, G being the bulk and shear modulus.

4) The component of the irreversible strain rate satisfies:

$$\dot{\boldsymbol{\varepsilon}}^I = k(\sigma, d)\left\langle 1 - \frac{W^I(t)}{H(\bar{\sigma},\sigma)}\right\rangle \frac{\partial F}{\partial \boldsymbol{\sigma}} , \tag{5}$$

where: 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 $\langle A \rangle = \left(A + |A|\right)/2 = A^+$. The quantity:

$$W^I\left(T\right) = \int_0^T \boldsymbol{\sigma}\left(t\right)\cdot\dot{\boldsymbol{\varepsilon}}^I\left(t\right)\mathrm{d}t = W_v^I\left(T\right) + W_d^I\left(T\right), \tag{6}$$

represents the irreversible stress work, being used as a hardening parameter or internal state variable, split into volumetric and deviatoric parts.

We introduce the damage parameter d as well [7], defined by:

$$d\left(t\right) = W_v^I\left(\max\right) + W_v^I\left(t\right), \tag{7}$$

describing the energy released by micro-cracking during the entire dilatancy period.

$H(\boldsymbol{\sigma})$ represents the loading function, generally a function of stress tensor $\boldsymbol{\sigma}$ with:

$$H\left(\sigma,\bar{\sigma}\right) = W^I\left(t\right) \tag{8}$$

the creep stabilization boundary equation, the function H depending on the two stress invariants noted above.

$F(\sigma)$ represents a viscoplastic potential that establishes the orientation of $\dot{\boldsymbol{\varepsilon}}^I$.

5) The initial yield stress of the material is zero or very close to zero.

6) 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 equation.

We will consider therefore the following constitutive equation:

$$\dot{\boldsymbol{\varepsilon}} = \left(\frac{1}{3K} - \frac{1}{2G}\right)\dot{\sigma}\mathbf{1} + \frac{1}{2G}\dot{\boldsymbol{\sigma}} + k(\sigma, d)\left\langle 1 - \frac{W^I(t)}{H(\bar{\sigma},\sigma)}\right\rangle \frac{\partial F}{\partial \boldsymbol{\sigma}} . \tag{9}$$

For the model describing the *Borod coal* behaviour, constitutive functions and material constants are:

$$H(\sigma,\bar{\sigma}) := a_0 \frac{(\bar{\sigma}/\sigma_*)^2}{a_2\sigma/\sigma_* + a_1} + b_0\bar{\sigma}/\sigma_* + \begin{cases} c_0 \sin(\omega\sigma/\sigma_* + \varphi) + c_1 & \text{if } 0 \le \sigma \le \sigma_0, \\ c_0 + c_1 & \text{if } \sigma \ge \sigma_0, \end{cases} \quad \text{the yield surface,} \tag{10}$$

$$F(\sigma,\bar{\sigma}) := H(\sigma,\bar{\sigma}) \qquad \text{the plastic potential, (associated flow rule),} \tag{11}$$

where a_0 = 7.65×10^{-4} MPa; a_1 = 0.55; a_2 = 8.159×10^{-3}; b_0 = 0.001MPa; c_0 = 4.957×10^{-4} MPa; c_1 = 4.8955×10^{-4} MPa; ω = 171.927^0; σ_0 = 1.0996MPa; k = 6×10^{-6}; d_f = 4.48×10^3 Jm^{-3}; σ_* = 1 MPa, and the elastic constants are: E = 798.385 MPa, υ = 0.327.

3. THE INFLUENCE OF THE TUNNEL FACE ON THE CREEP

Many works in tunneling assume a plane strain state in which a cross section of the tunnel is analysed. In that kind of approach the tunnel face influence is neglected, so the analysis of cross section is valid for a great distance from the face, over 4*a*, *a* being the tunnel radius.

On contrary, for an analysis along the tunnel axis, for a distance from the face less than 4 times the tunnel radius, that was observed in practice, it is necessary taking into account another important factor of this analysis, namely the distance *d* from the tunnel face.

It has been shown that the evolution of the rock-support interface displacement (or the outline of the tunnel in the case of unlined tunnel) exhibits, in the case of a stabilization in time of the displacement, in accordance with Fig. 3.1 the specific feature:

$$U(d,t) = \Lambda(d)u(t), \tag{12}$$

where $U(d,t)$ is the displacements that takes into account the tunnel face influence and $u(t)$ represents the radial displacement of the rock-support interface calculated without the influence of the tunnel face [20].

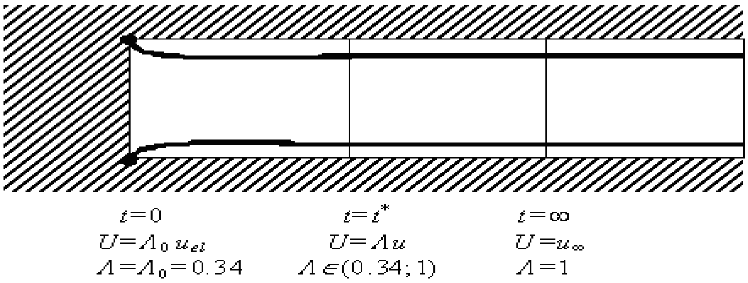

Fig. 3.1 – The evaluation of rock-support interface displacement.

The scalar function Λ depends on the distance *d* in the sense that it is increasing progressively with the distance *d* to the tunnel face of the section to be studied, simulating thus the fact that more the tunnel face advances, the walls are gradually loaded.

So, $\Lambda = \Lambda(d)$, $\Lambda \in [0, 1]$, $\Lambda(0)$ corresponding to the zone behind the tunnel face where $U = 0$, while $\Lambda(1)$ correspond to the zone far from the face of the tunnel where its influence is negligible.

The hypothesis (12) for the displacements means that the displacements can be calculated analitically with the plane strain hypothesis when on the tunnel walls a pressure of $(1-\Lambda)\sigma_v$ is applied. So, we can consider that the proximity of the tunnel face limits the radial displacements (in a given section) and that this action of the tunnel face is equivalent with a fictive pressure of the lining equal with $(1-\Lambda)\sigma_v$ acting on the tunnel surface. The introduction of the scalar function Λ as a parameter, function of the distance *d* from the tunnel face, is thus equivalent with a gradual decompressing of the primary stress on the outline of the tunnel from the initial value σ_v to 0 that means that the unlined tunnel is stress free on the outline.

Thus, considering such an hypothesis of dependency on distance from the tunnel face, represents a simulation of the three-dimensional problem of the excavation [10, 12, 22].

The general shape for the function Λ suggests for instance, the relation:

$$\Lambda(d) = 1 - 0.66/(d/a+1)^2, \tag{13}$$

where *a* is the tunnel radius and it can be observed in Fig. 3.2.

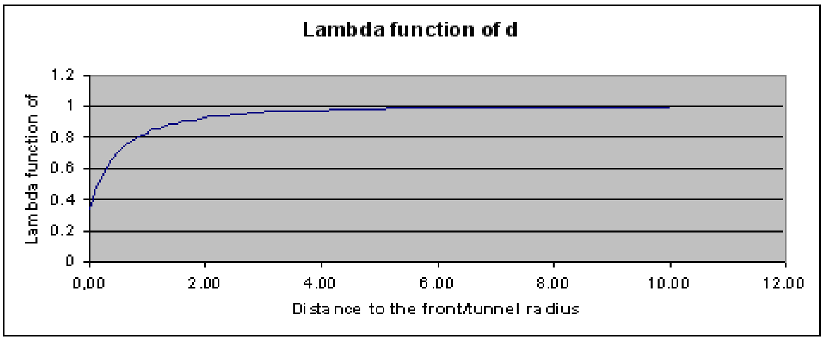

Fig. 3.2 – The shape of function $\Lambda(d)$.

4. FINITE ELEMENT SOLUTION

The rock mass behaviour is considered elasto-viscoplastic, while the concrete lining is elastic. A possible behaviour of sliding at the rock-support interface, which requests some additional contact elements of the mesh, is neglected. The rock mass is homogeneous, for the simplicity of data input, though introducing another layer of rock/soil, with different mechanical characteristics represents no difficulty.

The numerical integration of constitutive equation is performed alternatively by two methods: semi-implicit Euler method (θ scheme) and Runge-Kutta method of fourth order is used in the integration of equation (5) for the viscoplastic strain increment and for evolution differential equation of the hardening parameter W^I that is the irreversible work (6).

It is obvious that it is preferable to lead the calculation to a two-dimensional one, plane strain or axisymetrical, if it is possible – as it is not always the case that is less costly in data input and the running time than a three-dimensional analysis.

Numerical model concerns three successive phases tunnel excavation and lining mounting, as follows:

− *Phase 1*: tunnel excavation and the calculation of strain, stress, damage parameter, without a lining mounting on the new excavated zone. The structure elements corresponding to the support are inactive, that means null mechanical characteristics.

− *Phase 2*: lining installing at a certain given time T_0, on the unexcavated zone in phase 1.

− *Phase 3*: tunnel face advancing on a distance of unit radius, namely 1.2 m.

It is used an 8 nodded-quadrilateral mesh, at least 2 layers of quadrilateral elements in the concrete lining. As usual, in the tunnel surface region the mesh must be quite refine, while elsewhere a minimum possible number of elements is considered (Fig. 4.1).

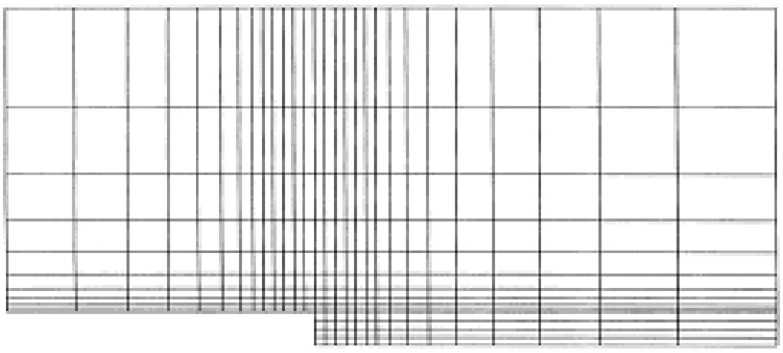

Fig. 4.1 − The domain discretization of the boundary value problem.

It is used a numbering of elements group as they are activated/dezactivated in the excavation and lining installing process, as follows:

− *1st group* is the rock-mass considered infinite,

− *2nd group* corresponds to the already mounted lining,

– *3rd group* is in the first phase the rock-mass that is going to be excavated and in the second phase is replaced by the concrete,

– *4th group* is in the first phase the rock-mass that is going to be excavated,

– *5th group* is in the first two phases the rock-mass that is going to be excavated in the third phase and eventually replaced by concrete in a possible fourth phase,

– *6th group* is in the first two phases the rock-mass that is going to be excavated in the third phase.

Although the example is relatively simple, it reproduces the main situations that intervene in this process: the excavation, advancing of the tunnel face and the mounting of an elastic concrete support.

Let us detail the succession of phases of the example:

Phase 1 – Excavation of element group 3 and 4 .
Element group "concrete" is kept inactive (Young modulus $E = 0$).
Use of a stress field of geostatic type, for instance.
Output storage for the following phase (phase 2).

Phase 2 – The element group "concrete" is activated (Young modulus $E \neq 0$).
Use again of a stress field of geostatic type.
Output storage for the following phase (phase 3).
Displacement and stress initialization starting from the previous phase output storage.

Phase 3 – Realization of a new excavation by inactivation of element group 5 and 6, considering null mechanical characteristics.
Displacement and stress initialization with the 2nd phase state.

We consider the tunnel radius $a = 1.2$ m and the lining thickness 0.2 m. The depth at which the tunnel is excavated is 273.5 m. One phase duration is 12 hours and respectively 1 day.

For the rock mass the *Borod coal* is used, whose material constants were presented in section two. For the concrete the following material constants were used: Young modulus E = 20000 MPa, Poisson coefficient $\nu = 0.3$ and the volumetric weight $\gamma = 0.02$ MN/m^2.

In the following, we present some of the following results of the calculation. In fig. 4.2 izovalues zones for the damage parameter W_v^1 corresponding to the three phases are presented, respectively. It is observed that in the tunnel face zone the damage is maximum (the white area).

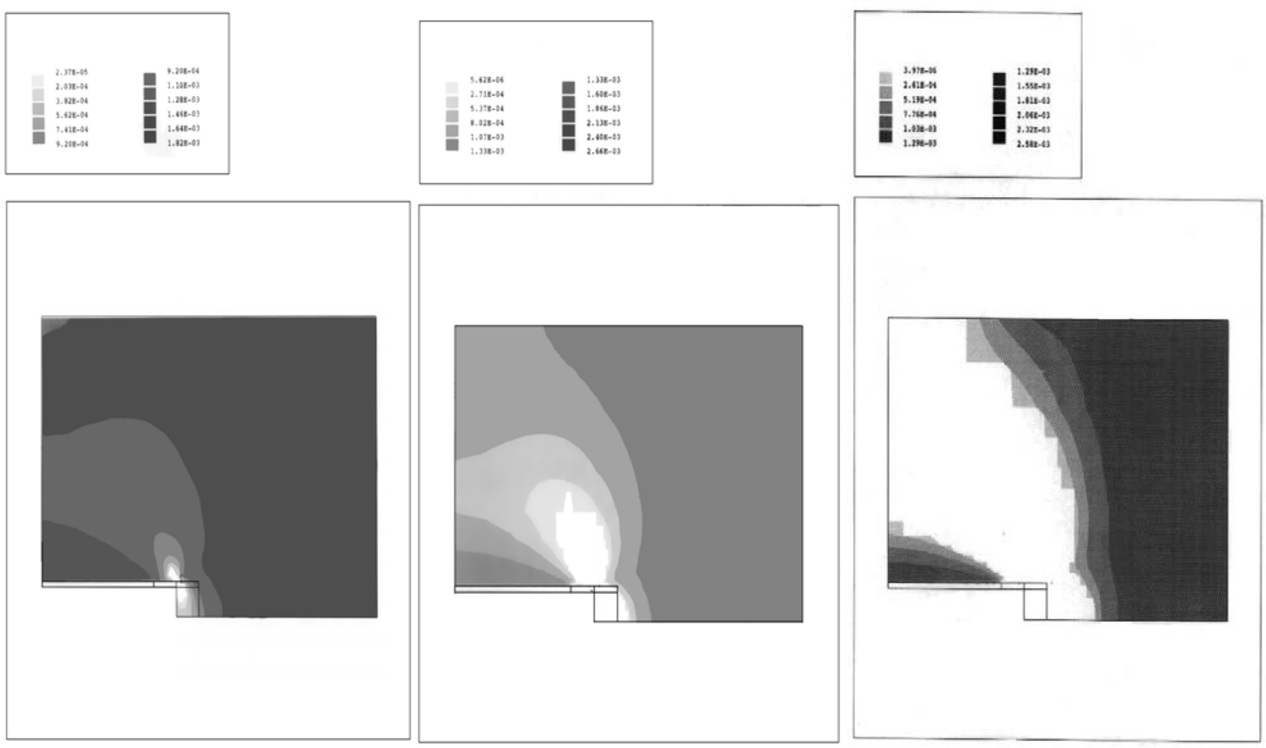

Fig. 4.2 – Izovalues zones for damage parameter W_v^1 for the first, second, respectively, the third phase.

Fig. 4.3 presents izovalues zones for equivalent stress corresponding to the three phases, observing small tractions in the second, respectively the third phase. That signifies the possibility of fracture by exceeding the traction resistance, as it is known that it is very low for the rock type material.

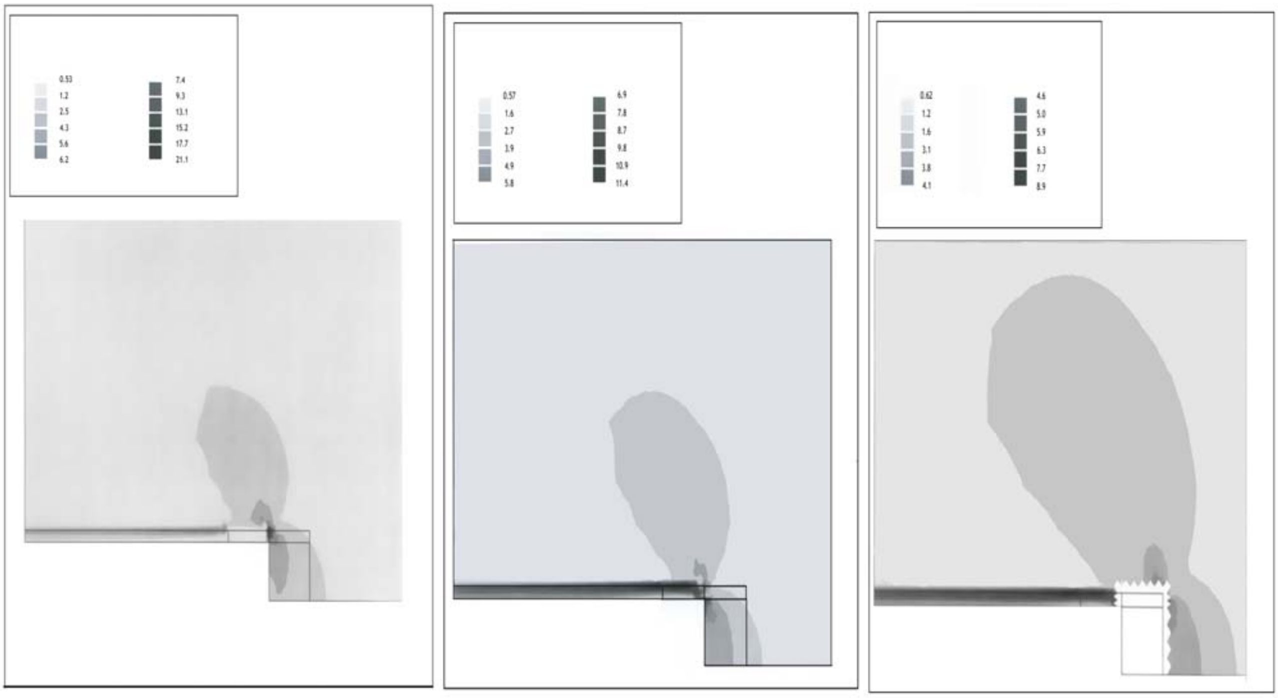

Fig. 4.3 – Izovalues zones for equivalent stresses for first, second and respectively third phase.

5. CONCLUSIONS

In this paper, the simulation of a tunnel excavation with successive tunnel face advancing and the lining mounting was performed. Due to the symmetry of the geometry and loadings, the problem was treated as an axisymetrical one, but taking into account the three-dimensional aspect of the problem, namely the tunnel face advancing and its proximity influence. So, the approach of a tunnel calculation in two-dimensional analysis along the tunnel axes, simulating thus the three-dimensional aspect of the problem, is more realistic than the classical cross section analysis and obviously less costly than an actual three-dimensional analysis.

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