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THE EFFECT OF ADVANCE-DRAINAGE ON THE SHORT-TERM BEHAVIOUR OF SQUEEZING ROCKS IN TUNNELING

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ABSTRACT: *Experience shows that high pore water pressures favor the development of squeezing pressures or deformations, while drainage ahead of the working face considerably reduces the intensity of squeezing. The present paper aims to improve understanding of the effects of drainage and to provide a simple computational model for assessing these effects quantitatively. Attention is given to the short-term behavior of the ground, i.e. to its behavior in the vicinity of the working face during excavation. The favorable effects of advance-drainage are studied within the framework of the ground response curve. In order to understand the effects of advance-drainage on the short-term behavior of the opening it is necessary to understand the factors governing this behavior. For this reason we revisit the problem of the instantaneous response of the ground for the reference case without advance drainage. The closed-form equations derived for this case show that the short-term deformations depend strongly on the initial effective stress. The higher the latter, the higher also will be the shear resistance of the ground and the smaller will be the deformations. Advance-drainage improves the ground response to tunnel excavation as it causes consolidation of the ground and increases the effective stresses ahead of the face.*

1 INTRODUCTION

The term "squeezing" refers to the phenomenon of large deformations in tunneling. Experience and theoretical considerations show that high pore water pressures favor the development of squeezing. On the other hand, repeated observations indicate that drainage considerably reduces the intensity of squeezing (Steiner, 1996; Kovári, 1998; Barla, 2001; Vogelhuber et al., 2004). The present paper aims to improve understanding of the effects of drainage and to provide a simple computational model for assessing this effect quantitatively.

The Simplon twin railway tunnel - an engineering landmark – provides an example of the favorable effects of drainage. The first tube of this 20 km long tunnel (as well as a small pilot adit of the second tube) was excavated in the period 1898 - 1906. Over a 42 m long stretch, heavily squeezing rock was encountered that looked like a "dough, mainly consisting of soft micaceous limestones" (Pressel, 1906). Tunnel construction over this short section took seven months. In the second tube, however, which crossed the same formation, relevant deformations did not occurr. The second tube was excavated about ten years later than the first (due to the World War I). It is believed that the improvement of the behavior of the ground was caused by its lengthy drainage and consolidation (Steiner, 1996). This hypothesis is supported by the considerable reduction of the water inflows during this period. Steiner (1996) draws similar conclusions from the excavation of the Vereina railway tunnel. Drainage was a particularly effective measure for coping with squeezing ground in this tunnel as well. Another interesting example is the Gotthard motorway tunnel (Lombardi, 1976). In addition to the main road tunnel, this underground structure includes a safety adit of smaller cross-section. The axial distance between the safety adit and the main tunnel amounts to 30 m. In a short stretch consisting of soft clayey shales, the safety adit, which was excavated first, experienced very large deformations which necessitated extensive re-profiling works. It is interesting to note that pore pressures of up to 3 MPa have been monitored at a distance of only 5 - 10 m from the adit wall. The excavation of the main tunnel followed that of the safety adit with a delay of about one year. The squeezing phenomena in the main tunnel have been far less serious in spite of the same geology and of the larger cross-section. This observation can be explained only by the positive effect of the drainage which took place in the period elapsing since the excavation of the safety adit.

Squeezing normally develops slowly, although cases have also been known where rapid deformations occur very close to the working face (Ramoni & Anagnostou, 2007). The time-dependency of squeezing can be traced back to the rheological behavior of the rock (creep) and, in the case of saturated, low-permeability rocks, also to consolidation processes that are triggered by the tunnel excavation and develop slowly over the course of time: The long-term deformations of the ground include, in general, changes to its pore volume and water content. The latter needs more or less time, depending on the seepage flow velocity and thus on the permeability of the ground. In a low-permeability ground, the water content remains constant in the short term. Tunnel excavation generates excess pore pressures, however. As these are higher in the vicinity of the tunnel than they are further away, seepage flow starts to develop. So the excess pore pressures dissipate over the course of time, thereby changing the effective stresses and leading to additional time-dependent deformations, i.e. consolidation.

The present paper focuses on the short-term behavior of the ground, i.e. to its behavior in the vicinity of the working face during continuous excavation. The short-term behavior is of paramount importance from a constructional point of view: Depending on the intensity of squeezing, it may be necessary to apply large amounts of support close to the working face in order to control the ground. This slows down tunnel advance considerably, as support installation close to the face greatly interferes with the excavation works. The short-term behavior is, furthermore, decisive concerning the risk of shield jamming in mechanized tunneling (Ramoni & Anagnostou, 2007).

The favorable effect of advance-drainage will be illustrated by considering the classic, rotationally symmetric, plane-strain problem of a deep, cylindrical tunnel (Figure 1). The assumptions underlying rotational symmetry are: uniform pressure σ_a, homogeneous and isotropic ground, homogeneous and isotropic initial stress and pore pressure fields (with σ_0, σ_0' and p_0 denoting the initial values of total stress, effective stress and pore pressure, respectively). Under these conditions, the ground response to the tunneling operation can be represented by a single curve, the so-called characteristic line (also referred to as "ground response curve"), which expresses the relationship between the support pressure σ_a and the radial displacement u_a of the excavation boundary (i.e. at $r = a$, where r and a denote the distance from the tunnel centre and the tunnel radius, respectively). This relationship is in general non-linear due to the plastification of the ground around the tunnel up to a radius ρ (Figure 1).

In order to understand the effect of advance-drainage on the short-term behavior of the opening it is necessary to understand the factors governing this behavior. For this reason we will first revisit the problem of the instantaneous response of the ground for the reference case without advance drainage (Section 2). More specifically, we will see that the effective stress prevailing before excavation is decisive for the short-term behavior. Afterwards we will show how much the drainage increases the effective stress field ahead of the working face and how much this improves the ground response to tunnel excavation (Section 3).

2 SHORT TERM GROUND RESPONSE WITHOUT PRE-CONSOLIDATION

Labiouse & Giraud (1998) and Giraud et al. (2002) have presented closed form equations for the instantaneous response of a saturated, porous medium with compressible constituents (pore water and solid grains). In the present paper closed-form solutions will be derived for the ground response curve under the simplifying assumption of incompressible ground constituents. In this case, the condition of constant water content, which characterizes the short-term response, becomes equivalent to the condition of zero volumetric strain. As a consequence of this simplification (which is reasonable for soft ground or weak rock) the mathematical derivation of the ground response curve is much more straightforward than in the general case of compressible constituents. We will see, furthermore, that the resulting equation is considerably simpler than the existing solutions, thus illustrating clearly the factors governing short-term response.

In order to model the short-term behavior of the ground, two basic possibilities exist (Lambe and Whitman, 1969; Labiouse & Giraud, 1998): either to consider the ground as a single-phase, frictionless medium (so-called "$\phi = 0$" - concept) and carry-out an analysis in terms of total stress; or to take it as a saturated porous medium obeying the principle of effective stresses and to carry out an effective stress analysis. A total stress analysis is simpler but unsuitable for the purposes of this paper as it does not take into account the pore pressures and their alteration by drainage. The analysis will be carried out therefore in terms of effective stresses.

The mechanical behavior of the ground is assumed to be linearly-elastic and perfectly-plastic according to the Mohr-Coulomb yield criterion and is characterized by the following material constants: the Young's modulus E, the Poisson's number ν, the cohesion c' and the friction angle ϕ'. For the sake of simplifying the mathematical derivations (and without loss of generality in terms of the most relevant conclusions) we will assume non-dilatant plastic behavior (i.e., the dilatancy angle $\psi = 0$).

For understanding the instantaneous pore pressure changes caused by the excavation, it helps to discuss first what would happen in the case of *dry ground*. The well-known closed-form solutions existing for the stress field show that both the radial and the tangential stresses decrease within the plastic zone and this leads (according to Hooke's law, which interconnects the elastic strain increments with the stress-changes) to elastic volumetric strains. These in general, together with a plastic dilatancy accompanying yielding, lead to an increase of the pore volume within the plastic zone. In the outer, elastic zone, however, the excavation does not produce any change of the mean stress or any volumetric strain. According to Kirsch's solution, the ground there experiences pure shearing: it contracts tangentially by the same amount as it extends in the radial direction. Consequently, the pore volume remains constant.

In the case of *water bearing ground*, the changes of the pore volume mentioned above go together with changes of water content. The latter need more or less time depending on the seepage flow rate and thus on the permeability of the ground. The augmentation of the pore volume within the plastic zone cannot take place immediately in a low-permeability ground.

Instead negative excess pore pressures develop, i.e. the pore pressure drops. (The instantaneous pressure drop has been observed also *in situ*, e.g. in the Mont Terri laboratory; Bossart, 2008.) This happens of course only within the plastic zone. In the surrounding elastic zone the pore water does not impose any constraint on the pure shearing deformations. The pore pressure remains therefore equal to its initial value p_0 everywhere within the elastic zone and, since the pore pressure gradient is equal to zero, all equations underlying Kirsch's solution apply to the case of saturated ground as well, the only difference being that the total stresses should be replaced by the effective stresses. Thus, the radial displacement u_ρ at $r = \rho$ is given by the equation

$$u_\rho = \frac{1+\nu}{E}\,\rho\left(\sigma_0' - \sigma_\rho'\right), \tag{1}$$

where σ_ρ' denotes the effective radial stress at $r = \rho$; Furthermore, the radial stress decreases by the same amount as the tangential stress decreases and, consequently, their average value remains equal to the initial stress:

$$0.5\left(\sigma_r' + \sigma_t'\right) = \sigma_0'. \tag{2}$$

This equation together with the yield condition

$$\sigma_t' = \frac{1+\sin\phi}{1-\sin\phi}\sigma_r' + \frac{2c\cos\phi}{1-\sin\phi}, \tag{3}$$

which has to be satisfied specifically for $r = \rho$, yields the effective radial stress σ_ρ' as well as the tangential stress at the elasto-plastic interface:

$$\sigma_\rho' = \sigma_0' - s_u, \quad \sigma_{t(r=\rho)}' = \sigma_0' + s_u, \tag{4}$$

where the constant

$$s_u = \sigma_0' \sin\phi + c\cos\phi. \tag{5}$$

The system remains within the elastic domain as long as the support pressure σ_a is higher than $\sigma_\rho = \sigma_0 - s_u$. At lower support pressures, plastification takes place. Within the plastic zone, the volumetric strain

$$\varepsilon_{vol} = \varepsilon_{vol}^{EL} + \varepsilon_{vol}^{PL} = 0, \tag{6}$$

where the two terms of the sum denote the elastic and the plastic part. Since the latter is equal to zero for the assumed non-dilatant plastic behavior, the elastic volumetric strain must be zero as well:

$$\varepsilon_{vol}^{EL} = \varepsilon_r^{EL} + \varepsilon_t^{EL} + \varepsilon_z^{EL} = 0, \tag{7}$$

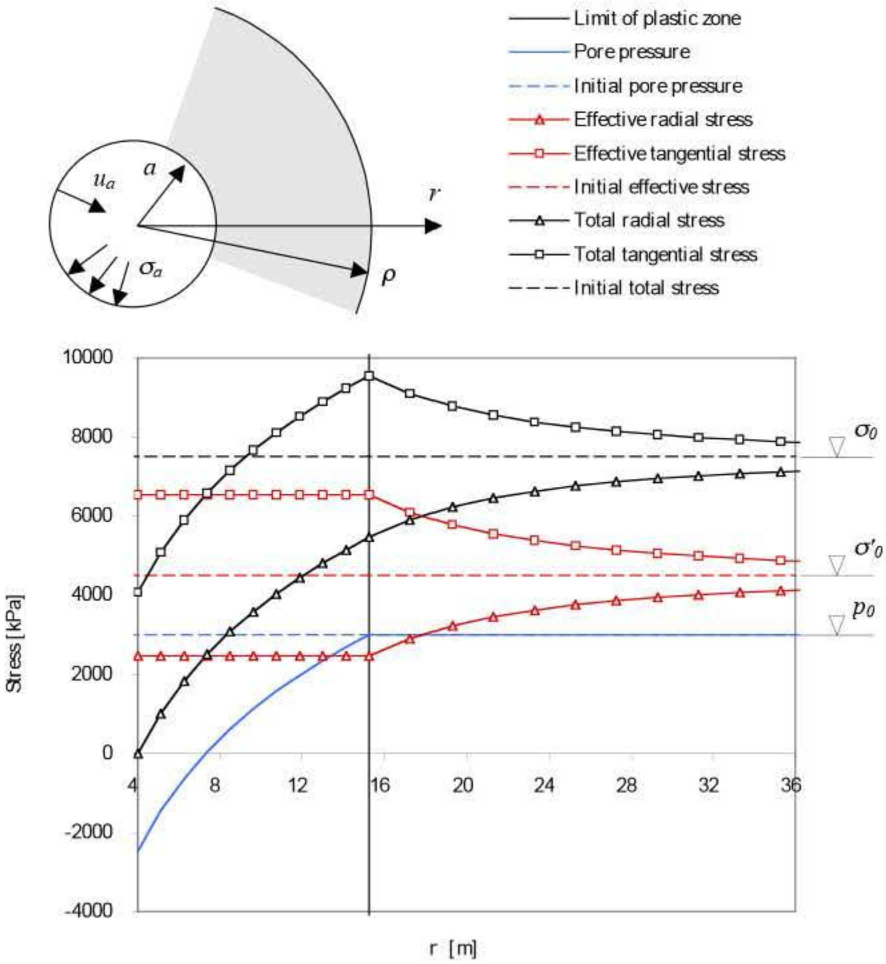

Fig. 1. Problem layout and short-term distribution of the pore pressure, of the effective stresses and of the total stresses without pre-consolidation of the ground (parameters values: see Table 1)

Table 1. Parameters of the numerical example

Parameter	Value
Tunnel radius a	4 m
Initial total stress σ_0	7.5 MPa
Initial pore pressure p_0	3 MPa
Young's modulus E	400 MPa
Poisson's number ν	0.25
Cohesion c'	150 kPa
Friction angle ϕ'	25°

where the terms of the sum denote the elastic strains in the three principal directions. Since $\varepsilon_z = 0$ (plane strain condition) and the out-of-plane plastic flow can be neglected in most cases (Cantieni and Anagnostou, 2008),

$$\varepsilon_z^{EL} = -\varepsilon_z^{PL} = 0 \tag{8}$$

and, consequently (Eq. 7),

$$\varepsilon_r^{EL} + \varepsilon_t^{EL} = 0. \tag{9}$$

Eqs. (8) and (9) in combination with Hooke's law, which expresses the relationship between the elastic strain components and the change of the effective stresses, lead to the following expressions for the latter:

$$\sigma'_z = \sigma'_0, \quad \sigma'_r + \sigma'_t = 2\sigma'_0. \tag{10}$$

According to this equation, the average effective stress remains equal to the initial stress σ'_0 not only within the elastic zone but in the plastic zone as well. In addition to Eq. (10), the effective stresses must satisfy the yield criterion (Eq. 3) at each point within the plastic zone. These two equations determine completely the effective stress field. Since the radius r does not appear in Eqs. (3) and (10), the effective stresses are constant within the plastic zone, i.e. they do not depend on the position r for $a < r < \rho$ (Figure 1) and are, therefore, equal to the stresses at $r = \rho$ which have been derived before. So, Eq. (4) can be replaced by

$$\sigma'_r = \sigma'_0 - s_u, \quad \sigma'_t = \sigma'_0 + s_u \quad (a \le r \le \rho). \tag{11}$$

Since the effective radial stress at the excavation boundary is now given by Eq. (11) and the total stress at $r = a$ is also known (it is equal to the given support pressure σ_a), one can calculate the pore pressure at the excavation boundary:

$$p_a = \sigma_a - \sigma'_a = \sigma_a - \sigma'_0 + s_u. \tag{12}$$

For obtaining the complete distribution of the pore pressure, we insert the known effective stresses (Eq. 11) into the equilibrium condition

$$\frac{d\sigma'_r}{dr} = \frac{\sigma'_t - \sigma'_r}{r} - \frac{dp}{dr}, \tag{13}$$

thus obtaining the following equation for the pore pressure field:

$$r\frac{dp}{dr} = 2s_u. \tag{14}$$

The solution of this equation for the boundary condition at $r = a$ (Eq. 12) reads as follows:

$$p = p_a + 2s_u \ln\frac{r}{a}. \tag{15}$$

According to this equation, the pore pressure increases monotonously with the radius r and must therefore reach its initial value p_0 at a certain distance from the tunnel. This distance marks the interface between plastic and elastic zone (as explained above, the pore pressure remains equal to p_0 in the elastic zone). Consequently applying Eq. (15) for $p = p_0$ yields the radius ρ of the plastic zone:

$$\frac{\rho}{a} = e^{\frac{\sigma_0 - \sigma_a - s_u}{2s_u}}. \tag{16}$$

As the volume of the plastic zone remains constant, the excavation boundary displacement u_a is related to the displacement u_ρ of the elasto-plastic interface by the following geometrical relationship:

$$a\, u_a = \rho\, u_\rho. \tag{17}$$

Combining Eqs. (1), (16) and (17) yields the following, very simple expression for the radial displacement:

$$\frac{u_a}{a} = \frac{1+\nu}{E} s_u\, e^{\frac{\sigma_0 - \sigma_a}{s_u} - 1}. \tag{18}$$

Note that if the support pressure $\sigma_a \geq \sigma_0$ - s_u, the system remains elastic and the ground response curve is given by Kirsch's solution, i.e. by Eq. (1) with a and σ_a instead of ρ and σ_ρ, respectively.

As shown in the Appendix, Eqs. (16) and (18) can be obtained also by a common total stress analysis for a material having the same shear modulus $(1+\nu)/E$ as above, a friction angle $\phi_u = 0$ and a shear strength s_u according to Eq. (5). The effect of drainage becomes evident when observing Eq. (5), as this equation shows that the equivalent shear strength s_u increases linearly with the effective stress prevailing before excavation. Advance-drainage causes consolidation of the ground ahead of the working face, i.e. a pore pressure relief, a higher effective stress and, on account of Eq. (5), a higher shear strength. The drainage-induced modification of the effective stress field will be studied in the next Section.

3 SHORT TERM GROUND RESPONSE AFTER PRE-CONSOLIDATION

3.1 Effect of advance-drainage on stress field

We consider a cross-section ahead of the working face and assume that the drainage via the advance boreholes causes a complete pore pressure relief inside the core, i.e. that the pore pressure $p = 0$ for $r \leq a$, where a denotes the radius of the future opening. As a consequence of the drainage, seepage flow starts to occur, the pore pressures decrease around the core as well and this leads to consolidation of the ground, i.e. to an increase of the effective stresses and to radial displacements towards the core.

This problem has been analyzed by Anagnostou & Kovári (2003) in the context of tunneling through geological fault zones and under the following simplifying assumptions: the ground response to drainage is elastic; the seepage flow obeys Darcy's law; the region affected by the drainage extends up to a distance R from the tunnel (i.e., $p = p_0$ for $r \geq R$), where the influence radius R can be taken equal to the initial head p_0/γ_w (γ_w is the unit weight of the water). According to Anagnostou & Kovári (2003), due to the drainage, the future excavation boundary experiences an inward radial displacement of

$$u_{a,DR} = \frac{(1+\nu)(1-2\nu)}{2(1-\nu)} a \frac{p_0}{E} \tag{19}$$

and an increase of the effective radial stress by

$$\sigma'_{a,DR} - \sigma'_0 = \frac{p_0}{2(1-\nu)}, \tag{20}$$

while the total radial stress decreases by

$$\sigma_{a,DR} - \sigma_0 = -p_0 \frac{1-2\nu}{2(1-\nu)}. \tag{21}$$

In the borderline case of $\nu = 0.5$ (which characterizes the theoretical case of an incompressible material), the volume of the core remains constant and, consequently, the drainage-induced radial displacement $u_{a,DR} = 0$ (cf. also Eq. 19). Furthermore, as can be seen from Eqs. (20) and (21), the total stress remains unaffected by the drainage, while the effective stress increases by the same amount as the pore pressure decreases. So, with reference to Eqs. (5) and (18), the effect of drainage is an increase of the parameter s_u (which is the shear strength of the equivalent material in the total stress analysis, as stated above) by $\Delta s_u = p_0 \sin\phi'$.

For typical values of the Poisson's ratio ($\nu = 0.20$ - 0.30), the drainage causes a smaller increase in the equivalent shear strength (by about $\Delta s_u = 65\% \, p_0 \sin\phi'$) but, at the same time, a reduction in the total stress that prevails before excavation at $r = a$ by about $\Delta\sigma_a = 35\% \, p_0$.

The complete distribution of the pore pressure p and of the effective stresses over the radius r are given by the following equations:

$$\frac{p}{p_0} = \frac{\ln(r/a)}{\ln(R/a)}, \tag{22}$$

$$\frac{\sigma'_{r,DR} - \sigma'_0}{p_0} = f_1(r) - f_2(r), \quad \frac{\sigma'_{t,DR} - \sigma'_0}{p_0} = f_1(r) + f_2(r), \quad \frac{\sigma'_{z,DR} - \sigma'_0}{p_0} = 2\nu f_1(r), \tag{23}$$

where

$$f_1(r)=\frac{2\ln(R/r)}{4(1-\nu)\ln(R/a)},\quad f_2(r)=\frac{(1-2\nu)\left[1-\left(\frac{a}{r}\right)^2\right]}{4(1-\nu)\ln(R/a)}. \tag{24}$$

One can readily verify that the radial and the tangential effective stress change at $r = a$ by the same amount, but their difference increases with the distance r from the tunnel and this implies the possibility of yielding. As discussed by Anagnostou & Kovári (2003), the assumption of elastic behavior is valid in most cases but should be checked, if the initial pore pressure p_0 is high relatively to the initial effect stress σ_0' (this may happen in the case of subsea tunnels).

3.2 Ground response curve after pre-consolidation

As discussed above, advance-drainage modifies in general both the effective and the total stress fields. The state prevailing after drainage represents the initial state for the calculation of the ground response curve. As can be seen from Figure 2, this "modified" initial state is neither homogeneous nor isotropic and therefore the closed form equations of Section 2, which have been derived assuming homogeneous and isotropic initial stress fields, may be applied (with modified initial stresses $\sigma_0 = \sigma_{a,DR}$ and $\sigma'_0 = \sigma'_{a,DR}$ according to Eqs. 20 and 21) only in the sense of an approximation.

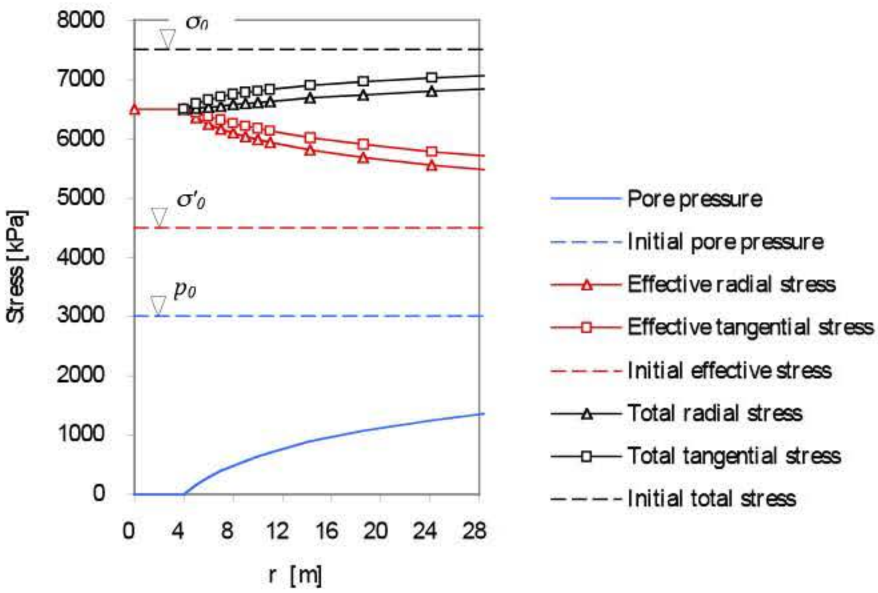

Fig. 2. Distribution of the pore pressure, of the effective stresses and of the total stresses after pre-consolidation of the ground (parameters values: see Table 1)

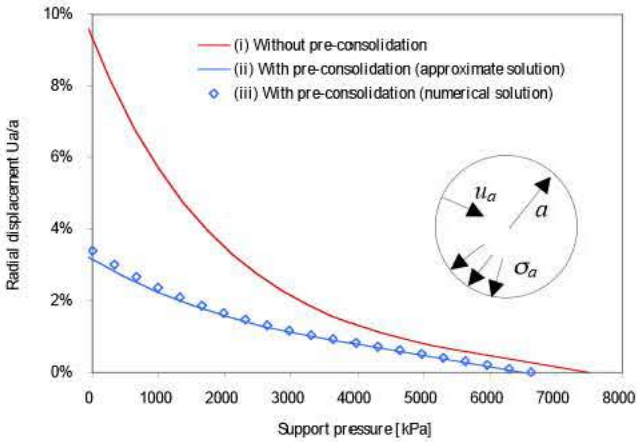

Fig. 3. Ground response curve without / with pre-consolidation (parameters values: see Table 1)

An exact solution can be obtained numerically by the Finite Element Method. Such a calculation proceeds in two steps. The first step simulates the advance drainage and gives as a result the state prevailing before tunnel excavation (basically the results of Section 3.1). In the second computational step, we calculate the ground response curve by reducing the radial stress at $r = a$ monotonously from the value prevailing after drainage (Eq. 21) to zero.

Figure 3 shows for a numerical example (i) the exact ground response curve without pre-consolidation of the ground (based on Eq. 18); (ii) the approximate ground response curve with pre-consolidation of the ground (based on Eq. 18 with modified initial stresses); (iii) the exact ground response curve with pre-consolidation of the ground (obtained numerically by the Finite Element Method). It is clear that the error introduced by the approximate closed-form solution is relatively small, while the effect of drainage is considerable from the practical engineering point of view (at common support pressures of 0.5 - 1 MPa, the convergence amounts to about 2 - 3% instead of 7 - 9%).

4 CLOSING REMARKS

The equations given in Section 3.1 apply to the steady state, i.e. they pre-suppose completion of the transient consolidation process associated with the advance-drainage. As squeezing rocks are often fine-grained and have a very low permeability, the time needed for the pre-consolidation may be very long. Auxiliary structures (such as side adits or pilot tunnels) offer the possibility of drainage sufficiently in advance of the actual excavation works.

We focused here on the effect of pre-consolidation on the short-term ground response. The advance-drainage has, however, a considerable favorable effect on the long-term behavior as well. This issue is being investigated by Anagnostou (2009).

APPENDIX: TOTAL STRESS ANALYSIS OF SHORT-TERM GROUND RESPONSE

As in the effective stress analysis of Section 2, the mechanical behavior is assumed to be linearly-elastic and perfectly-plastic (this time however with a Tresca yield condition, since the material is frictionless in terms of total stress) and is characterized by three material constants: the undrained shear strength s_u (also referred to as "undrained cohesion"); the Young's modulus E_u; and the Poisson's number ν_u. (We use here the subscript "u" in order to differentiate the material constants from those appearing in the constitutive equations in terms of effective stress.)

The derivation of the short-term ground response curve in terms of total stress is routine because the Tresca criterion represents a special case of the Mohr-Coulomb criterion for which well-known closed form solutions do exist (cf. e.g. Anagnostou & Kovári, 1993). However, the derivation will be outlined here in order to illustrate the relationship between total and effective stress analysis (see Section 2).

The radial displacement u_a of the excavation boundary can be obtained again from the geometrical relationship Eq. (17), while Kirsch's solution applies for the displacement u_ρ of the elasto-plastic interface ($r = \rho$):

$$u_\rho = \frac{1+\nu_u}{E_u}\,\rho\left(\sigma_0 - \sigma_\rho\right), \tag{25}$$

where σ_ρ denotes the radial stress at the elasto-plastic interface $r = \rho$. The radial stress σ_ρ can be obtained by considering the outer elastic zone. More specifically, the stress components at $r = \rho$ must satisfy both the yield condition

$$\sigma_t = \sigma_r + 2s_u \tag{26}$$

and Kirsch's solution, according to which the average stress is equal to the initial stress:

$$0.5\left(\sigma_r + \sigma_t\right) = \sigma_0. \tag{27}$$

The solution of the system of the linear Eqs. (26) and (27) yields the stresses at $r = \rho$:

$$\sigma_\rho = \sigma_0 - s_u, \quad \sigma_{t(r=\rho)} = \sigma_0 + s_u. \tag{28}$$

In addition to σ_ρ, the radius ρ of the plastic zone is needed by the equation for the radial displacement u_a (Eq. 17). In order to calculate ρ, we determine the stress field within the plastic zone based upon the yield condition and the equilibrium equation

$$\frac{d\sigma_r}{dr} = \frac{\sigma_t - \sigma_r}{r}. \tag{29}$$

Eqs. (26) and (29) represent a system, whose solution yields the radial stress within the plastic zone $r \leq \rho$:

$$\sigma_r = \sigma_a + 2s_u \ln\left(r / a\right). \tag{30}$$

For the determination of the radius ρ of the plastic zone, we apply this equation to $r = \rho$ and solve the resulting equation with respect to ρ. Taking into account σ_ρ according to Eq. (28), we obtain again Eq. (16) for the radius ρ of the plastic zone, while combining Eqs. (17), (25), (28) and (16) yields again Eq. (18) for the ground response curve.

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