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The effect of the stress path on the interaction between yielding supports and squeezing ground

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ABSTRACT
In this paper we investigate the interaction between yielding supports and squeezing ground by means of spatial numerical analyses that take into account the stress history of the ground. The idea behind yielding supports is that squeezing pressure will decrease by allowing the ground to deform. When estimating the amount of deformation required, one normally considers the characteristic line of the ground, i.e. the relationship between the ground pressure and the radial displacement of the tunnel wall under plane strain conditions. The computation of the characteristic line assumes a monotonic decrease of radial stress at the excavation boundary, while the actual tunnel excavation involves a temporary complete radial unloading and due to the support a subsequent reloading of the tunnel wall. This difference, in combination with the stress-path dependency of the ground behavior, is responsible for the fact that the results obtained by spatial analysis are not only quantitatively, but also qualitatively different from those obtained by plane strain analysis. More specifically, the relationship between ground pressure and deformation at the final state prevailing far behind the face is not unique, but depends on the support characteristics, because these affect the stress history of the ground surrounding the tunnel. The yield pressure of the support, i.e. its resistance during the deformation phase, therefore proves to be an extremely important parameter. The higher the yield pressure of the support, the lower will be the final ground pressure. A targeted reduction in ground pressure can be achieved not only by installing a support that is able to accommodate a larger deformation (which is a well-known principle), but also by selecting a support that yields at a higher pressure.

Keywords: Squeezing, Yielding support, Stress path

INTRODUCTION
The term "squeezing" refers to the phenomenon of large deformations that develop when tunneling through weak rocks. If an attempt is made to stop the deformations with the lining (so-called "resistance principle", Kovári 1998), a so-called "genuine rock pressure" builds up, which may reach values beyond the structurally manageable range. The only feasible solution in heavily squeezing ground is a tunnel support that is able to deform without becoming damaged, in combination with a certain amount of over-excavation in order to accommodate the deformations. Supports that are based on this so-called "yielding principle" can be structurally implemented in two main ways (Anagnostou and Cantieni, 2007): either by arranging a compressible layer between the excavation boundary and the extrados of a stiff lining (Fig. 1a) or through a suitable structural detailing of the lining that will allow a reduction in its circumference (Fig. 1b). In the first case the ground experiences convergences while the clearance profile remains practically constant. This solution has been proposed particularly for shield tunneling with practically rigid segmental linings (Schneider et al.
2005, Billig et al. 2007). The second solution is the one usually applied today. It involves steel sets having sliding connections in combination with shotcrete (Fig. 1b, sections c-c and d-d, respectively).

Steel sets applied in squeezing ground usually have a top hat cross-section. The hoop force in the steel sets is controlled by the number and by the pre-tensioning of the friction loops connecting the steel segments. Shotcrete may be applied either after the occurrence of a pre-defined amount of convergence (as a recent example the lot Sedrun of the Gotthard Base Tunnel can be mentioned, see Kovář et al., 2006) or, more commonly, right from the start. In order to avoid overstressing of the shotcrete and, at the same time, to allow it to participate in the structural system, special elements are inserted into longitudinal slots in the shotcrete shell (Fig. 1b, section d-d). The load - deformation behavior of such elements is in general characterized by yielding at a specific load level, thereby limiting the stress in the shotcrete shell. Two successfully applied types of deformable elements are the so-called “lining stress controllers” (Schubert 1996, Schubert et al. 1999) and the recently-developed "highly-deformable concrete" elements (Kovář, 2005). The “lining stress controllers” consist of co-axial steel cylinders which are loaded in their axial direction, buckle in stages and shorten up to 200 mm at a load of 150 - 250 kN. The "highly-deformable concrete" elements are composed of a mixture of cement, steel fibers and hollow glass particles. The glass particles collapse at a pre-defined compressive stress which is dependent on the composition of the concrete, thereby providing the desired deformability.

The number, the size s (in circumferential direction, Fig. 1b) and the deformability of the compressible elements (or the sliding ways of the steel set connections) will determine the possible reduction of the circumference of the lining during the yielding phase, limiting the amount of radial displacement that can occur without damaging the lining. The elements selected must therefore be compatible with the planned amount of over-excavation. The latter represents an important design parameter. If it is too low, costly and time-consuming re-profiling works will be necessary. On the other hand, if it is too high, the over-profile will have to be filled by the cast-in-situ concrete of the final lining.

The idea behind all yielding support systems is that the ground pressure will decrease if the ground is allowed to deform. During construction, the support system and the amount of over-excavation can be adapted to changes in squeezing intensity through the use of advance probing, monitoring results and observations made in tunnel stretches excavated previously. In the planning phase, however, decision-making has to rely solely upon experience and geomechanical calculations. When estimating the required amount of over-excavation, the usual approach is to consider a tunnel cross-section far behind the tunnel face and to assume plane strain conditions. Where there is rotational symmetry, the plane strain problem is mathematically one-dimensional. The so-called characteristic line of the ground (also referred
to as the "ground response curve" (GRC), Panet & Guenot, 1982) expresses the relationship between the radial stress \( p \) and the radial displacement \( u \) of the ground at the excavation boundary (Fig. 2a). Closed-form solutions exist for the ground response curve in a variety of constitutive models. The ground response curve can be employed for estimating the radial convergence \( u(\infty) \) of the ground that must occur in order for the ground pressure to decrease to a chosen, structurally manageable value \( p(\infty) \).

Figure 2a illustrates the ground - support interaction using the characteristic line method. The solid polygonal line in Figure 2a represents an idealized model of the characteristic line of a yielding support. Phase I is governed by the stiffness \( k_I \) of the system up to the onset of yielding. In Phase II the support system deforms under a constant pressure \( p_y \). When the amount of over-excavation \( u_r \) is used-up, the system is made practically rigid (stiffness \( k_{III} \)), e.g. by applying shotcrete, with the consequence that an additional pressure builds up upon the lining (Phase III). Figure 2b shows schematically the development of radial pressure along the tunnel. The amount of over-excavation \( u_r \) and the yield pressure \( p_y \) are the main design parameters for a yielding support, while the stiffnesses \( k_I \) and \( k_{III} \) are of secondary importance for the cases that are relevant in practical terms.

The intersection point of the ground response curve with the characteristic line of the support (Fig. 2a, point A) fulfills the conditions of equilibrium and compatibility and shows the radial ground convergence and the final pressure acting upon the lining far behind the face.

The first problem with this approach is that the radial convergence that has occurred ahead of the tunnel face and over the unsupported span \( u(e) \) can not be determined under the assumption of plane strain conditions (Fig. 2a). This pre-deformation \( u(e) \) introduces an element of uncertainty into the estimation of the required amount of over-excavation. This uncertainty is particularly serious in the case of heavily squeezing ground because its behavior is highly non-linear and, consequently, small variations in the deformation will have a large effect on the pressure.

A second, more fundamental problem is that all plane strain solutions (whether closed-form solutions for the ground response curve or numerical simulations involving a partial stress release before lining installation) assume that the radial stress at the excavation boundary decreases monotonically from its initial value (far ahead of the face) to the support pressure (far behind the face), while the actual load history will include an intermediate stage with a complete unloading of the excavation boundary in the radial direction: the radial boundary stress is equal to zero over the unsupported span \( e > x > 0 \) between the tunnel face and the installation point of the lining (Fig. 2b). Cantieni and Anagnostou (2007) have shown that the assumption of a monotonically decreasing radial stress may lead (particularly under heavily squeezing conditions) to a more or less serious underestimation of ground pressure and deformation. The actual \( (u(\infty), p(\infty)) \) - points prevailing at the equilibrium far behind the face.

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**Figure 2.** (a) Ground - support interaction in the rotational symmetry model under plane strain conditions; (b) Development of ground pressure and deformation along the tunnel wall.
are consistently located above the ground response curve (point B in Fig. 2a). This means that, a plane strain analysis cannot reproduce at one and the same time both the deformations and the pressures: in order to determine the ground pressure through a plane strain solution, the deformations have to be underestimated (point C in Fig. 2a) or, vice versa, in order to determine the deformations, the ground pressure has to be underestimated (point D in Fig. 2a). This is particularly relevant from the design standpoint for a yielding support, because in this case one needs reliable estimates both of the deformations (for determining the amount of over-excavation) and of the pressures (for dimensioning the lining).

Further, the investigations have shown that the higher the yield pressure of the support, the lower will be the final pressure (the points B and B' in Figure 2a apply to yield pressures of \( p_y \) and \( p_y' \), respectively). Using the characteristic line method one would obtain exactly the same intersection point (point A in Fig. 2a), i.e. the same values of pressure and deformation.

Moving on from these results, which are not only quantitatively but also qualitatively different from the ones obtained by plane strain analyses, the present paper investigates the interaction between yielding supports and squeezing ground by means of numerical analyses that take into account the evolution of the spatial stress field around the advancing tunnel heading. The influence of the yield pressure on the final rock pressure has never before been investigated. The literature contains only a few project-specific three-dimensional numerical studies (e.g., Amberg 1999, Fellner and Amann 2004).

All of the computations in the present paper apply to the case of rotational symmetry. The underlying assumptions are: a cylindrical tunnel; uniform support pressure over the circumferential direction (but of course variable in the longitudinal direction); a homogeneous and isotropic ground; a uniform and hydrostatic initial stress field. The mechanical behavior of the ground was modeled as linearly elastic and perfectly plastic according to the Mohr-Coulomb yield criterion, with a non-associated flow rule. The lining was modeled as a radial support having a deformation-dependent stiffness \( k = dp/du \). Tunnel face support has not been taken into account, and nor have any time dependencies of the behavior of the ground or of the shotcrete lining. The numerical solutions have been obtained by means of the Finite Element Method. The advancing tunnel heading was handled using the so-called "steady state method" (Corbetta 1990, Anagnostou 2007).

THE EFFECT OF THE STRESS PATH

This chapter provides some useful insights into the effect of the stress path by a comparative analysis of two hypothetical support cases. Both of them represent an extreme case of yielding supports. Consider firstly a stiff, almost rigid lining (Stiffness of the shotcrete \( E_l = 30 \) GPa, thickness of the shotcrete shell \( d = 35 \) cm), which is installed at a distance of \( e = 24 \) m behind the tunnel face (the other parameters are given in Table 1). Figures 3a and 3b show on their left hand sides ("Support case 1") the support pressure development in the longitudinal direction and the radial displacement of the ground at the excavation boundary, respectively. The final ground pressure \( p(\infty) \) prevailing far behind the lining installation point amounts to 700 kPa, while the radial convergence \( u(\infty) - u(0) \) of the opening is about 52 cm. As it is assumed that the 24 m long span between the face and the lining has been left unsupported, this case is rather theoretical. It is equivalent, however, to a yielding support which is installed immediately at the tunnel face and which is able to accommodate a radial convergence of at least 52 cm while offering only negligible resistance to the ground in the deformation phase.
Consider now ("Support case 2") a yielding support that is installed directly at the face and offers a resistance of \( p_y = 700 \text{ kPa} \) during the deformation phase (i.e., the yield pressure is assumed to be as high as the final ground pressure \( p(x) \) in the first case). Let us, additionally, assume that the support is able to accommodate a sufficiently large deformation so that the ground — support system reaches equilibrium at the yield pressure \( p_y \). The final ground pressure will therefore also amount to 700 kPa in this case. As the final radial convergence of the opening amounts in this case to about 24 cm (Fig. 3b, right hand side), the assumption made (that the system reaches equilibrium at the yield pressure) presupposes that the support is able to accommodate a convergence of at least 24 cm in the deformation phase. The only difference between the two support cases mentioned above lies in the stress history: in the first case the pressure starts to develop at a distance \( e = 24 \text{ m} \) from the face and reaches its final value of 700 kPa far behind the face, while in the second case the support pressure of 700 kPa acts right from the start (see Fig. 3a).

This difference leads to a considerable variation in the longitudinal deformation profiles, particularly in the region behind the tunnel face (Fig. 3b): the final radial displacement of the ground at the excavation boundary is twice as high in support case 1 as it is in support case 2, while the displacements ahead of the tunnel face are approximately equal. In view of the support pressure distributions in Figure 3a, this result makes sense intuitively, while clearly differing from what one might expect under the characteristic line method. According to the latter, the radial displacement should be the same in both support cases, because the relationship between ground pressure and deformation is unique (the ground response curve is one and the same for both cases as it does not depend on the support behavior) and both cases have the same final ground pressure \( p(x) \) of 700 kPa. For this pressure, a plane strain analysis would predict a radial displacement \( u_{2D} \) of approximately 35 cm. This value agrees well with the results of the axisymmetric analysis obtained for support case 2, but underestimates considerably the final displacement in support case 1 (Fig. 3b). For the latter, the axisymmetric analysis leads to a radial displacement, which is closer to the plane strain displacement for an unsupported opening (marked by \( u_{2D}(p = 0) \) in Fig. 3b).

Similar observations can be made regarding the extent of the plastic deformation zone (Fig. 3c): in support case 2 the final radius of the plastic zone amounts to 10.5 m, which is very close to the plane strain analysis result for a support pressure of 700 kPa (\( \rho_{2D} = 11.0 \text{ m} \)). In support case 1 the plastic zone extends up to a radius of 14.5 m. This value is closer to the plane strain analysis prediction for an unsupported opening (15.6 m).

Figure 3d shows the evolution of the stresses \( (\sigma_{xx}, \sigma_{yy}, \sigma_{bb}, \sigma_{rs}) \) at a point located at the tunnel boundary. According to Figure 3d, the stress paths ahead of the tunnel face are very similar for the two support cases: with the approaching excavation, the axial stress \( \sigma_{xx} \) decreases from

<table>
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<th>Table 1. Model parameters</th>
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<td>Parameter</td>
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<td>Initial stress</td>
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<td>Depth of cover</td>
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<td>Unit weight of ground</td>
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<td>Tunnel radius</td>
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<td>Young’s Modulus (Ground)</td>
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<td>Dilatancy angle (Ground)</td>
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Ground evolution can be considered as an example of a finite element analysis (FEA) to determine the stress and strain distribution around a tunnel. The FEA model takes into account the ground properties and the support system, allowing for a realistic simulation of the excavation process. The results show the effectiveness of the support system in preventing excessive ground deformation and ensuring the stability of the excavation.

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The table above lists the model parameters for the analysis, indicating the initial stress, depth of cover, unit weight of ground, tunnel radius, Young’s Modulus, Poisson’s ratio, angle of internal friction, cohesion, and dilatancy angle. Each parameter is crucial in understanding the behavior of the ground and the support system under different conditions.
its initial value $p_0$ (in the field far ahead of the face) to zero (at the face). In the region far ahead of the tunnel face, a stress concentration can be observed, while close to the face the axial confinement is lost to such a degree that the core yields and, as a consequence of Coulomb's yield criterion, the radial stress $\sigma_r$ and the tangential stress $\sigma_\theta$ decrease. Immediately after excavation the radial stress becomes equal to zero, while both the tangential and the axial stresses become equal to the uniaxial compressive strength of the ground $f_c$. This stress state persists in support case 1 over the entire unsupported span. The radial stress increases to its final value of 700 kPa after the installation of the practically rigid lining, because the latter hinders further ground convergence. In support case 2, the radial stress
increases (due to the initial support stiffness $k_t$, see Fig. 2a) practically immediately after support installation to the yield pressure $p_t$ and remains constant thereafter.

Plane strain analysis assumes a monotonic decrease in the radial pressure from its initial value (that prevails in the natural state far ahead the face) to the final value (that prevails after excavation far behind the face), while both support cases considered here involve an intermediate stage characterized by a complete unloading of the excavation boundary in the radial direction. The difference between the two support cases lies in the length of this intermediate stage: in support case 1, the biaxial stress state persists over the entire, 24 m long unsupported span, while in support case 2, the stress state remains biaxial only very briefly, almost instantaneously at the face (Fig. 3d). This is why the intermediate biaxial stress state (zero radial stress) governs both the extent of the plastic zone and the magnitude of ground deformations in support case 1 (the results are closer to the plane strain predictions for an unsupported opening), while the final triaxial stress state (700 kPa radial stress) governs the support case 2 results (the final radius of the plastic zone and the radial displacement agree well with the plane strain results for an actual final support pressure of 700 kPa).

Note that for an elastic ground both support cases would lead to the same final ground displacements. The final ground displacement does not depend on how the radial stress at the excavation boundary evolves before reaching its final value. The above mentioned effect can only be observed when considering stress-path dependent material behavior. However, the case of elastic ground behavior is interesting only from the theoretical point of view, because large convergences that necessitate a yielding support are always associated with an overstressing of the ground and cannot be reproduced by assuming elastic material behavior.

The results of a parametric study with different values for the unsupported span $e$ provide additional evidence for the significance of the intermediate stage in elasto-plastic ground. Figure 4a shows the ground pressure $p(\infty)$ and the radial displacement of the ground $u(\infty)$ at the final equilibrium prevailing far behind the face for the two support cases discussed above as well as, for the purpose of comparison, the ground response curve obtained under plane

![Figure 4](imageLink)
strain conditions. Note that each point under support case 1 applies to another value of the unsupported length $e$ (the $e$ - values are reported besides the ordinate axis), while each point under support case 2 applies to another value of the yield pressure $p_y$ (the $p_y$ - values are equal to the final pressures $p(\infty)$ on the ordinate axis). The diagram illustrates quite plainly the non-uniqueness of the ground pressure vs. ground displacement relationship: the equilibrium points for support case 1 are consistently located above the ground response curve, while all of the support case 2 results agree well with the plane strain predictions.

Figure 4b is more useful from a practical point of view as it shows the radial convergence $u(\infty) - u(0)$ of the opening instead of the radial displacement $u(\infty)$ of the ground. This diagram includes only the results of the axisymmetric computations because plane strain analysis yields the total radial displacement $u(\infty)$, but not the pre-deformation $u(0)$. Note that $u(\infty) - u(0)$ represents the minimum amount of over-excavation that is necessary in order to preserve the clearance profile. The purpose of a yielding support is to reduce ground pressure to a pre-defined, structurally manageable level. Figure 4b points to the interesting conclusion that the amount of over-excavation - an essential design parameter - does not depend only on the ground quality and on the desired design load level, but also on the characteristics of the support. In order to reduce the ground pressure to 0.7 MPa, for example, the amount of over-excavation has to be 24 - 52 cm depending on the support system (see points A and B in Fig. 4b). On the other hand, for a given amount of over-excavation, the final ground load will depend on the support characteristics as well. For an over-excavation of, e.g., 24 cm, the final ground pressure in support case 1 will amount to 2.4 MPa, that is three to four times higher than the load in case 2 (see points B and C in Fig. 4b). This difference is considerable from a design standpoint. Note that support case 1, which necessitates a larger amount of over-excavation for a given design load level (or attracts a higher ground load for a given amount of over-excavation), is equivalent to the case of a deformable support with negligible yield pressure. So Figure 4b actually indicates that a high yield pressure is favorable in terms of the final ground load and the amount of over-excavation. Moving on from this finding, in the next section we will examine the influence of yield pressure in more detail.

**THE INFLUENCE OF YIELD PRESSURE AND YIELD DEFORMATION**

In this Section we investigate numerically the effects of the main yielding support characteristics. All of the numerical analyses have been carried out for the parameters of Table 1 and an unsupported span of $e = 1$ m. For the sake of simplicity, the tunnel radius was kept fixed, i.e., it was not increased by the amount of over-excavation that is required in order to preserve the clearance profile when the support deforms. Ten different characteristic lines of the tunnel support have been investigated. The yielding supports are assumed to deform at a constant pressure $p_y$ of 0, 150, 425, 850 and 1200 kPa up to a radial displacement of $u_r = 0.15$ m (cases O, A, B, C and D) or 0.30 m (cases O', A', B', C' and D') and to be practically rigid after the deformation phase ($k_{III} = 656$ MPa/m). The stiffness before yielding $k_I$ is for all cases set equal to 100 MPa/m. Yielding pressures like the ones assumed for cases A, A', B and B' are today realistic. Recently developed ductile concrete elements of particularly high yield strength (up to 20 MPa, Solexperts 2007) make higher yield pressures (such as in cases C and C') seem feasible at least in principle. Cases D and D' are only of theoretical interest (the assumed yield pressure is unrealistically high) and are considered here only in order to show complete model behavior.

Figure 5a shows the equilibrium points $(u(\infty), p(\infty))$ of the ground and additionally, for the purposes of comparison, the ground response curve under plane strain conditions (solid line GRC). As the effect of yield deformation $u_y$ is well known, attention is paid here to the effect of yield pressure $p_y$. We examine how the equilibrium point changes in relation to the yield
pressure $p_y$ (cases O, A, B, ...) starting with a support that can undergo a radial displacement of $u_r = 15$ cm without offering any resistance to the ground (case O). The support O starts to develop pressure only after the utilization of the deformation margin $u_r$. As can be seen from Figure 5b (curve O), this happens at a distance of about 7 m behind the tunnel face. So, support O is structurally equivalent to the case of a practically rigid lining with a 7 m long unsupported span (cf. Support Case 1 discussed in Section 2). Due to the stress path dependency of the ground behavior (cf. Section 2), the equilibrium point for this case is located above the plane strain response curve. When the yield pressure increases, the final ground pressure will decrease (Fig. 5b) and the equilibrium point will move towards the ground response curve (O→A→B, see Fig. 5a), i.e. the deviation from the ground response curve will become smaller at higher yield pressure values. Note that the ground deformation remains approximately constant as it is governed by the yield deformation $u_r$ which is the same for the support cases O, A and B. At a sufficiently high yield pressure (850 kPa in this numerical example), the equilibrium point reaches the ground response curve (case C, Fig. 5a). In case C the ground - support system reaches equilibrium just before reaching the final rising branch of the characteristic line of the support, i.e. the deformation margin $u_r = 0.15$ m is just used up. At yield pressures higher than in case C, the amount of over-excavation is not utilized completely (Fig. 8a, case D) and the system reaches equilibrium at a ground pressure which is equal to the yield pressure. Consequently, the ground pressure, after reaching a minimum value (case C), increases with the yield pressure and, as discussed in Section 2, the equilibrium points approximately follow the ground response curve (Fig. 5a, C→D). A further increase of the yield pressure finally leads to the equilibrium point S in Figure 5a. In this case the yield pressure has been set so high that the ground - support system reaches equilibrium before the support yields. Similar conclusions can be drawn from an examination of the numerical results for the larger deformation margin of $u_r = 0.30$ m (cases O', A', B', ...).

It is remarkable that a similar reduction in the final ground pressure can be achieved not only by installing a support that is able to accommodate a larger deformation (which is a well-known principle), but also by selecting a support that yields at a higher pressure (see, e.g., cases A, C and C' in Fig. 5a).

CLOSING REMARKS

We have shown that an analysis of the ground – yielding support interaction that takes into account the stress history of the ground leads to conclusions which are qualitatively different from those obtained through plane strain analysis. The ground pressure developing far behind the tunnel face in a heavily squeezing ground depends considerably on the amount of support.
resistance during the yielding phase. The higher the yield pressure of the support, the lower will be the final load. A targeted reduction in ground pressure can be achieved not only by installing a support that is able to accommodate a larger deformation (which is a well-known principle), but also through selecting a support that yields at a higher pressure. Furthermore, a high yield pressure reduces the risk of a violation of the clearance profile and increases safety level against roof instabilities (loosening) during the deformation phase.

REFERENCES


