ABSTRACT. The paper illustrates the importance of spatial effects in tunnelling by addressing a number of practical design and analysis issues. It also provides an overview of recent research at ETH Zurich dealing with the evolution of the stresses and deformations around the advancing face. At the same time, it shows that taking due account of spatial effects leads to results which may be qualitatively different to those obtained through plane strain analyses.

1 INTRODUCTION

In tunnel design it is essential for a number of questions to take due account of the evolution of the stress and deformation fields in the ground around the advancing tunnel face. It is well known, for example, that the main source of settlement in closed-shield tunnelling is localized in the shield area. The case of squeezing ground provides another example from a completely different context. When tunnelling with a shielded TBM, limited levels of convergence will not cause problems, thanks to the gap between the shield and the surrounding ground. If the gap closes, however, pressure will start to develop upon the shield. In this way, squeezing ground may slow down or even obstruct TBM advance. The quicker the development of the convergences, the higher the risk of shield jamming. The rate of convergence close to the tunnel heading is also important for conventional tunnelling; rapidly developing squeezing may slow down the advance rate considerably, as the installation of the support required for controlling the ground interferes with the actual excavation work.

The recognition of such spatial effects – and of the importance of taking them into account at the design stage – came relatively recently in tunnelling practice. Professor Lunardi was among the first engineers not only to attempt to understand what happens in the ground ahead of the tunnel face, but also to appreciate the importance of spatial effects in his tunnel designs. Today’s celebration therefore gives us an opportunity to revisit some related issues that are relevant from a practical perspective and to give an overview of recent research at ETH Zurich into the evolution of stress and deformation around the advancing tunnel face.

The use of a planar statical system to estimate ground deformations or pressures in the vicinity of the face introduces uncertainties as a result of the need to make a priori assumptions about the development of deformations and stresses in the longitudinal direction. Plane models also fail to take account of important information concerning the ground response to excavation. In more specific terms, the current paper shows that if we take adequate account of the third dimension – the stress history of the ground together with the sequence of excavation
and support installation – we will obtain results, which may be markedly different – both quantitatively and qualitatively – to those obtained through plane strain analyses. We might add that these results are surprising at first glance and cannot be reproduced from two-dimensional models.

The paper subsequently considers the development of surface settlement during the continuous excavation of a shallow tunnel crossing a low-permeability saturated ground (Section 4), and continues with the issue of shield jamming in overstressed rock (Section 5) before closing with some results obtained on the interaction between yielding supports and squeezing ground (Section 6). Before addressing these practical questions, however, some general issues are discussed concerning the computational mechanics of an advancing tunnel heading (Section 2) and a fundamental shortcoming of plane strain computational models (Section 3).

2 NUMERICAL MODELLING OF CONTINUOUS EXCAVATION

The numerical modelling of an advancing tunnel heading is particularly demanding in the case of time-dependent ground behaviour. The time-dependency of ground behaviour may be linked to consolidation, creep and, in some rocks, chemical processes as well. It manifests itself in a variety of ways depending upon both the type of ground and the construction method, and may have important implications for the construction process or the life of a tunnel.

Creep is associated with the rheological properties of the ground and becomes evident if the ground is overstressed – particularly as the failure state approaches. It is, therefore, of paramount importance in the case of weak rock under high stress (squeezing conditions).

The time-dependency of low-permeability soft ground is due mainly to transient seepage flow processes which are triggered by tunnel excavation and which develop slowly over the course of time. The long-term deformations of the ground generally include changes to its pore volume and water content (the latter requires 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, however, generates excess pore pressures. As these will be higher in the vicinity of the tunnel than further away, seepage flow starts to develop. The excess pore pressures therefore dissipate over the course of time, thereby altering the effective stresses and leading to additional time-dependent deformations (consolidation). The permeability of the ground has a decisive effect on the rate of pore pressure dissipation and thus on the time-development of ground deformations or (where the latter are constrained by a lining) ground pressure.

In geological conditions where there is pronounced time-dependent ground behaviour during construction (e.g., shallow tunnels through clay deposits or deep tunnels through weak rock), the advance rate greatly influences the development of ground pressure and deformation in the region around the working face because the deformations caused by creep or consolidation are superimposed upon those caused by the three-dimensional redistribution of stress that results from excavation.

As far as consolidation processes are concerned, the higher the advance rate and the lower the permeability, the less dissipation there will be in respect of pore pressures in the vicinity of the face (leading to so-called "undrained" conditions) and, consequently, the deformations will be smaller. If, on the other hand, the permeability is high and the advance rate low, drained conditions will prevail in the vicinity of the working face, and these are less favour-
able. Depending on the ratio of advance rate to permeability, the ground response will be un-
drained, drained or somewhere in between. The ratio of advance rate $v$ to permeability $k$ gov-
erns stability, deformation and the ground pressure acting upon a lining or a shield in the vi-
cinity of the face.

Similar considerations apply to creep (Ghaboussi and Gioda, 1977): the lower the advance
rate, the greater the deformation. In the borderline case of a very high advance rate, only small, elastic deformations will develop in the vicinity of the face. A high advance rate is therefore advantageous, as the ground deformations close to the face will be smaller, regard-
less of whether the reason for the time-dependency is creep or consolidation.

An advancing tunnel heading is usually simulated (cf., e.g., Franzius and Potts, 2005) as a
successive removal of soil elements and an addition of elements representing the lining or the
shield, i.e. as a statical system that changes stepwise according to a sequence of excavation
and support operations. Such an approach, although quite natural, can be very problematic in
the case of time-dependent ground behaviour. The reason for this is the previously mentioned
simultaneous redistribution of spatial stress around the advancing face, together with consoli-
dation or creep processes in the ground. Since high deformation gradients and pore water
pressure gradients prevail in the vicinity of the tunnel face, either the finite element mesh has
to be fine everywhere along the tunnel axis or adaptive re-meshing must be carried out for
each excavation step in the simulation. The analysis therefore becomes extremely time-
consuming (even in the case of linear material behaviour) and generally presents serious prob-
lems in terms of both numerical accuracy and stability.

It should be noted, however, that in the case of uniform conditions in the direction of tunnel-
ling (Figure 1), the stress fields and deformation fields are steady with regard to the tunnel
heading, i.e. they "advance" together with the face in the direction of excavation. For this
large class of problems, the step-by-step solution method is highly inefficient, as it ap-
proaches the steady state asymptotically after the simulation of several excavation steps. The
advancing heading problem can instead be solved by means of a single computational step, i.e. without the need for integration in the time-domain. The central idea in respect of achieving this can be traced back to the work of Nguyen-Quoc & Rahimian (1981) on crack propagation in elastoplastic media and it involves eliminating the time-coordinate from the equations governing the steady state by carrying out appropriate transformations. Since the stress, pore pressure and deformation fields are apparently time-independent for an observer moving with the tunnel face, the obvious solution is to re-formulate the continuum-mechanical equations in a frame of reference that is fixed to the advancing heading (co-ordinate \( x_1^* \) in Figure 1). One might almost say that this method, rather than attempting to model the advancing tunnel face in a naturalistic way, considers the case of a mountain moving around a spatially-fixed tunnel face.

Corbetta (1990) and Anagnostou (1993) followed this approach when solving the viscoplastic tunnelling problem and when analysing transient seepage flow, respectively, while a recent work (Anagnostou, 2007) has extended this method for the coupled problem of tunnel excavation through a porous, saturated, elastoplastic ground. As shown in this work, almost all continuum-mechanical equations remain valid in a face-fixed co-ordinate system, the only exception being the mass balance equation, which is enhanced by one additional term incorporating the ratio \( v/k \) of advance rate to permeability as a parameter:

\[
\nabla \nabla h = -\frac{v}{k} \frac{\partial \varepsilon_{vol}}{\partial x_1^*},
\]

where \( h \) and \( \varepsilon_{vol} \) denote the hydraulic head and the volumetric strain of the ground, respectively, while \( x_1 \) is the excavation direction (Figure 1). The numerical solution of the enhanced coupled seepage flow and stress analysis equations yields the steady state displacement and hydraulic head fields around the advancing tunnel face straightforwardly in a single step, i.e. without the need for a time-iteration. The difficulties associated with the accuracy and stability of a marching scheme in the time-domain therefore do not arise in this method.

3 THE NON-UNIQUENESS OF THE GROUND RESPONSE CURVE

Computational models based on planar statical systems consider a tunnel cross-section far behind the tunnel face and 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" (Panet and Guenot, 1982) – expresses the relationship between the radial stress \( p \) and the radial displacement \( u \) of the ground at the excavation boundary. Assuming equilibrium and compatibility between ground and support, the ground response curve can be employed in combination with the characteristic line of the support in order to estimate the radial convergence of the ground that must occur in order for the ground pressure to decrease to a chosen, structurally manageable value.

One fundamental problem of this approach 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 the initial value (which prevails far ahead of the face) to the support pressure (which develops far behind the face). However, the actual load history includes a complete unloading of the excavation boundary in the radial direction over the unsupported span and a subsequent re-loading of the tunnel boundary starting with the installa-
tion of the lining. Cantieni and Anagnostou (2009a) have shown that the assumption of a monotonically decreasing radial stress may lead (particularly under heavily squeezing conditions and assuming an elastoplastic ground behaviour) to a more or less serious underestimation of ground pressure and deformation. The actual final equilibrium points – i.e., the tunnel wall displacement \( u(\infty) \) and the ground pressure \( p(\infty) \) prevailing far behind the face – are consistently located above the ground response curve.

Figure 2 shows the ground response curve obtained by a closed-form, plane strain solution (the solid line marked by "GRC"), as well as the results of axially-symmetric numerical calculations for the case of heavily squeezing ground (the points marked by circles, e.g. \( P_1, P_2, \ldots \)). The mechanical behaviour of the ground was modelled as isotropic, linearly elastic and perfectly plastic according to the Mohr-Coulomb yield criterion, while the lining was taken as an elastic radial support with stiffness \( dp/du = k \). All of the model parameters can be found in Table 1.

### Table 1. Assumed model parameters for the examples of Figures 2 and 15.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial stress ( p_0 )</td>
<td>12.5 MPa</td>
</tr>
<tr>
<td>Tunnel radius ( \alpha )</td>
<td>4 m</td>
</tr>
<tr>
<td>Young’s Modulus (Ground) ( E_R )</td>
<td>1000 MPa</td>
</tr>
<tr>
<td>Poisson’s ratio (Ground) ( \nu )</td>
<td>0.3</td>
</tr>
<tr>
<td>Angle of internal friction (Ground) ( \varphi )</td>
<td>25°</td>
</tr>
<tr>
<td>Cohesion (Ground) ( c )</td>
<td>500 kPa</td>
</tr>
<tr>
<td>Dilatancy angle (Ground) ( \psi )</td>
<td>5°</td>
</tr>
</tbody>
</table>

Figure 2. Ground response curve under plane strain conditions (GRC) and ground response points \( (P_1, P_2, \ldots) \) far behind the tunnel face for different unsupported spans \( e \) and lining stiffnesses \( k \) (after Cantieni and Anagnostou, 2009a).
As illustrated by Figure 2, plane strain analysis systematically underestimates ground pressures and deformations. The deviation is due to the inability of any plane strain model to map, (i), the complete radial unloading of the excavation boundary over the unsupported span and, (ii), the subsequent increase in radial stress following the installation of the lining.

(i) Let us consider first the effect of an unsupported span $e$ for a fixed value of lining stiffness $k$. Points $P_2$ and $P_4$ in Figure 2 involve a stiff lining ($k = 1$ GPa/m) installed at $e = 1$ or 8 m, respectively. The deviation from the ground response curve is larger in the case of a longer unsupported span. This is because the extent of the plastic zone and the magnitude of deformation are governed by the biaxial stress state (i.e. a radial stress equal to 0) prevailing over the long unsupported portion of the tunnel, rather than by the final stress state.

(ii) The effects of lining stiffness can be observed in points $P_3$ and $P_4$. The ground response point $P_3$ that results from the axisymmetric calculation for a soft support ($k = 0.1$ GPa/m, installed at $e = 1$ m) is closer to the plane strain ground response curve (GRC) than the ground response point $P_4$ that applies to a stiff support ($k = 1$ GPa/m, installed also at $e = 1$ m). In general, the stiffer the lining (the value of the unsupported span $e$ being fixed), the larger will be the final ground pressure and thus, because of the more distinct stress reversal, the larger will be the deviation from the ground response curve. The net deviation is, of course, governed both by effects (i) and (ii). This is particularly apparent when considering long unsupported spans. The error introduced by the plane strain assumption decreases again at very large values of $e$ (see, e.g., solid line for $k = 1$ GPa/m in Figure 2).

In conclusion, the actual relation between ground pressure and deformation is not unique. This conflicts with the plane strain results and suggests that a plane strain analysis cannot reproduce correctly at one and the same time both the deformations and the pressures. This is particularly relevant not only from the theoretical point of view but also with respect to practical design issues such as the assessment of a TBM-drive in squeezing ground (Section 5) or the design of a yielding support (Section 6). In both cases the tunnel engineer needs a reliable estimate of the ground deformations (in order to determine the amount of overcut or overexcavation) and of the ground pressures (in order to determine the required thrust force or the dimensions of the lining).

4 SETTLEMENT DEVELOPMENT IN SHALLOW SOFT GROUND TUNNELLING

Field measurements show clearly that settlements induced by tunnelling through low-permeability clay deposits may increase for several months after excavation. The settlement trough widens and deepens over the course of time (O’Reilly et al., 1991). As explained in Section 2, the advance rate and the ground permeability greatly influence the development of ground pressure and deformation when tunnelling through low permeability ground (e.g. clay deposits). Depending on the ratio of advance rate $v$ to ground permeability $k$, the ground response will be undrained (favourable), drained (less favourable) or somewhere in between. As a rule, for a given low ground permeability $k$, a higher as possible advance rate $v$ is advantageous. This Section investigates the opposed effects of consolidation and advance rate by means of numerical calculations for a cylindrical shallow tunnel crossing homogeneous ground (Figure 3). The advancing tunnel heading is taken into account as outlined in Section 2. Thanks to the economy and numerical stability of the method outlined in Section 2, it was possible to perform a comprehensive parametric study, clarifying the effect of advance rate on the transversal and longitudinal settlement troughs.
Table 2: Parameter values for the numerical example.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel radius</td>
<td>$R$ 5 m</td>
</tr>
<tr>
<td>Depth of cover (from tunnel axis)</td>
<td>$H$ 20 m</td>
</tr>
<tr>
<td>Elevation of water table (from tunnel axis)</td>
<td>$H_W$ 20 m</td>
</tr>
<tr>
<td>Advance rate</td>
<td>$v$ variable m/s</td>
</tr>
<tr>
<td>Coefficient of horizontal initial stress</td>
<td>$K$ 0.50 MPa</td>
</tr>
<tr>
<td>Young's Modulus (ground)</td>
<td>$E$ 50 MPa</td>
</tr>
<tr>
<td>Poisson's ratio (ground)</td>
<td>$\nu$ 0.15 -</td>
</tr>
<tr>
<td>Permeability coefficient</td>
<td>$k$ variable m/s</td>
</tr>
<tr>
<td>Total unit weight (ground)</td>
<td>$\gamma$ 20 kN/m³</td>
</tr>
<tr>
<td>Unit weight of water</td>
<td>$\gamma_W$ 10 kN/m³</td>
</tr>
<tr>
<td>Compressibility of water</td>
<td>$c_W$ 0.4 GPa⁻¹</td>
</tr>
<tr>
<td>Porosity</td>
<td>$n$ 0.10 -</td>
</tr>
<tr>
<td>Distance between lining and face</td>
<td>$e$ 1 m</td>
</tr>
<tr>
<td>Lining thickness</td>
<td>$d$ 0.20 m</td>
</tr>
<tr>
<td>Young's Modulus (lining)</td>
<td>$E_L$ 15 GPa</td>
</tr>
<tr>
<td>Poisson's Number (lining)</td>
<td>$\nu_L$ 0.15 -</td>
</tr>
</tbody>
</table>

Figure 3. Computational model for a shallow tunnel.
In the numerical examples, both the ground and the lining have been modelled as linearly elastic materials. Table 2 summarises the material constants and the other model parameters. The tunnel heading and the unsupported span of 1 m were modelled as seepage faces under atmospheric pressure. We assume, furthermore, that the groundwater recharge from the surface (e.g. through rainfall or an adjacent river or lake) is sufficient to maintain the elevation of the water table at a constant level. The draining action of the tunnel then results in decreasing pore water pressures in the ground surrounding the tunnel. The decrease in the pore pressures may be temporary or permanent, depending on the permeability of the lining. We have chosen to consider the case of a practically impervious lining (i.e. a no-flow boundary condition will apply to the supported section of the tunnel boundary).

Figure 4 shows the contour lines of vertical displacement for \( v/k \rightarrow 0 \) (i.e., either during excavation in a high permeability ground or at the steady state achieved during a long excavation standstill). A crater-like depression of the surface can be observed above the tunnel face (Figure 4): the soil surface experiences firstly a settlement as the tunnel face approaches ("Phase 1") and subsequently a heave ("Phase 2"). The Phase 1 settlement is caused by spatial stress redistribution and by the decreasing pore pressures (consolidation) in the vicinity of the heading. After the installation of the impervious lining, the pore pressures increase gradually and reach their natural values again at a certain distance behind the face. Consequently, the effective stresses decrease ("unloading", "swelling"). The simplified assumption of linear elastic behaviour (with the same levels of stiffness for loading and unloading) overestimates the swelling strains, with the result that the Phase 2 heave compensates for a considerable portion the Phase 1 settlement. With a more realistic material model (which takes account of the stiffer ground response to unloading), this effect would be less pronounced and the final settlement would be governed by the deformations around the tunnel heading.

![Figure 4. Contour lines of vertical displacement \( u_y \) at a steady state (\( v/k \rightarrow 0 \)).](image)
Figure 5. Contour lines of surface settlement for different values of the advance rate $v$ (permeability $k = 10^{-8}$ m/s; excavation direction from right to left).
Figure 5 provides a more complete picture of the surface settlement for different advance rates $v$, while Figure 6 shows the effect of advance rate $v$ and of permeability $k$ on the longitudinal settlement trough. According to Figure 6, the conditions in the vicinity of the heading are practically undrained for $v/k > 10^3$ and practically drained for $v/k < 10^3$. Depending on the ratio $v/k$, the ground response lies somewhere between the undrained response and the drained response. The higher this ratio, the greater will be the distance from the advancing heading at which the deformations reach a steady state. In fact, the crater-like depression at the surface will occur only at high permeability values or, everything else being equal, at low advance rates.

High $v/k$-ratios reduce the time available for the consolidation process in the vicinity of the advancing face to have a positive influence on the settlement trough in the vicinity of the face (the higher the excavation rate, the lower the angular distortion at the surface) and to contribute to settlement limitation. This is particularly true for closed shield tunnelling as the deformations of the ground ahead of the face and around the shield (gap closure) represent by far the greatest sources of volume loss and surface settlement.
Squeezing ground may slow down or even obstruct TBM operation. Due to the geometrical constraints of the equipment, even relatively small convergences of one or two decimetres may lead to considerable difficulties with the machine (e.g., sticking of the cutter head, jamming of the shield, Figure 7) or in the back-up area (e.g., jamming of the back-up equipment, inadmissible convergences of the bored profile, damage to the tunnel support). If occurring over frequent tunnel intervals or if persisting over longer portions of a tunnel these difficulties may undermine the economic viability and the feasibility of a TBM drive. In this context, a number of very difficult situations (including the complete loss of a TBM) have occurred in the past (Ramoni and Anagnostou, 2006, 2009a).

Figure 7. Photograph of the ground and the shield taken during operations to free a single shielded TBM (courtesy of W. Burger, Herrenknecht AG).

TBM performance is the result of a complex interaction between the ground, the tunnelling equipment and the support. The advance rate represents not only the "result" of this interaction, but at the same time influences the interaction too. Identifying the relevant interfaces between the main system components and understanding their interactions is essential to the assessment of critical situations (Ramoni and Anagnostou, 2009a). Moreover, as the TBM types are different with respect to the thrusting system, the type of support and the presence or absence of a shield, different hazard scenarios have to be considered depending on the machine type (the advance rate, which plays an important role, also depends on the machine type).
In this respect, numerical analyses provide a valuable contribution to decision-making in the design process, as they indicate the magnitude of the key parameters. The application of axially symmetric or fully three-dimensional numerical models eliminates the uncertainties associated with the inherently three-dimensional nature of the problem under consideration. For example, such models allow the magnitude and the distribution of the ground pressure acting upon the shield and the lining to be determined, taking into account the support variation alongside the tunnel (shield, segments or otherwise) and the existence of a gap between the shield and the ground. Furthermore, as they take due account of the three-dimensional stress redistribution in the vicinity of the advancing face, they eliminate the errors introduced by assuming plane strain conditions (Section 3).

Concerning the risk of shield jamming, it is essential to have information on frictional forces when designing a new TBM and when assessing the feasibility of a proposed TBM drive. In the case of a second-hand TBM, checks also have to be made as to whether the installed thrust force is sufficiently high or whether the TBM has to be refurbished. Ramoni and Anagnostou (2009b) provided a number of theory-based decision aids which support rapid, initial assessments of thrust force requirements. A comprehensive parametric study has also been carried out using the finite element method and, based on the numerical results, dimensionless design nomograms have been worked out that cover the relevant range of material constants, in situ stress and TBM characteristics. This is the first time that such a systematic and thorough investigation of the combined effects of the parameters governing shield loading has been attempted. It should be noted that the parametric study involved about 12,000 numerical simulations and was possible only after developing efficient numerical solution methods (in terms of computer time and stability) specifically for this purpose (Section 2).

A total of 45 dimensionless nomograms were developed (see Figure 8 for an example). Each nomogram applies to a different TBM type and normalized shield length \( L/R \), and to a different value of the angle of internal friction \( \varphi \) and includes a band of curves (each curve corre-

![Figure 8. Nomogram for determining the required thrust force \( F_r \) for overcoming the shield skin friction of a single shielded TBM (Ramoni and Anagnostou, 2009b).](image-url)

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**Figure 8**. Nomogram for determining the required thrust force \( F_r \) for overcoming the shield skin friction of a single shielded TBM (Ramoni and Anagnostou, 2009b).
sponding to another value of the normalized uniaxial compressive strength $f'_c / \sigma_0$) showing the normalized required thrust force $F_r$ as a function of the dimensionless product of $E/\sigma_0$ by $\Delta R / R$, where $E$, $\sigma_0$, $\Delta R$ and $R$ denote the Young's modulus, the initial stress, the tunnel radius and the size of the gap between the bored profile and the extrados of the shield, respectively.

The nomograms make it possible to assess the feasibility of a TBM drive in a given geotechnical situation, to perform rapid sensitivity analyses with respect to the ground parameters and to evaluate potential design measures or operational measures such as reductions in shield length, the installation of a higher thrust force, increases in overcut or the lubrication of the shield surface, thus making a valuable contribution to the decision-making process.

Tunnelling practice has shown that interruptions in the TBM drive may be unfavourable in squeezing ground. In several cases, the TBM has become jammed only when there was a slowdown or standstill in the TBM drive, which suggests that maintaining a high advance rate and reducing standstill times may have a positive effect. Maintaining a high advance rate is of course a major goal for any TBM drive. Nevertheless, high advance rates should not be seen as a panacea for coping with squeezing. Firstly, they are difficult to achieve (especially in the case of poor quality ground). Secondly, ground deformations may develop very rapidly and very close to the working face (Ramoni and Anagnostou, 2009a). In such a situation, the advance rate would play a secondary role (the TBM would become jammed even if operated at the highest feasible speed). Thirdly, standstills in TBM operation cannot be completely avoided.

In the remainder of this Section we would like to discuss in quantitative terms the positive effects of a high advance rate and of short standstills during TBM tunnelling in consolidating, squeezing ground by means of mechanically-hydraulically coupled numerical calculations. Consider, as an example, a 500 m deep, $\varnothing$ 10 m tunnel driven by a 10 m long single shielded TBM in weak ground at a depth of 100 m beneath the water table (the model parameters are summarised in Table 3).

Table 3. Assumed model parameters for the examples in Figures 9 to 12.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial stress (overburden 500 m)</td>
<td>$\sigma_0$ 12.5 MPa</td>
</tr>
<tr>
<td>Initial pore pressure (water depth 100 m)</td>
<td>1 MPa</td>
</tr>
<tr>
<td>Tunnel radius</td>
<td>$R$ 5 m</td>
</tr>
<tr>
<td>Permeability</td>
<td>$k$ $10^{-9}$ m/s</td>
</tr>
<tr>
<td>Skin friction coefficient (static)</td>
<td>$\mu$ 0.40</td>
</tr>
<tr>
<td>Skin friction coefficient (sliding)</td>
<td>$\mu$ 0.25</td>
</tr>
<tr>
<td>Young’s Modulus (ground)</td>
<td>$E$ 1000 MPa</td>
</tr>
<tr>
<td>Poisson’s ratio (ground)</td>
<td>$\nu$ 0.25</td>
</tr>
<tr>
<td>Cohesion (ground)</td>
<td>$c$ 500 kPa</td>
</tr>
<tr>
<td>Angle of internal friction (ground)</td>
<td>$\varphi$ 25°</td>
</tr>
<tr>
<td>Dilatancy angle (ground)</td>
<td>$\psi$ 5°</td>
</tr>
</tbody>
</table>
Figure 9 shows the thrust force $F_r$ required to overcome shield skin friction as a function of the advance rate $v$ in a low-permeability ground ($k = 10^{-9}$ m/s) during TBM boring. The favourable effects of a high advance rate and of overboring can be recognised immediately. Figure 10 shows how the thrust force $F_r$ required in order to restart excavation after a standstill increases with the length of the standstill. During continuous excavation the machine has to overcome sliding friction, while directly after a TBM-stop ($t = 0$) static friction becomes relevant. A higher skin-friction coefficient $\mu$ was therefore considered.

**Figure 9.** Required thrust force $F_r$ as a function of the advance rate $v$ during continuous excavation (numerical results after Ramoni and Anagnostou, 2007a).

**Figure 10.** Required thrust force $F_r$ for restarting operations after a standstill of length $t$ (numerical results after Ramoni and Anagnostou, 2007b).
The curves of Figures 9 and 10 have been calculated by integrating the ground pressure $p$ over the shield length $L$. The ground pressure $p$ acting upon the shield and the lining at different times $t$ (i.e. the time that has elapsed since the standstill began, based on the assumption that the preceding rate of excavation was $v = 10\, \text{m/d}$) is shown in Figure 11. In this numerical example, the face was regarded as being unsupported. The core yields and extrudes freely (Figure 12) with the outcome that the radial stress ahead of the face decreases, additional load is transferred to the shield via arching in the longitudinal direction and, therefore, a pressure peak occurs close to the face (Figure 11). These results indicate that the behaviour of the core ahead of the face may also be important with respect to the loading of a TBM.

![Figure 11. Radial pressure $p$ acting upon the shield and the lining at different time points $t$ after the start of a standstill (normal overcut of 5 cm; numerical results after Ramoni and Anagnostou, 2007b).](image)

![Figure 12. Core extrusion $e$ over time $t$ during a standstill (normal overcut of 5 cm; numerical results after Ramoni and Anagnostou, 2007b).](image)
YIELDING SUPPORT IN SQUEEZING GROUND

Tunnelling through weak rocks under a high overburden can generate large deformations. This so-called "squeezing" can destroy the lining if an attempt is made to hold back the deformations by installing a rigid lining close to the face. 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 (the so-called "yielding principle", Kovári, 1998). The so-called "yielding supports" are characterized by two main design parameters: the amount of over-excavation \( u_y \) and the yield pressure \( p_y \). Figure 13 shows two examples of structural detailing.

![Figure 13. Tunnel support yielding, (a), at a low rock pressure (TH-steel sets with sliding connections) and, (b), at a high rock pressure (HIDCON\textsuperscript{®} elements, Solexperts, 2007).](image)

The conceptual idea underlying all yielding supports is that the ground pressure will decrease if the ground is allowed to deform. Although it is beyond doubt that the squeezing pressure does decrease with deformation, the relationship between pressure and deformation is not unique, but depends on the support characteristics and installation point (Section 3). The ground response curve (which describes the relationship between pressure and deformation assuming plane strain conditions and a monotonous reduction in pressure at the excavation boundary) represents the lower limit of actual pressures and deformations (Figure 2). The reason for the deviation of the actual equilibrium points from the ground response curve is the complete radial unloading of the excavation boundary over the unsupported span and the subsequent increase in radial stress following the installation of the lining. The more pronounced this stress reversal, the bigger will be the deviation from the ground response curve.

This relationship has an important practical consequence for the design of yielding supports, because the magnitude of the stress reversal depends on the yield pressure \( p_y \) of the support. More specifically, the lower the yield pressure \( p_y \), the more pronounced will be the stress reversal, the bigger will be the deviation of the final equilibrium point from the ground response curve and – the value of over-excavation \( u_y \) being fixed – the higher will be the final pressure.
Let us investigate the effect of the yield pressure quantitatively by considering the axisymmetric model of a 500 m deep, cylindrical tunnel in weak rock (Figure 14a). The mechanical behavior of the ground is modeled as isotropic, linearly elastic and perfectly plastic according to the Mohr-Coulomb yield criterion (see Table 1 for the material constants). The initial stress field is taken to be uniform and hydrostatic with a pressure $\sigma_0$ of 12.5 MPa. The yielding support is modeled as a radial support having a deformation-dependent stiffness (Figure 14b): 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-excitation $u_y$ 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 14. (a) Model; (b) characteristic line of a yielding support.](image)

Figure 15 shows the development of the radial pressure on the lining for four yielding supports that allow for a deformation $u_y$ of 15 cm but yield at different pressures $p_y$ (Table 4). For the reasons explained above, the final lining pressure decreases with the yield pressure of the support.

Figure 16 shows the equilibrium points – $u(\infty)$, $p(\infty)$ – of the ground as well as (for the purposes of comparison) the ground response curve under plane strain conditions (solid line GRC). Let us consider next how the equilibrium point changes in relation to the yield pressure $p_y$ (in cases O, A, B and C) starting with a support that can endure a radial displacement of $u_y = 15$ cm without offering any resistance to the ground (case O). Due to the long unsupported span and the pronounced stress reversal, the equilibrium point for this case is located far above the plane strain response curve (Figure 16). At higher yield pressures, the stress reversal is less pronounced and, consequently, the equilibrium point is located closer to the ground response curve. This means – since the ground deformation is governed by the over-excitation $u_y$ and is therefore approximately constant – that the final pressure decreases (points A, B and C in Figure 16).

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. A more detailed discussion of the interaction between yielding supports and squeezing ground (along with design nomograms that banish the shortcomings of plane strain analyses and enable rapid assessments of support requirements) can be found in Cantieni and Anagnostou (2009b).
Figure 15. Ground pressure $p$ vs. distance $x$ behind the face for different values of the yield pressure $p_y$ (support cases O, A, B and C) according to Table 4 (numerical results after Cantieni and Anagnostou, 2009b).

Table 4. Support parameters (see also Figure 14b).

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
<th>$k_I^{(1)}$ [MPa/m]</th>
<th>$p_y$ [kPa]</th>
<th>$u_y$ [cm]</th>
<th>$k_{III}$ [MPa/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>O</td>
<td>35 cm thick shotcrete lining with open longitudinal slots ($E_L = 30$ GPa $^{(2)}$).</td>
<td>n/a</td>
<td>0</td>
<td>15</td>
<td>656</td>
</tr>
<tr>
<td>A</td>
<td>Steel sets TH-44 (Figure 13a) spaced at 1 m with sliding connections by 4 friction loops each offering a resistance of 150 kN $^{(3)}$.</td>
<td>100</td>
<td>150</td>
<td>15</td>
<td>656</td>
</tr>
<tr>
<td>B</td>
<td>Like A, but additionally 20 cm shotcrete with highly deformable concrete elements inserted into the slots (yield stress 7 MPa, cf. Figure 13b and Solexperts, 2007).</td>
<td>100</td>
<td>425</td>
<td>15</td>
<td>656</td>
</tr>
<tr>
<td>C</td>
<td>Like B, but with higher yield strength elements (17 MPa, Solexperts, 2007).</td>
<td>100</td>
<td>850</td>
<td>15</td>
<td>656</td>
</tr>
</tbody>
</table>

Notes:

$^{(1)}$ This parameter was kept constant in the numerical study as it is of subordinate importance. The actual values of $k_I$ of the support systems described in the last columns of the table are 74 MPa/m (case A) and 114 MPa/m (cases B and C). A sensitivity analysis has shown that a variation of $k_I$ in this range does not affect the numerical results.

$^{(2)}$ This value assumes that, by the time the longitudinal slots close, the shotcrete will have developed its final stiffness.

$^{(3)}$ After the yield phase, the support is set practically rigid ($d_{III} = 35$ cm, $E_L = 30$ GPa).
7 CONCLUSIONS

Spatial considerations are important for a number of practical design and analysis issues arising in tunnel engineering. It is almost trivial – or even a tautology – to say that the prediction uncertainties of planar models can be reduced by taking account of the third dimension. It is more interesting to note that if we take adequate account of the stress history of the ground and of the sequence of excavation and support installation we may obtain results which are qualitatively different to those obtained through plane strain analyses, i.e. results which are surprising at first glance and cannot be reproduced from two-dimensional models.

8 REFERENCES


