3D modeling of a NATM tunnel in high K_0 clay using two different constitutive models

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Abstract

The paper studies the accuracy of 3D finite element predictions of a displacement field induced by NATM tunnelling in stiff clays with high K_0 conditions. The studies are applied to the Heathrow express trial tunnel. Two different constitutive models are used to represent London Clay, namely a hypoplastic model for clays and the Modified Cam clay model. Good quality laboratory data are used for parameter calibration and accurate field measurements are used to initialise K_0 and void ratio. The hypoplastic model gives better predictions than the MCC model with satisfactory estimate for the displacement magnitude and slightly overestimated width of the surface settlement trough. Parametric studies demonstrate the influence of variation of the predicted soil behavior in the very small to large strain range and the influence of the time-dependency of the shotcrete lining behavior.

CE Database subject headings

Nonlinear analysis; Tunneling; Clays; Constitutive models

Introduction

Predicting the deformation field induced by tunnelling in fine-grained soils is an important problem of geotechnical design, and as such it has been the scope of many detailed studies. It has been soon recognized that conventional finite element analysis (conventional in terms of material models used) of tunneling in stiff clays under high K_0 conditions predicts too wide surface settlement troughs when compared with field data. Under the current state-of-the-art, focusing on articles describing the tunnelling in London Clay, which is the scope of this study, the following reasons of this discrepancy appear to be the most important:

Small-strain non-linearity and high initial stiffness – The requirement to predict soil non-linearity is now being generally accepted, but only few studies give direct comparison between predictions by using ”small-strain linear” and ”small-strain non-linear” models. In this paper,

”small-strain linear” model will denote a constitutive model that is not capable of predicting high initial very-small-strain stiffness and stiffness degradation in the small-strain range. Addenbrooke et al. (1997) performed 2D FE analyzes of a tunnel in London Clay with $K_0 = 1.5$ by small-strain linear and non-linear elastic perfectly plastic models. The non-linear models, which were calibrated to fit the decay of soil stiffness measured with accurate local strain transducers, gave better results in comparison with linear models, although the predicted surface settlement trough was still shallower and wider than the measured one. It was concluded that ”unrealistic soil stiffness was required to achieve an improved prediction with $K_0 > 1$”. Similar results were reported by Gunn (1993).

The necessity to model small-strain non-linearity has been accepted by many other researchers (Dasari et al. 1996; Franzius et al. 2005; Grammatikopoulou et al. 2002; Yazdchi et al. 2006). In general, the predicted settlement trough was of a reasonable shape, although still wider than measured. In all cases K_0 value between 1 and 1.5 was assumed. Similar predictions were reported also by the small-strain linear models, but detailed investigation reveals that the predictions were obtained with soil parameters that did not represent the soil behavior at the element level (namely high and strain-independent elastic stiffness used by Tang et al. 2000; Karakus and Fowell 2005; Ng et al. 2004).

Soil anisotropy – Direct investigations into the influence of soil anisotropy were presented by Addenbrooke et al. (1997), Gunn (1993) and Franzius et al. (2005). In all cases small-strain non-linear models were used and in all cases it was concluded that incorporation of soil anisotropy (i.e., higher stiffness in horizontal than in vertical direction) improved the predictions by narrowing and deepening the settlement trough.

Neglecting 3D effects – Tunnel excavation is clearly a three-dimensional problem and therefore intuitively, considering the third dimension should lead to more accurate predictions. Available studies are, however, not conclusive in this respect. This is due to the fact that additional assumptions about the excavation sequence, lining installation procedure and time-dependent behavior of shotcrete (in the case of NATM tunnels) have to be made in 3D analyzes. Moreover, many methods to account for 3D effects in 2D modeling of NATM tunnels are available (see, e.g., Karakus 2007), which makes direct comparison of 2D and 3D results cumbersome. Dasari et al. (1996) compared 2D and 3D simulations of a NATM tunnel. They used small-strain non-linear soil model and considered both constant and time-dependent shotcrete lining stiffness. Displacement magnitude predicted by the 3D analysis was in a relatively good agreement with measurements, but the settlement trough was still wider than observed, even though unrealistically low K_0 value was used ($K_0 \simeq 1$). They concluded that the time-dependent shotcrete lining had little influence on the results, but only 2D study with either wished-in-place lining (lining installed prior to tunnel excavation) or lining installed after the full element removal was considered, which rendered these conclusions as well as any direct comparison of 2D and 3D simulations questionable. The same problem was studied by Tang et al. (2000). They used, however, small-strain linear model with unrealistically high elastic stiffness. Reasonable predictions were obtained only with a 10 m unsupported span, which was twice the span applied experimentally. Franzius et al. (2005) studied a different tunnel in London Clay, excavated using an open-faced shield, which may be simulated in 2D using so-called volume loss method (Potts and Zdravkovic 2001). In this method the volume loss is a parameter controlling the simulation and the lining properties are not taken into account. Using a small-strain non-linear model, the 2D and 3D models led to practically identical normalized settlement troughs, which were wider than the measured one. The 3D method was incapable of predicting the settlement magnitude, even when unrealistic $K_0 = 0.5$ was used. The settlement magnitude was predicted reasonably only in 2D with $K_0 = 0.5$. This is, however, not surprising, as $K_0 = 0.5$ leads to a realistic shape of the settlement trough and at the same time the known volume loss is a parameter controlling the calculation.

K_0 conditions – Franzius et al. (2005) made also a direct investigation into the influence of K_0 conditions in 3D FE analysis ($K_0 = 1.5$ and $K_0 = 0.5$). Low K_0 value (unrealistic for London Clay) led to improved predictions, namely the normalized settlement trough was narrower and deeper. In absolute values, however, low K_0 induced overprediction of vertical surface settlements by a factor of 4. With $K_0 = 1.5$ the predicted trough was too wide and vertical displacements were underpredicted by the factor of 4. Similar conclusions were drawn by Doležalová (2002)

– decreasing the K_0 value from 1.5 to 0.5 closes up the settlement trough and increases vertical settlements in absolute terms.

This review of analyzes of tunneling in stiff clay shows that regardless of the significant advances made, it is still difficult to make reliable predictions of deformation field due to tunnelling. Lowering K_0 improved the shape of the predicted settlement trough. All the above analyzes were performed with $K_0 \leq 1.5$. The measured value in London Clay is, however, considerably higher, especially near the surface (see data by Hight et al. 2007 in Fig. 3b). Moreover, it appears that the authors were able to fit either the shape of the settlement trough (by considering unrealistic K_0 and degree of anisotropy, as, e.g., Franzius et al. 2005) or the displacement magnitude (with a too wide settlement trough predicted and, in the case of Tang et al. 2000, unrealistic excavation sequence). None of the works presented satisfactory predictions unless unrealistic assumptions for initial conditions, boundary conditions or constitutive parameters were made.

This paper aims at investigating to what extent it is possible to obtain correct predictions using advanced material models with: (1) Initial conditions (namely K_0 and e) set up according to accurate field measurements; (2) Model parameters (for both soil and shotcrete) calibrated solely on the basis of high quality laboratory experimental data and (3) realistic modeling of the excavation sequence in 3D FE analysis. The Heathrow Express trial tunnel (Deane and Basset 1995), a NATM tunnel excavated in London Clay, has been chosen for the purpose of this study as detailed monitoring data are available and as London Clay properties have been studied thoroughly in the past.

For calibration of the models, the results from detailed experimental and field studies dealing with the Heathrow Terminal 5 site have been adopted (Hight et al. 2007; Gasparre et al. 2007a; Gasparre et al. 2007b; Gasparre 2005). The Terminal 5 site is located approximately 1.5 km NW from the trial tunnel. The variability of London Clay across the London Clay basin has been studied in detail by (Burnett and Fookes 1974). They noted that plasticity of London Clay increases from west to east across the basin due to increasing depth of sedimentation. In the small scale, however, the London Clay is very homogeneous. The data from Terminal 5 are thus believed to represent well soil properties at the trial tunnel site.

Details of numerical analyzes

The finite element analyzes were performed using the finite element program *Tochnog Professional* (Rodemann 2008). Different constitutive models have been implemented by means of a user-defined material model subroutine *umat*, which has originally been developed for the *ABAQUS* [trademark Abaqus, Inc., USA. www.simulia.com] finite element program. An explicit adaptive stress point algorithm with local substepping based on the Runge-Kutta-Fehlberg method of the second and third order of accuracy (RKF-23) (Enright et al. 1995; Hull et al. 1996) and numerical estimation of the consistent stiffness matrix is used for the time integration of the constitutive models. The *umat* subroutine for constitutive models used in this paper is freely available on the web (Gudehus et al. 2008).

Geometry

Problem geometry corresponds to the Heathrow Express trial tunnel (Deane and Basset 1995), namely to the "Type 2" excavation sequence (vertical sequence, left drift excavated first, followed by the right drift). The excavation sequence implied the problem to be non-symmetric, so the full geometry had to be modelled in the FE analysis. The finite element mesh, including dimensions, is shown in Fig. 1. Vertical mesh boundaries are located approximately 50 m (6 diameters) from the tunnel. To demonstrate that their position does not influence the calculated results, additional analyzes with extended mesh (boundaries 110 m, i.e. 13 diameters, from the tunnel) have been performed. The mesh consists of 7636 eight-noded brick elements, which were used to model both soil and tunnel primary lining. The use of continuum elements rather than shell elements to model tunnel lining in 3D FE analysis is supported by a comparative study by Ausgrade and Burd (2001). No interface elements have been used between the tunnel lining and the soil. Therefore sliding of the lining with respect to soil has not been allowed for, which is a reasonable assumption for shotcrete lining. On the vertical sides of the mesh normal horizontal movements have been restrained, whereas the base has been fixed in all directions.

The tunnel (including primary lining) is 9.2 m wide and 7.9 m high, its crown is located 16 m below the ground level. The tunnel is 30 m long in the longitudinal direction. The primary lining is 25 cm thick at tunnel boundaries, the middle temporary lining separating the left and right drifts is 15 cm thick. The top five meters represent gravel sediments of the Thames River and backfill, modelled as a single material. The 35 m below represent the London Clay strata.

Modelling the excavation sequence

The particular sequence of the ”Type2” excavation of the Heathrow Express trial tunnel consisted of the excavation of the left drift at the whole length (30 m), followed by the excavation of the right drift and simultaneous removal of the middle lining. Sketch of the longitudinal and transversal sections through the excavation sequence as performed in the field is shown in Fig. 2 (Karakus and Fowell 2005, Bowers 1997).

Fig. 2 also shows the approximation of the excavation sequence used in FE analyzes. The tunnel excavation has been simulated by step-by-step removal of elements inside the tunnel. The left drift was excavated first, followed by the right drift. Excavation of the tunnel crown was, as in the experiment, always two meters in advance of the bench. Unlike in the field trial, however, the corresponding crown and bench elements were excavated simultaneously. After the full element removal the adjacent lining elements were activated. This procedure led to 4 m unsupported span between the crown face and bench lining after the element removal, which represented quite well the experimental procedure (the real unsupported span was 5 meters at maximum). The middle lining was removed during the right drift excavation simultaneously with the adjacent soil elements. The temporary backfill that was needed in the experiment to protect bench lining against tunnel machinery was not modelled.

Initial conditions

Initially it was necessary to specify the effective vertical stress, effective horizontal stress (calculated through the K_0 value) and void ratio e. In the London Clay, the initial void ratio e was calculated from the water content profile given by Hight et al. (2007), particularly for London Clay sub-unit B2 (i.e., sub-unit in which the tunnel was located). Available measurements are shown in Fig. 3a. Although there is some scatter in the experimental data, no clear trend in the change of the water content with depth can be observed. An approximate average value $w_c = 25.5$ %, was adopted, which led to $e = 0.7$ (calculated through the specific gravity $G_s = 2.75$, measured for London Clay by Gasparre 2005).

The assumed water content led to the total unit weight of the saturated clay $\gamma = 19.6$ kN/m^3. Vertical stresses were calculated by assuming full saturation and ground water table level 2 m below the ground level. Hight et al. (2007) provided K_0 profile measured by suction probes (Ridley and Burland 1993). The measured K_0 profile is given in Fig 3b. Clearly, the K_0 value varies significantly with depth and it reaches relatively high values, especially near the top of the London Clay strata ($K_0 \simeq 3$), but even at the tunnel depth $K_0 \geq 1.5$.

Finally, the Thames gravel (top 5 m of the soil) was characterized by $\gamma = 19.6$ kN/m^3 and constant K_0 value that corresponds to normally consolidated conditions ($K_0 = 0.43$).

Drainage conditions

The analyzes were performed as undrained, only the top five meters of the soil (Thames gravel and made ground) were modelled as drained. For undrained analyzes a penalty method, that is adding a water bulk modulus K_w term to the material stiffness matrix (Potts and Zdravkovic 1999, e.g.), was used. In reality the process of the tunnel excavation is not fully undrained. Volumetric deformations occurring due to excess pore pressure dissipation were allowed for by using a reduced value of K_w. A suitable value of K_w was found by comparing results of preliminary 2D coupled consolidation analyzes with a realistic anisotropic permeability profile (Hight et al. 2007, Kovacevic et al. 2007) and undrained analyzes. The chosen value $K_w = 10^5$ kPa led to practically identical shapes of the normalized surface settlement troughs, while the undrained analysis predicted approximately 25% lower vertical surface settlements than the corresponding coupled consolidation analysis.

Constitutive models

In this paper, constitutive modeling focused on detailed description of the London Clay behavior. Particular attention was paid to the calibration of the models, which was always done solely on the basis of laboratory experimental data, rather than by back-analysis of field measuremens. For simplicity in all analyzes the Thames gravel was modelled using elasto-plastic Mohr-Coulomb model, with $E = 75$ MPa, $\nu = 0.25$, $\varphi = 35°$, $c = 0$ kPa and $\psi = 17.5°$.

The predictions of an advanced constitutive model for soils, detailed in the next section, were compared with predictions by the Modified Cam clay model (MCC). Both models were calibrated using the same laboratory experiments. The parameters of the MCC model are given in Table 1.

Hypoplastic model for clays

The model was proposed by Mašín (2005) and investigated further by Mašín and Herle (2005). It is based on hypoplasticity, an alternative approach to constitutive modeling of geomaterials, tn which the non-linear soil behavior is modelled using a rate equation non-linear in $\dot{\boldsymbol{\epsilon}}$.

The basic model requires five parameters (N, λ^*, κ^*, φ_c and r). The parameters have the same physical interpretation as the parameters of the MCC model (see Gudehus and Mašín 2008). N and λ^* define the position and the slope of the isotropic normal compression line in the $\ln(1+e)$ vs. $\ln p$ plane (Butterfield 1979)

$$\ln(1+e) = N - \lambda^* \ln(p/p_r) \tag{1}$$

where $p_r = 1$ kPa is a reference stress; parameter κ^* defines slope of the isotropic unloading line in the same plane. φ_c is the critical state friction angle and the parameter r controlls the large-strain shear modulus.

The basic hypoplastic model is capable of predicting the behavior of fine-grained soils upon monotonic loading at medium to large strain levels. In order to improve the model performance in the small-strain range, its mathematical formulation is enhanced by the so-called intergranular strain concept (Niemunis and Herle 1997). In this concept the total strain can be thought of as the sum of a component related to the deformation of interface layers at intergranular contacts, quantified by the intergranular strain tensor $\boldsymbol{\delta}$, and a component related to rearrangement of soil skeleton. The enhancement and its physical interpretation has originally been developed for granular materials. The predictions however reproduce well also the observed behavior of fine-grained soils, namely high initial stiffness (e.g., Jardine et al. 1984, Viggiani and Atkinson 1995) and the effects of recent history (Atkinson et al. 1990, Stallebrass and Taylor 1997). The enhancement is therefore considered suitable in combination with the hypoplastic model for clays. The intergranular strain concept requires four additional parameters: R controlling the size of the elastic range, β_r and χ controlling the rate of stiffness degradation and m_R controlling the initial shear stiffness G_0. It may be calculated from

$$G_0 \simeq \frac{m_R}{r\lambda^*} p \tag{2}$$

where r and λ^* are parameters of the basic hypoplastic model, p is mean stress and G_0 is the initial very-small-strain shear modulus.

As investigated in detail by Cotecchia and Chandler (2000) and Baudet and Stallebrass (2004), the basic parameters characterizing the soil behavior (in the case of the hypoplastic model parameters λ^*, κ^*, φ_c and r) may be found on the basis of experiments on reconstituted clay. The approach, based on the so-called sensitivity framework, was followed also in this paper. The structure of natural clay was incorporated by calibration of the parameters controlling the small-strain stiffness using high-quality experiments on *natural* London clay (see below), and by increasing the size of the state boundary surface using equation

$$N_{nat} = N_{rec} + \lambda^* \ln(s) \tag{3}$$

where N_{rec} is the value of the parameter N calibrated using data on reconstituted clay, N_{nat} is its value used for predicting the behavior of natural clay and s is the measure of soil structure. Eq. (3) follows from the sensitivity framework applied to natural stiff clays with stable structure. In this work, the value $s = 2.5$, estimated for natural London Clay from sub-unit B2 by Gasparre (2005), is adopted.

The calibration of parameters N_{rec}, λ^* and κ^* using isotropic loading and unloading tests on reconstituted London Clay specimens by Gasparre (2005) is demonstrated in Fig. 4a. The critical state friction angle $\varphi_c = 21.9°$ was estimated by Gasparre (2005). The parameter r which controls the shear modulus was found by back-analysis of an undrained shear test on reconstituted overconsolidated soil, see Fig. 4b. The figure shows that the chosen value $r = 0.5$ leads to a slight under-estimation of the initial stiffness. The fit is, however, improved by the use of intergranular strain concept (see also Fig. 4b).

The parameters of the intergranular strain concept were selected based on experiments on natural London Clay from sub-unit B2(a) (Gasparre 2005, Gasparre et al. 2007b). Parameter m_R, controlling the initial stiffness, was estimated using Eq. (3). The bender element measurements by Gasparre (2005) revealed $G_{0hv} \simeq 80$ MPa at $p = 420$ kPa, and therefore $m_R \simeq 9$. The parameters R, β_r and χ were found by fitting the stiffness degradation curve of three undrained tests that followed the stress history of the London Clay, see Fig. 5. The parameters of the hypoplastic model are summarised in Table 2.

The simulations with the hypoplastic model required the specification of the initial values for the new state variable intergranular strain ($\boldsymbol{\delta}$). As generally accepted (e.g., Clayton and Heymann 2001; Niemunis and Herle 1997), it was assumed that the long creep period during the geological history erased any effects of recent history, so soil state is inside the very-small-strain elastic range and high initial stiffness is obtained in any loading direction. This assumption is supported by observations from Gasparre et al. (2007b). In the context of the hypoplastic model it implies the initial value $\boldsymbol{\delta} = \mathbf{0}$.

Model for shotcrete lining

The time-dependent stress-strain behavior of shotcrete lining is rather complex and the chosen constitutive model and its parameters have a significant influence on the predicted deformations due to NATM tunnelling. The complexity of the constitutive assumption for shotcrete varies significantly throughout literature. From linear elasticity (Karakus and Fowell 2005; Möller et al. 2003; Ng et al. 2004; Dasari et al. 1996; Farias et al. 2001; Tang et al. 2000), followed by linear elasticity with time-dependent Young modulus (Yazdchi et al. 2006; Powell et al. 1997; Dasari et al. 1996), to complex constitutive models based on viscoplasticity (Meschke et al. 1996), time-dependent non-linear elasticity (Histake 2003) and chemomechanical models (Lackner et al. 2002; Boldini et al. 2005).

In this work, linear elastic perfectly plastic von Mises model with time dependent and strain independent Young modulus and time dependent strength was considered. Non-linear stress-strain behavior, creep and relaxation were not taken into account. Preliminary analyzes have shown that considering plasticity limit did not influence significantly the calculated results (the same conclusions were given by Yazdchi et al. 2006) and therefore only the results with time dependent linear elasticity are presented.

An empirical exponential dependency of shotcrete Young modulus E on time (Pöttler 1990; Oreste 2003) was adopted:

$$E = E_f \left(1 - e^{-\alpha t/t_r}\right) \tag{4}$$

where E_f is the final Young modulus, α is a parameter and $t_r = 1$ day is the reference time. Eq. (4) was calibrated using experimental data by Bae et al. (2004), who tested a shotcrete with alkali-free accelerator. The data and calibration curves are shown Fig. 6. The chosen values are $E_f = 14.5$ GPa and $\alpha = 0.14$. It must be noted that no information on the shotcrete type used in the trial tunnel is available to the author, and therefore parameters adopted must be considered as approximate, as the shotcrete composition influences remarkably its time-dependent behavior (John and Mattle 2003). A parametric study of the influence of the parameter α is presented later in the text. Shotcrete Poisson ratio was assumed to have a constant value of $\nu = 0.1$.

Using the time-dependent constitutive model required the model to reproduce the real time needed for the excavation, which was 24 days for the whole trial tunnel, 12 days for each drift (Deane and Basset 1995).

Results of analyzes

Three sets of analyzes were performed. The influence of the soil constitutive model, the influence of the increase of shotcrete lining stiffness with time, and finally the influence of stiffness in the small-strain range were addressed.

The influence of soil constitutive model

The influence of soil constitutive model is shown in Fig. 7. When possible, predictions are compared with monitoring data from Deane and Basset (1995) and Bowers (1997). Figs. 7a and 7b show the surface settlement troughs in transverse and longitudinal directions, respectively. Fig. 7c depicts horizontal displacements in the horizontal distance of 7.7 m from the tunnel centerline. Finally, Fig. 7d shows time development of surface settlements above the tunnel centerline. Simulations by the hypoplastic model using extended mesh are also included in Fig. 7. The used and extended meshes lead to practically identical results.

The following are the most important observations when analyzing the simulation data:

- The MCC model, which is incapable of predicting the behavior in the small-strain range gives unrealistic predictions with surface heave above the tunnel centerline and the largest downward vertical displacement in a certain horizontal distance from the centerline (Figs. 7a,b,d). This model also significantly overpredicts horizontal displacements at the tunnel level (Fig. 7c). The hypoplastic model gives more realistic predictions with the U-shape surface settlement trough.

- Fig. 7d shows that the surface heave predicted by the MCC model occurs exclusively during excavation of the left drift (the first 12 days in Fig. 7d). The deformation mechanism is driven by high K_0 conditions, with large horizontal stresses and consequently large horizontal inwards displacements of the lining. The initial stiffness predicted by the MCC model is too low and the lining acts in this case as a stiff ring, and pushes the soil upwards at the top corner. The hypoplastic model predicts a high enough initial stiffness to suppress this effect. The overall displacement field at the end of the tunnel excavation is shown in Fig. 8. The figure demonstrates the upward heave of the soil wedge above the tunnel predicted by the MCC model, and the more realistic predictions by the hypoplastic model.

- The hypoplastic model gives more accurate predictions, both in the qualitative and quantitative way. However, the predicted surface settlement trough is wider then the observed one in both transversal and longitudinal directions and horizontal displacements in the tunnel level are slightly overestimated. This is a common shortcoming of the predictions of deformations induced by tunneling in high K_0 environment. It could be further improved by incorporating anisotropic initial stiffness (see Introduction).

The influence of time-dependent behavior of shotcrete lining

Four additional analyzes were performed to study the influence of the increase of shotcrete Young modulus with time. The hypoplastic model was used, five different values were assigned to the parameter α in Eq. (4), namely 0.14 (original value); 0.25; 0.5; 1; $\rightarrow \infty$ (i.e., constant Young modulus $E = 14.5$ GPa). The resulting E vs. time relationship is shown in Fig. 6.

Simulation results are shown in Fig. 9. As expected, increasing the value of the parameter α (i.e., stiffening the lining) decreases the displacement magnitude. The influence is significant. It is interesting to point out that for $\alpha = 1$ and $\alpha =\rightarrow \infty$ an upward vertical displacement above the tunnel centerline is observed (Fig. 9b), similarly to predictions by the MCC model (although to a lower extent). In this case the lining is too stiff with respect to the soil stiffness and the mechanism discussed in the previous section is applicable.

We may conclude that the shotcrete behavior influences significantly the calculated results and the shotcrete time-dependent behavior needs to be considered in 3D simulations of NATM tunneling. However, a more detailed study on the influence of other characteristics, such as non-linear stress-strain behavior, creep and relaxation, should be performed.

Detailed investigation of the influence of soil stiffness characteristics in the small-strain range

To investigate the role of stiffness in detail, small-strain stiffness parameters of the hypoplastic model were varied in such a way that the predicted stiffness vs. strain curves follow upper and lower bounds of experimentally measured behavior.

Three sets of analyzes were performed: First, the initial very-small-strain shear stiffness was varied by changing the parameter m_R (and m_T), while keeping the rate of stiffness degradation and large-strain stiffness (Fig. 5a). Second, the large-strain stiffness was varied by changing the parameter r, while keeping the initial very-small-strain stiffness and rate of stiffness degradation. Although this influence appears to be negligible when studying the predictions in the small-strain range (Fig. 5b), it becomes important when looking at the overall large-strain stress-strain behavior (Fig. 4b). Third, the initial very-small-strain stiffness and the large-strain stiffnesses were kept unchanged, but the rate of stiffness degradation was varied (by changing the parameter β_r, see Fig. 5c). Six analyzes were performed, the used parameters are summarized in Table 3.

The surface settlement troughs in transverse direction are presented in Fig. 10. All three studied aspects affect the results. The following observations, applicable to predictions of sprayed concrete tunnels in fine-grained soils with high K_0, appear to be the most important:

- Perhaps surprisingly, varying the large-strain stiffness while keeping the stiffness degradation curve in the small-strain range has a pronounced effect on the magnitude of predicted settlements. Less important is the effect on the trough shape, see Fig. 10b. This suggests that calibrating the small-strain non-linear models on the basis of the stiffness degradation curve in logarithmic scale only (as done, e.g., by Addenbrooke et al. 1997; Dasari et al. 1996; Franzius et al. 2005) is in this case insufficient. Large-strain behavior (Fig. 4b) should be taken into account (cf. the three curves in Fig. 5b may be at the first insight considered as equally suitable).

- Increasing the very-small-strain stiffness while keeping the rate of stiffness degradation and large-strain stiffness (simulation for $m_R = 13.5$ in Fig. 5a) improves predictions by narrowing and deepening the settlement trough. This agrees with the observations on the influence of anisotropy discussed in the Introduction. The used hypoplastic model does not allow one to specify the degree of inherent small-strain stiffness anisotropy. As the ratio of horizontal to vertical stiffnesses measured by Gasparre (2005) was high ($G_{0hh}/G_{0hv} \simeq 2$) and the hypoplastic model was calibrated to fit the measurements of G_{0hv}, it underestimates the horizontal stiffness. This indicates an important direction for future development of the hypoplastic model.

- Varying the rate of stiffness degradation, while keeping the very-small-strain and large-strain stiffness influences the results considerably (Fig. 10b). The need for accurate local strain measurements is thus demonstrated. Calibrating the initial stiffness using dynamic methods (such as bender elements) and large-strain stiffness using conventional external axial strain measurements only is insufficient.

Final remarks

The displacement field induced by NATM tunnelling in high K_0 environment can be predicted with reasonable accuracy without unrealistic assumptions about the initial conditions, boundary conditions, or the constitutive model parameters. The analyzes with the hypoplastic model gave a satisfactory estimate of the settlement magnitude and slightly overestimated the settlement trough width. The results are applicable to NATM tunnels in fine-grained soils with high K_0 conditions. Detailed conclusions have been outlined in individual sections describing the results of the analyzes.

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References

Addenbrooke, T., Potts, D., and Puzrin, A. 1997. The influence of pre-failure soil stiffness on the numerical analysis of tunnel construction. Géotechnique, **47**(3): 693–712.

Atkinson, J. H., Richardson, D., and Stallebrass, S. E. 1990. Effects of recent stress history on the stiffness of over–consolidated soil. Géotechnique, **40**(4): 531–540.

Ausgrade, C. E. and Burd, H. J. 2001. Three-dimensional finite element analysis of lined tunnels. International Journal for Numerical and Analytical Methods in Geomechanics, **25**: 243–262.

Bae, G.-J., Chang, S.-H., Lee, S.-W., and Park, H.-G. 2004. Evaluation of interfacial properties between rock mass and shotcrete. International Journal of Rock Mechanics and Mining Sciences, Proc. of the ISRM SINOROCK 2004 symposium, Hudson, J. A. and Xia-Ting, F. (Eds.), **41, Supplement 1**: 106–112.

Baudet, B. A. and Stallebrass, S. E. 2004. A constitutive model for structured clays. Géotechnique, **54**(4): 269–278.

Boldini, D., Lackner, R., and Mang, H. A. 2005. Ground-shotcrete interaction of NATM tunnels with high overburden. Journal of Geotechnical and Geoenvironmental Engineering, **131**(7): 886–897.

Bowers, K. H. 1997. An appraisal of the New Austrian Tunnelling Method in soil and weak rock. Ph. D. thesis, The University of Leeds.

Burnett, A. D. and Fookes, P. G. 1974. A regional engineering geological study of the London Clay in the London and Hampshire basins. Quarterly Journal of Engineering Geology, **7**(3): 257–295.

Butterfield, R. 1979. A natural compression law for soils. Géotechnique, **29**(4): 469–480.

Clayton, C. R. I. and Heymann, G. 2001. Stiffness of geomaterials at very small strains. Géotechnique, **51**(3): 245–255.

Cotecchia, F. and Chandler, J. 2000. A general framework for the mechanical behaviour of clays. Géotechnique, **50**(4): 431–447.

Dasari, G. R., Rawlings, C. G., and Bolton, M. D. 1996. Numerical modelling of a NATM tunnel construction in London Clay. In Mair and Taylor (Eds.), Geotechnical Aspects of Underground Construction in Soft Ground, pp. 491–496. A.A.Balkema, Rotterdam.

Deane, A. P. and Basset, R. H. 1995. The Heathrow express trial tunnel. Proc. Instn. Civil Engineers, **113**: 144–156.

Doležalová, M. 2002. Approaches to numerical modelling of ground movements due to shallow tunnelling. In Proc. 2^{nd} Int. Conference on Soil Structure Interaction in Urban Civil Engineering, ETH Zürich.

Enright, W. H., Higham, D. J., Owren, B., and Sharp, P. W. 1995. A survey of the explicit Runge-Kutta method. Technical Report 94-291, University of Toronto, Department of Computer Science.

Farias, M. M., Assis, A., P., and Moraes A., H., V. J. 2001. Three-dimensional numerical simulation of a tunnel excavated using NATM - Brasilia underground case. In Proc. XV Int. Conf. Soil. Mech. and Geot. Eng., Istanbul, Turkey, Volume 2, pp. 1391–1394. A. A. Balkema.

Franzius, J. N., Potts, D. M., and Burland, J. B. 2005. The influence of soil anisotropy and K_0 on ground surface movements resulting from tunnel excavation. Géotechnique, **55**(3): 189–199.

Gasparre, A. 2005. Advanced laboratory investigation of London clay. Ph. D. thesis, University of London, Imperial College of Science, Technology and Medicine.

Gasparre, A., Nishimura, S., Coop, M. R., and Jardine, R. J. 2007. The influence of structure on the behaviour of London Clay. Géotechnique, **57**(1): 19–31.

Gasparre, A., Nishimura, S., Minh, N. A., Coop, M. R., and Jardine, R. J. 2007. The stiffness of natural London Clay. Géotechnique, **57**(1): 33–47.

Grammatikopoulou, A., Zdravković, L., and Potts, D. M. 2002. The behaviour of 'bubble' models in tunneling problems. In Proc. 2^{nd} Int. Conference on Soil Structure Interaction in Urban Civil Engineering, Zürich.

Gudehus, G., Amorosi, A., Gens, A., Herle, I., Kolymbas, D., Mašín, D., Muir Wood, D., Nova, R., Niemunis, A., Pastor, M., Tamagnini, C., and Viggiani, G. 2008. The soilmodels.info project. International Journal for Numerical and Analytical Methods in Geomechanics, **32**(12): 1571–1572.

Gudehus, G. and Mašín, D. 2008. Graphical representation of constitutive equations. Géotechnique (in print),.

Gunn, M. J. 1993. The prediction of surface settlement profiles due to tunnelling. In Predictive soil mechanics: Proceedings of the Worth Memorial Symposium, London, pp. 304–316. Thomas Telford, London.

Hight, D. W., Gasparre, A., Nishimura, S., Minh, N. A., Jardine, R. J., and Coop, M. R. 2007. Characteristics of the london clay from the Terminal 5 site at Heathrow airport. Géotechnique, **57**(1): 3–18.

Histake, M. 2003. Effects of steel fiber reinforced high-strength shotcrete in a squeezing tunnel. Tunnelling and Underground Space Technology, **18**: 197–204.

Hull, T. E., Enright, W. H., and Jackson, K. R. 1996. Runge-Kutta research at Toronto. Applied Numerical Mathematics: Transactions of IMACS, **22**(1–3): 225–236.

Jardine, R. J., Symes, M. J., and Burland, J. B. 1984. The measurement of soil stiffness in the triaxial apparatus. Géotechnique, **34**(3): 323–340.

John, M. and Mattle, B. 2003. Factors of shotcrete lining design. Tunnels and Tunnelling International, **35**(10): 42–44.

Karakus, M. 2007. Appraising the methods accounting for 3D tunnelling effects in 2D plane strain FE analysis. Tunnelling and Underground Space Technology, **22**: 47–56.

Karakus, M. and Fowell, R. J. 2005. Back analysis for tunnelling induced ground movements and stress redistribution. Tunnelling and Underground Space Technology, **20**: 514–524.

Kovacevic, N., Hight, D. W., and Potts, D. M. 2007. Predicting the stand-up time of temporary london clay slopes at Terminal 5, Heathrow airport. Géotechnique, **57**(1): 63–74.

Lackner, R., Macht, J., Hellmich, C., and Mang, H. A. 2002. Hybrid method for analysis of segmented shotcrete tunnel linings. Journal of Geotechnical and Geoenvironmental Engineering, **128**(4): 298–308.

Mašín, D. 2005. A hypoplastic constitutive model for clays. International Journal for Numerical and Analytical Methods in Geomechanics, **29**(4): 311–336.

Mašín, D. and Herle, I. 2005. State boundary surface of a hypoplastic model for clays. Computers and Geotechnics, **32**(6): 400–410.

Meschke, G., Kropik, C., and Mang, H. A. 1996. Numerical analyses of tunnel linings by means of a viscoplastic material model for shotcrete. International Journal for Numerical Methods in Engineering, **39**: 3145–3162.

Möller, S. C., Vermeer, P. A., and Bonnier, P. G. 2003. A fast 3D tunnel analysis. In K. J. Bathe (Ed.), 2^{nd} MIT Conference on Computational Fluid and Solid Mechanics, Cambridge MA, Volume 1, pp. 486–489. Elsevier, Amsterdam.

Ng, C. W. W., Lee, K. M., and Tange, D. K. W. 2004. Three-dimensional numerical investigations of new Austrian tunnelling method (NATM) twin tunnel interactions. Canadian Geotechnical Journal, **41**: 523–539.

Niemunis, A. and Herle, I. 1997. Hypoplastic model for cohesionless soils with elastic strain range. Mechanics of Cohesive-Frictional Materials, **2**: 279–299.

Oreste, P. P. 2003. A procedure for determining the reaction curve of shotcrete lining considering transient conditions. Rock Mechanics and Rock Engineering, **36**(3): 209–236.

Pöttler, R. 1990. Time-dependent rock-shotcrete interaction. A numerical shortcut. Computers and Geotechnics, **9**: 149–169.

Potts, D. M. and Zdravkovic, L. 1999. Finite element analysis in geotechnical engineering. Volume I: Theory. Thomas Telford, London.

Potts, D. M. and Zdravkovic, L. 2001. Finite element analysis in geotechnical engineering. Volume II: Application. Thomas Telford, London.

Powell, D. B., Sigl, O., and Beveridge, J. P. 1997. Heathrow Express – design and performance of the platform tunnels at Terminal 4. In Tunneling '97, pp. 565–593. Institution of Mining and Metalurgy, London.

Ridley, A. M. and Burland, J. B. 1993. A new instrument for measuring soil moisture suction. Géotechnique, **43**(2): 321–324.

Rodemann, D. 2008. Tochnog Professional user's manual. http://www.feat.nl.

Stallebrass, S. E. and Taylor, R. N. 1997. Prediction of ground movements in overconsolidated clay. Géotechnique, **47**(2): 235–253.

Tang, D. K. W., Lee, K. M., and Ng, C. W. W. 2000. Stress paths around 3-D numerically simulated NATM tunnel in stiff clay. In Fujita and Miyazaki (Eds.), Geotechnical Aspects of Underground Construction in Soft Ground, pp. 443–449. A.A.Balkema, Rotterdam.

Viggiani, G. and Atkinson, J. H. 1995. Stiffness of fine–grained soil at very small strains. Géotechnique, **45**(2): 245–265.

Yazdchi, M., Macklin, S. R., and Yeow, H.-C. 2006. 3D modelling of sprayed-concrete-lined tunnels in clay. Geotechnical Engineering, Proc. Inst. Civil. Eng., London, **159**(GE4): 243–250.

List of Tables

Table 1: *Parameters of the Modified Cam clay model.*

M	λ^*	κ^*	N_{rec}	G	N_{nat}
0.85	0.095	0.026	1.257	3000 kPa	1.344

Table 2: *Parameters of the hypoplastic model.*

	φ_c	λ^*	κ^*	N_{rec}	r	N_{nat}	m_R	R	β_r	χ
this study	21.9°	0.095	0.015	1.257	0.5	1.344	9	5×10^{-5}	0.1	1

Table 3: *Parameters of the enhanced hypoplastic model for the parametric study on the stiffness vs. strain characteristics. Parameters not given here take their values from Table 2.*

analysis label	r	m_R	β_r
orig. param.	0.5	9	0.1
$m_R = 6$	0.5	6	0.1
$m_R = 13.5$	0.5	13.5	0.1
$r = 0.33$	0.33	6.1	0.1
$r = 0.75$	0.75	13	0.1
$\beta_r = 0.033$	0.5	9	0.033
$\beta_r = 0.3$	0.5	9	0.3

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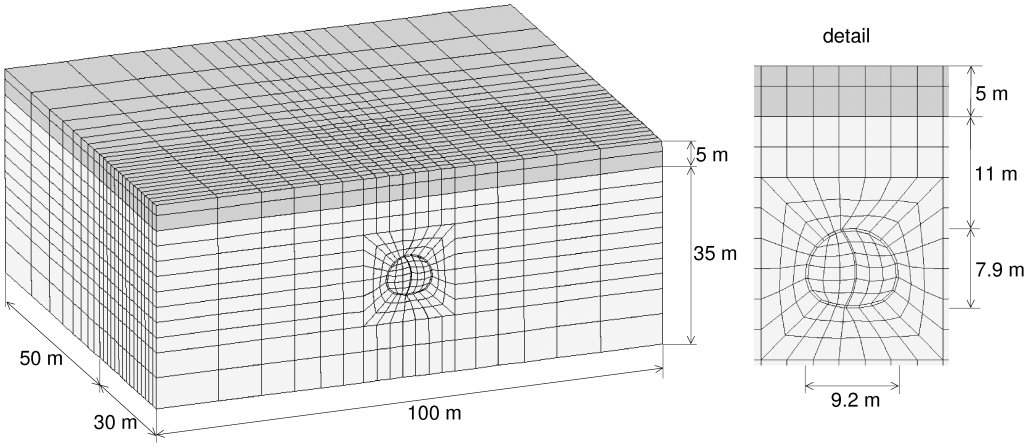


Figure 1: Finite element mesh used in the analyses.

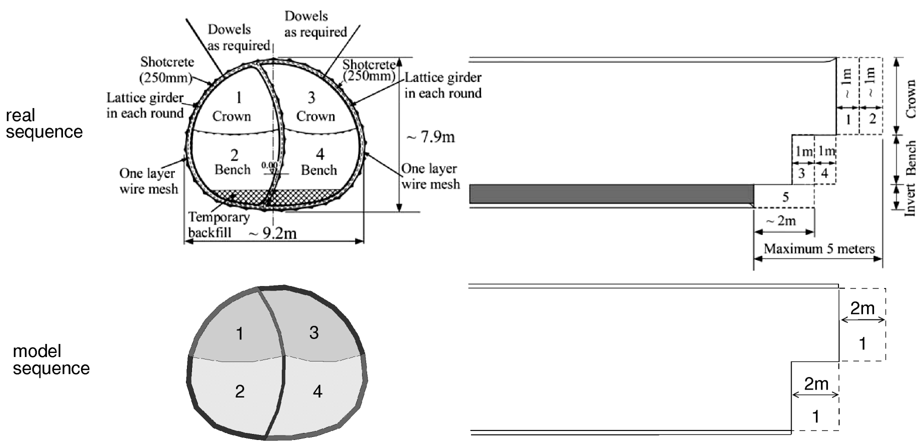


Figure 2: ”Type 2” excavation sequence as performed in the trial tunnel experiment (top) and as approximated in FE simulation (bottom) (”real sequence” from Karakus and Fowell 2005; Bowers 1997).

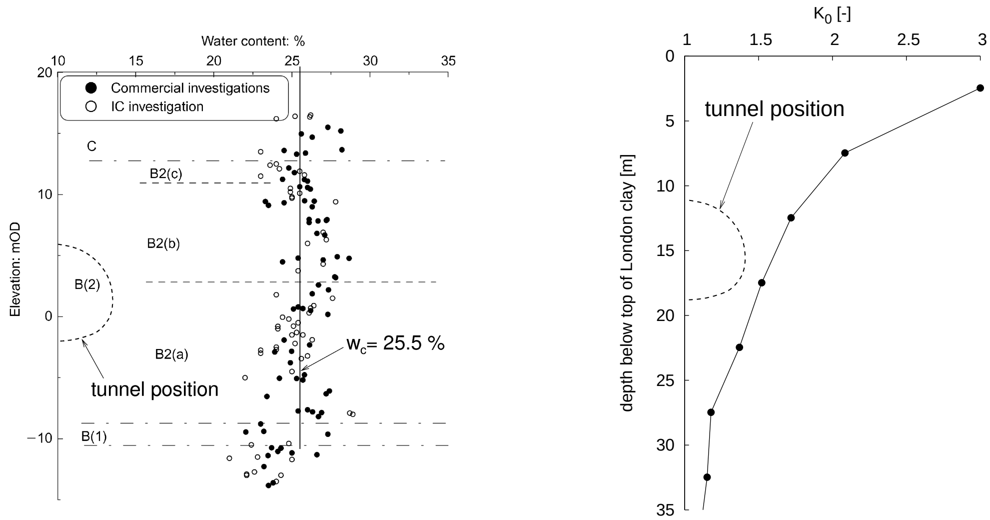


Figure 3: (a) Water content profile through the London Clay (from Hight et al. 2007, modified), (b) K_0 profile used in all analyses (data from Hight et al. 2007).

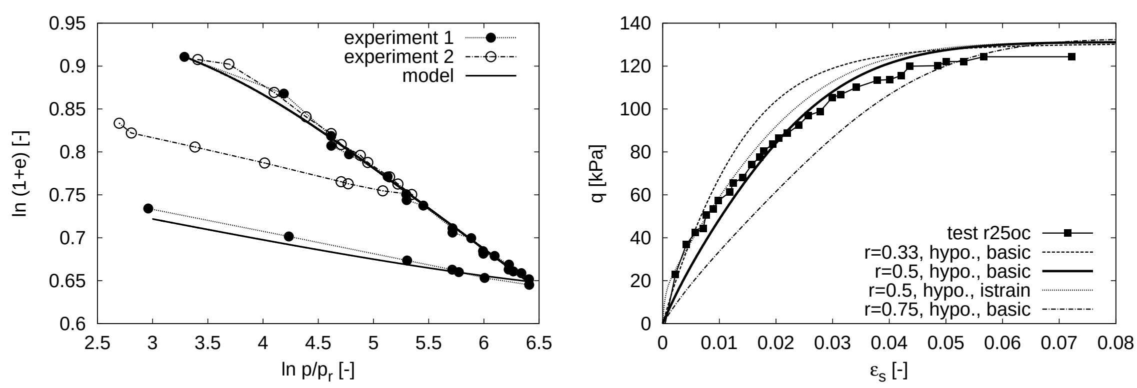
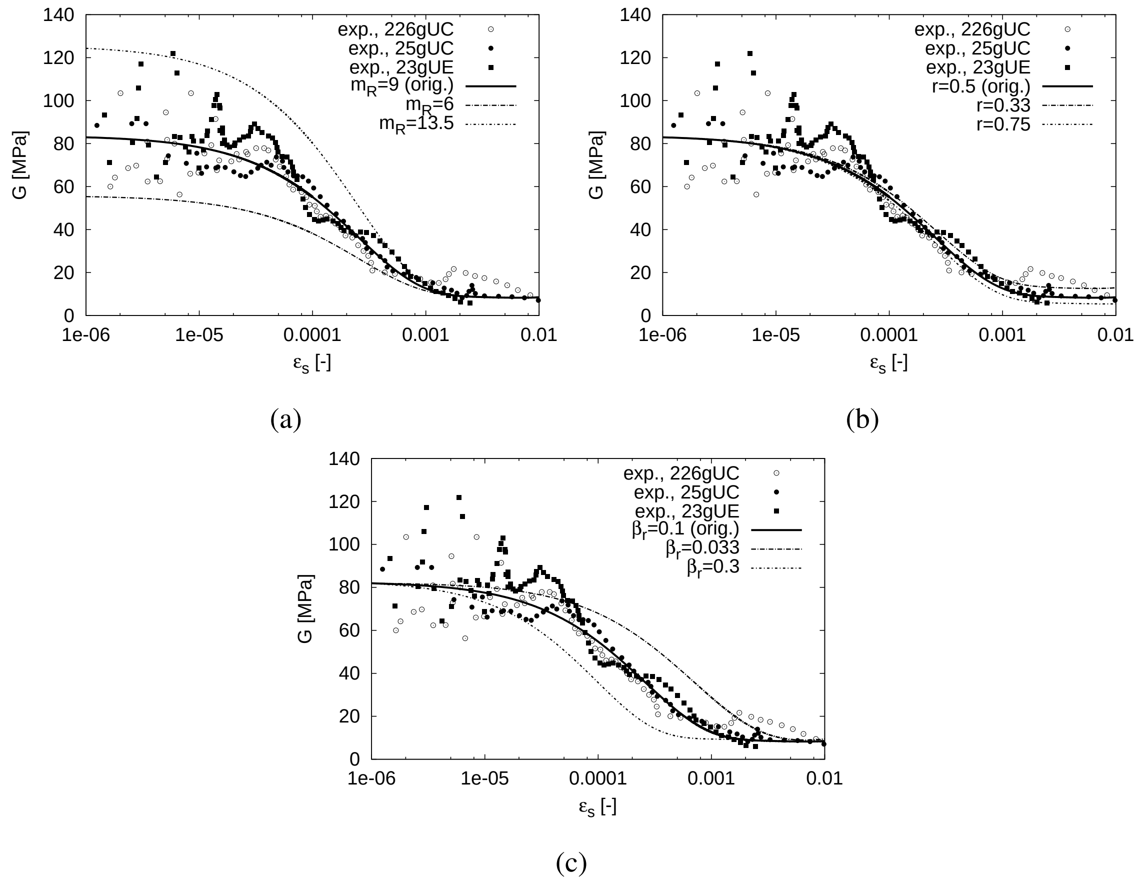


Figure 4: (a) Calibration of parameters N, λ^* and κ^* using isotropic tests on reconstituted samples from sub-unit B2, (b) calibration of the parameter r using undrained shear test on reconstituted overconsolidated sample from sub-unit B2 (exp. data from Gasparre 2005).



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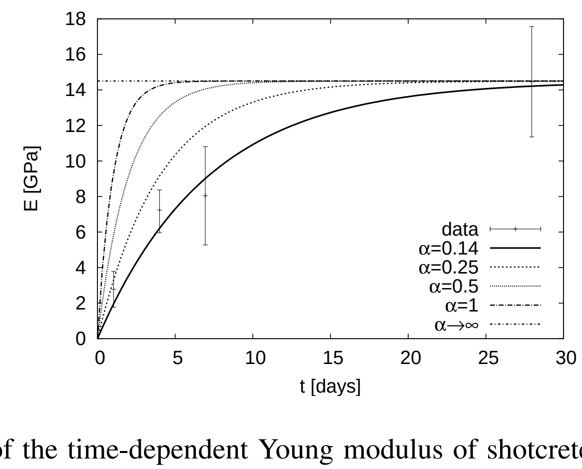


Figure 6: Calibration of the time-dependent Young modulus of shotcrete. Experimental data by Bae et al. (2004).

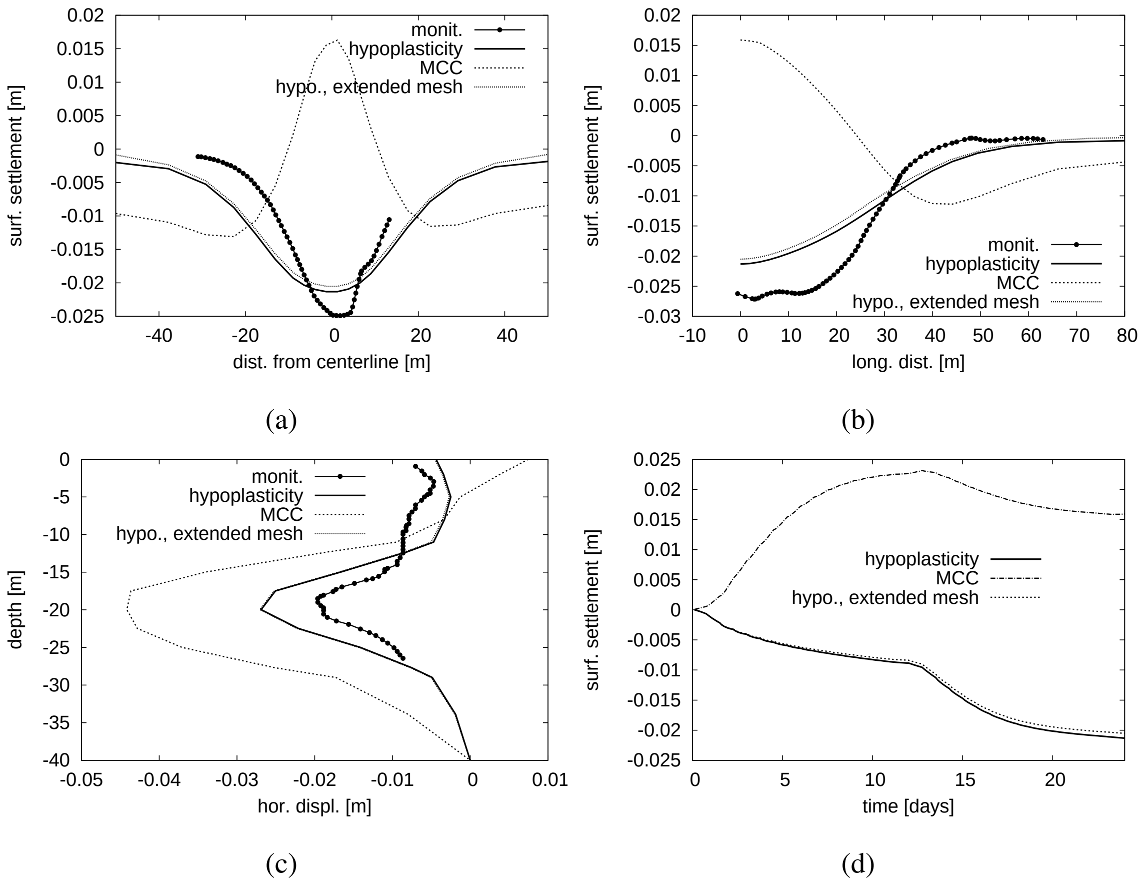


Figure 7: The influence of soil constitutive model on numerical predictions (monitoring data from Deane and Basset 1995).

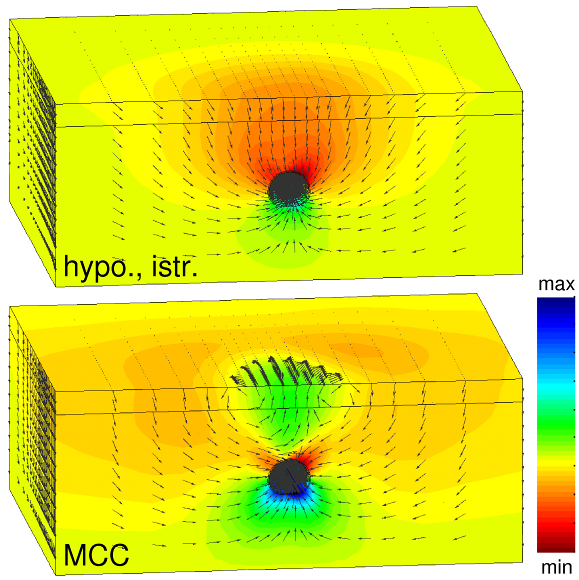


Figure 8: Predictions by different constitutive models. Contour lines show vertical displacement (scale $\min = -0.06$ m, $\max = 0.1$ m), vectors show displacement direction.

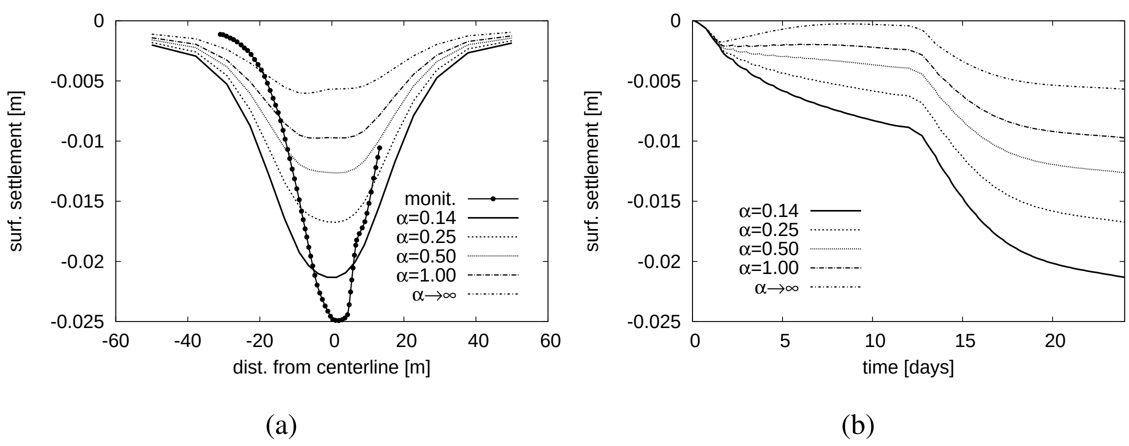
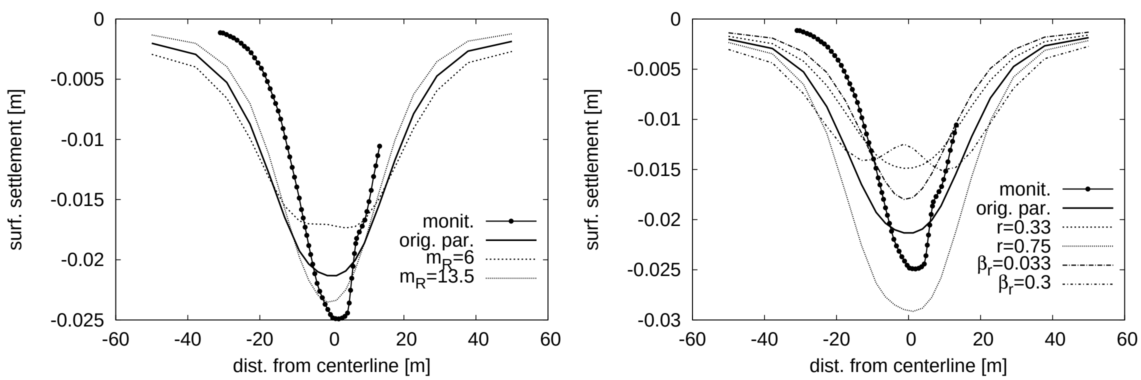


Figure 9: The influence of the shotcrete stiffness vs. time relationship on numerical predictions (monitoring data from Deane and Basset 1995).



(a) (b)

Figure 10: The influence of the small-strain stiffness characteristics, predictions by the enhanced hypoplastic model (monitoring data from Deane and Basset 1995).