The influence of a constitutive model on predictions of a NATM tunnel in stiff clays

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Summary

The paper presents predictions of a displacement field induced by NATM tunnelling in stiff clays with high K_0 conditions. Initial conditions (stresses, void ratio, K_0) are taken from accurate field measurements, constitutive models are calibrated solely on the basis of good quality laboratory experimental data, and the excavation sequence is approximated realistically within 3D finite element analysis. Predictions obtained with four constitutive models are compared: Modified Cam clay model, hypoplastic model for clays in basic form and enhanced by the intergranular strain concept and kinematic hardening 3-SKH model. The most accurate predictions are obtained with the enhanced hypoplastic model. The investigation indicates that predicting soil non-linearity in the small strain range is the most important factor conrolling the results.

1 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 recognised that conventional finite element (FE) 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, the following reasons of this discrepancy appear to be the most important: small-strain non-linearity and high initial stiffness, soil anisotropy, neglecting 3D effects in 2D FE analysis and K_0 conditions.

This paper aims at investigating to what extent it is possible to reach reliable predictions using advanced material models with: (1) Initial conditions (namely K_0 and e) set up according to the most accurate field measurements available; (2) Model parameters (for both soil and shotcrete) calibrated solely on the basis of high quality laboratory experimental data and (3) realistic modelling of the excavation sequence in 3D FE analysis. The Heathrow Express trial tunnel [Deane&Basset, 1995], a NATM tunnel excavated in London Clay, has been chosen for this purpose as detailed monitoring data are available and as the London Clay properties have been studied thoroughly in the past. Results from detailed experimental and field studies performed at Imperial College, London, at the Heathrow Terminal 5 site [Hight et al., 2007, Gasparre, 2005] are used for calibration of the models.

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2 Details of numerical analyses

The finite element analyses have been performed using finite element program *Tochnog Professional*. Different constitutive models have been implemented by means of a user-defined material model subroutine *umat*, which has originally been developed for the *ABAQUS* (a trademark Abaqus, Inc., USA.) 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 (RKF-23) and numerical estimation of the consistent (algorithmic) stiffness matrix is used for the time integration of the constitutive models. The *umat* subroutine for all constitutive models used in this paper is freely available on the web [Gudehus et al., 2007].

Problem geometry corresponds to the Heathrow Express trial tunnel [Deane&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. The mesh consists of 7636 8-noded brick elements, which were used to model both soil and tunnel primary lining. The tunnel 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 top five meters represent gravel sediments of the river Thames and made ground, modelled as a single material. The 35 m below represent the London Clay strata.

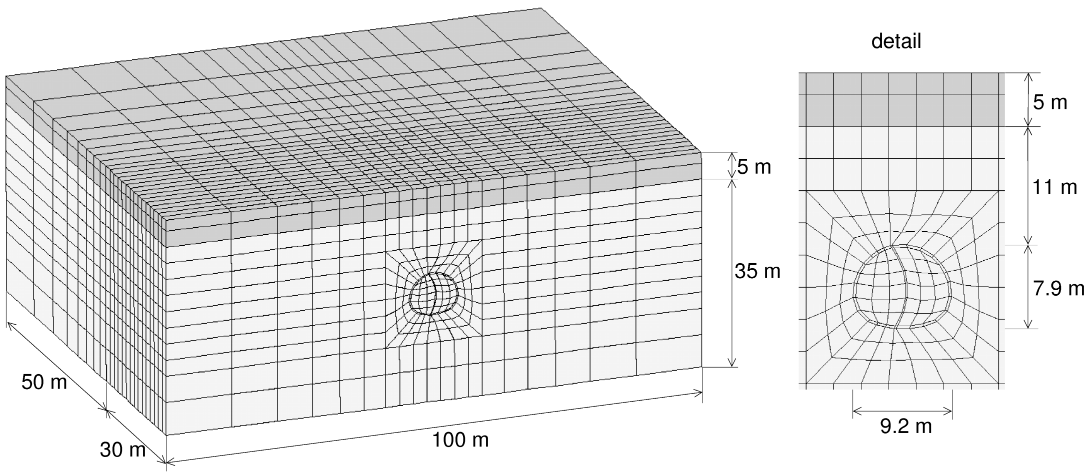


Figure 1: Finite element mesh used in the analyses.

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&Fowell, 2005, Bowers, 1997]. Fig. 2 also shows the approximation of the excavation sequence used in FE analyses.

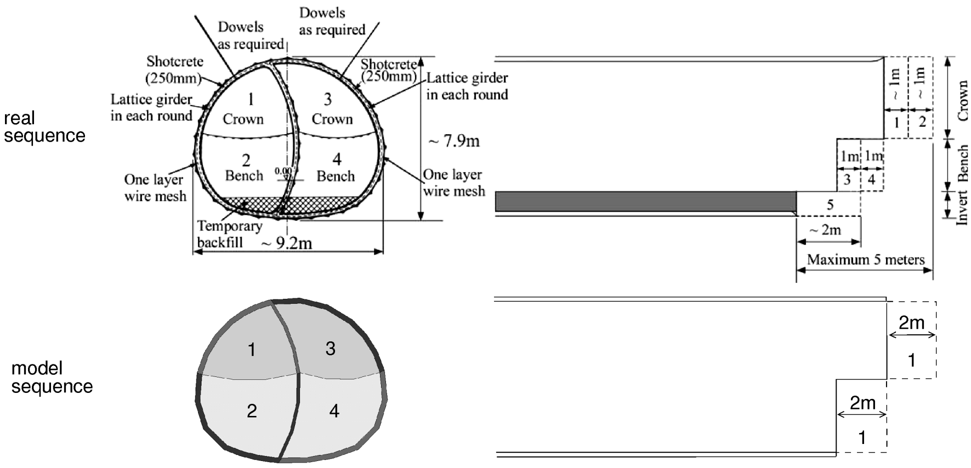


Figure 2: "Type 2" excavation sequence as performed in the trial tunnel experiment (top) and as approximated in FE simulation (bottom) ("real sequence" from [Karakus&Fowell, 2005, Bowers, 1997]).

Initial conditions consist of the vertical effective stress, horizontal effective stress (calculated through the K_0 value) and void ratio e. In the London Clay, the initial void ratio e is calculated from the water content profile given in [Hight et al., 2007]. Available measurements are shown in Fig. 3a. Although there is a scatter in the experimental data, no clear trend the water content with depth can be observed. An approximate average value $w_c = 25.5$ %, has been adopted, which leads to $e = 0.7$. The assumed water content leads to the total unit weight of the saturated clay $\gamma = 19.6$ kN/m^3. Vertical stresses are calculated by assuming full saturation and ground water table level 2 m below the ground surface. [Hight et al., 2007] provided also K_0 profile measured by suction probes, given in Fig 3b. The K_0 value varies significantly with depth and it reaches relatively high values. Above the tunnel $K_0 \geq 1.5$.

All analyses have been performed as undrained using a penalty method, that is by adding a water bulk modulus K_w term to the material stiffness matrix (e.g., [Potts&Zdravković, 1999]). Suitable value of K_w has been found by comparing results of preliminary 2D coupled consolidation analyses with a realistic anisotropic permeability profile [Hight et al., 2007, Kovacevic et al., 2007] and undrained analyses. The chosen value $K_w = 10^5$ kPa led to practically identical shapes of the normalised surface settlement troughs, the undrained analysis predicted approximately 25% lower vertical surface settlements above the tunnel centreline than the corresponding coupled consolidation analysis.

3 Constitutive models for soil

In this paper, constitutive modelling focuses on detailed description of the London Clay behaviour. Particular attention is paid to the calibration of the models, which is always done solely on the basis of laboratory experimental data, rather than by back-analysis of field measuremens. The Thames gravel (top five meters of the modelled sequence) is

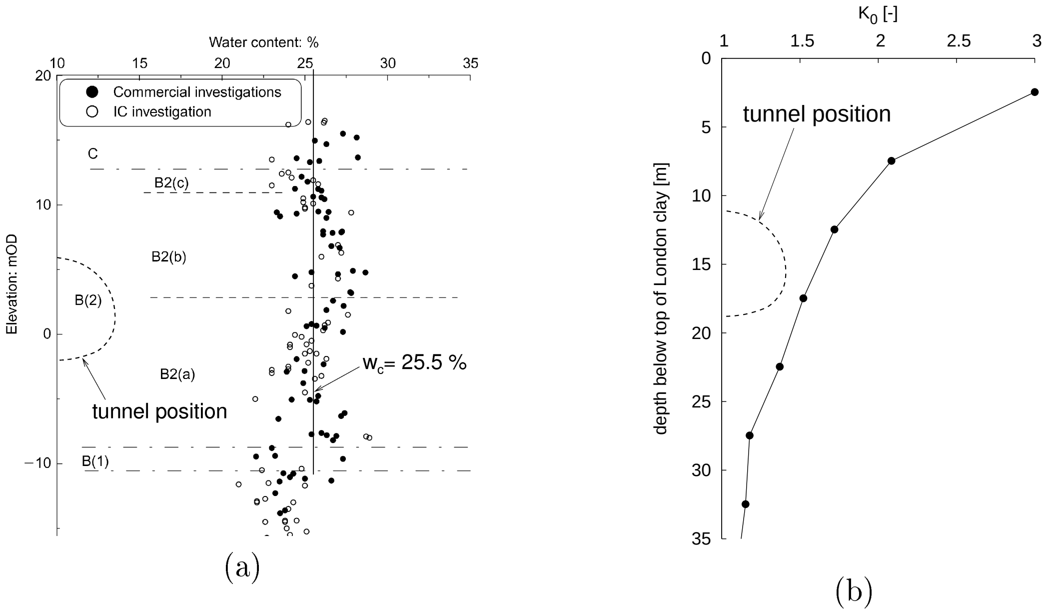


Figure 3: (a) Water content profile through the London Clay (from [Hight et al., 2007], modified), (b) K_0 profile used in all analyses (data from [Hight et al., 2007]).

for simplicity modelled in all analyses using a basic elasto-plastic Mohr-Coulomb model, with $E = 75$ MPa, $\nu = 0.25$, $\varphi = 35°$, $c = 0$ kPa and $\psi = 17.5°$.

3.1 Hypoplastic model for clays

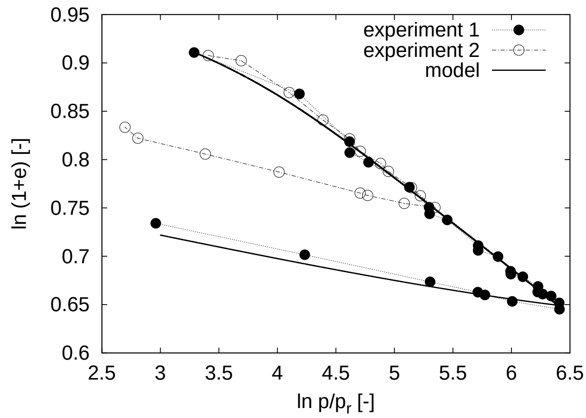
The first advanced model used in this work is a *hypoplastic constitutive model for clays* (abbreviated in this paper as "basic hypoplastic model", or "hypo., basic"), proposed by [Mašín, 2005] and investigated further by [Mašín&Herle, 2005].

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

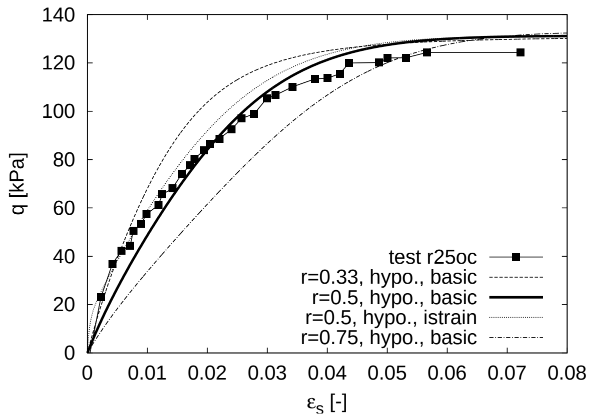
$$\ln(1+e) = N - \lambda^* \ln(p/p_r) \tag{1}$$

where $p_r = 1$ kPa is a reference stress; parameter κ^* defines slope of the isotropic unloading line in the same plane. Their calibration using isotropic loading and unloading tests on *reconstituted* London Clay specimens by [Gasparre, 2005] is demonstrated in Fig. 4a.

Critical state friction angle $\varphi_c = 21.9°$ has been estimated by [Gasparre, 2005]. The last parameter r controls the shear modulus and it has been found by back-analysis of undrained shear test on reconstituted overconsolidated soil, see Fig. 4b. This figure shows that the chosen value $r = 0.5$ leads to slight under-estimation of the initial stiffness. The fit is, however, improved by the use of intergranular strain concept (discussed in the next section, see also Fig. 4b).



(a)



(b)

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

To use the model calibrated on reconstituted clay to predict the behaviour of natural London Clay, the influence of soil structure needs to be taken into account. Sensitivity framework by [Cotecchia&Chandler, 2000], incorporated into hypoplastic models by [Mašín, 2007], has been adopted in this respect. The effects of structure degradation are in stiff clays not important [Baudet&Stallebrass, 2004] and the parameter N is thus modified in the following way

$$N_{nat} = N_{rec} + \lambda^* \ln(s) \tag{2}$$

where N_{rec} is the value of parameter N calibrated using data on reconstituted clay, N_{nat} is its value used for predicting the behaviour of natural clay and s is a measure of soil structure. In this work, value $s = 2.5$, estimated for natural London Clay from sub-unit B2 by [Gasparre, 2005], is adopted. The parameters of the hypoplastic model are summarised in Table 1.

Table 1: *Parameters of the basic hypoplastic model.*

φ_c	λ^*	κ^*	N_{rec}	r	N_{nat}
21.9°	0.095	0.015	1.257	0.5	1.344

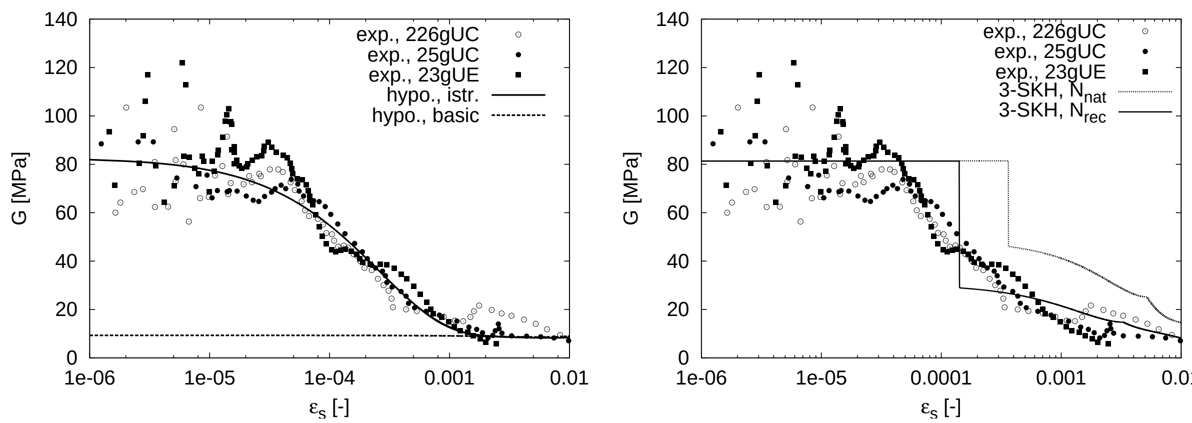
3.2 Hypoplastic model for clays with intergranular strain concept

The model discussed in the previous section 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 intergranular strain concept [Niemunis&Herle, 2007]. This model is in this paper abbreviated as "enhanced hypoplastic model", or "hypo., istr.".

The intergranular strain concept requires the following additional parameters: R controlling the size of the elastic range, β_r and χ controlling the rate of stiffness degradation and m_R and m_T controlling the initial stiffness. Parameters of the intergranular strain concept have been found based on experiments conducted on *natural* London Clay from sub-unit B2(a) [Gasparre, 2005]. Parameter m_R, controlling the initial stiffness, has been estimated using formula derived in [Mašín, 2005]:

$$G_0 \simeq \frac{m_R}{r\lambda^*} p \tag{3}$$

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. Bender element measurements on samples from sub-unit B2(a) by [Gasparre, 2005] gave $G_{0hv} \simeq 80$ MPa at $p = 420$ kPa, which leads to $m_R \simeq 9$. Parameters R, β_r and χ have been found by fitting the stiffness degradation curve of three undrained tests on specimens with pre-shear history designed to follow the actual stress history of the London Clay, see Fig. 5a. Clearly, the enhanced hypoplastic model reproduces closely the measured stiffness degradation curve. The basic hypoplastic model, which is incapable of predicting correctly the small-strain stiffness, is included in Fig. 5a for comparison. The parameters of the intergranular strain concept are provided in Table 2.



(a) (b)

Figure 5: Predictions of the stiffness degradation curve in the small-strain range by the hypoplastic (a) and 3-SKH (b) model (exp. data from [Gasparre, 2005]).

Simulations with the enhanced hypoplastic model require specification of the initial values for the new state variable intergranular strain ($\boldsymbol{\delta}$). It is 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. In the context of the enhanced hypoplastic model it implies the initial value $\boldsymbol{\delta} = \mathbf{0}$ [Niemunis&Herle, 2007].

Table 2: *Parameters of the intergranular strain concept.*

m_R	m_T	R	β_r	χ
9	9	5×10^{-5}	0.1	1

3.3 Three surface kinematic hardening model

Three surface kinematic hardening model (3-SKH) by [Stallebrass&Taylor, 1997], is an advanced example of the kinematic hardening plasticity models for soils, which makes it possible to model small-strain non-linearity within the elasto-plastic framework. Altogether, 10 parameters need to be specified (A, n, m, N, λ^*, κ^*, M, T, S and ψ).

Parameters N, λ^* and κ^* have the same meaning as the parameters of the hypoplastic model. M, slope of the critical state line in the p vs. q plane, is calculated from φ_c. The shear modulus inside the elastic range G is calculated from an equation by [Viggiani&Atkinson, 1995]

$$\frac{G}{p_r} = A \left(\frac{p}{p_r} \right)^n OCR^m \tag{4}$$

with parameters A, n and m and overconsolidation ratio OCR. The parameters T and S characterise relative sizes of the kinematic surfaces and the last parameter ψ controlls the rate of decay of stiffness as the state moves towards bounding surface.

The model parameters have been calibrated using the same data as used for the other models, with the exception of n, m, T and S, which were taken over from [Mašín, 2004] (experiments required for their calibration are not available from Terminal 5 site). The predicted stiffness degradation curve in the small strain range is shown in Fig. 5b. It is shown that the value of N_{nat} following from the sensitivity framework could not be used, as the measured shape of the stiffness curve could not be reproduced. For this reason, N_{rec} is used in simulations. The stiffness degradation curve is then overall predicted reasonably, although it is not possible to tune the model predictions into greater detail. Particularly, there is a jump in the stiffness curve – a common shortcoming of kinematic hardening models, which may be overcome by additional modifications [Grammatikopoulou et al., 2006]. They are, however, outside the scope of this paper. The parameters of the 3-SKH model are given in Table 3.

Table 3: *Parameters of the 3-SKH model.*

A	n	m	M	κ^*	λ^*	T	S	ψ	N_{rec}	N_{nat}
722	0.71	0.27	0.85	0.003	0.095	0.24	0.16	2	1.257	-

Simulations with the 3-SKH model require specification of the initial positions of the two kinematic surfaces. Following the same reasoning as in the case of the enhanced hypoplastic model, they were initially centred about the stress state. It is, however, noted, that according to the findings by [Grammatikopoulou et al., 2002] the initial position of the kinematic surfaces does not influence significantly the predicted tunnel-induced deformations.

In addition to predictions by the three advanced constitutive models, predictions by the *Modified Cam clay* model ([Roscoe&Burland, 1968]; abbreviated in this paper as "MCC") are included in the paper. This model is still widely used in practice and thus creates a valuable reference to the advanced models. The version with Butterfield's [Butterfield, 1979] compression law - Eq. (1) - and constant shear modulus G is used. MCC parameters adopted are provided in Table 4.

Table 4: *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

Model for shotcrete lining

The time-dependent stress-strain behaviour of shotcrete lining is rather complex. The complexity of the constitutive assumption for shotcrete varies significantly throughout literature on NATM tunnelling from linear elasticity, followed by linear elasticity with time-dependent Young modulus, to complex constitutive models based on viscoplasticity, time-dependent non-linear elasticity and chemomechanical models.

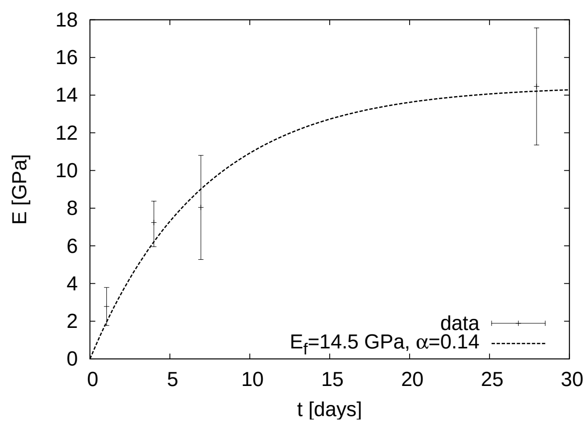


Figure 6: Calibration of the time-dependent Young modulus of shotcrete. Experimental data by [Bae et al., 2004].

In this work, linear elastic perfectly plastic von Mises model with time dependent Young modulus and time dependent strength has been considered, i.e. non-linear stress-strain behaviour, creep and relaxation have not been not taken into account. Preliminary analyses have shown that considering plasticity limit does not influence significantly the calculated results and therefore only results with time dependent linear elasticity are presented.

An empirical exponential dependency of shotcrete Young modulus E on time [Pöttler, 1990, Oreste, 2003] is assumed:

$$E = E_f \left(1 - e^{-\alpha t/t_r}\right) \tag{5}$$

where E_f is the final Young modulus, α is a parameter and $t_r = 1$ day is the reference time. Eq. (5) has been calibrated using experimental data by [Bae et al., 2004] (see Fig. 6), who tested a shotcrete with alkali-free accelerator. The selected parameters 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 behaviour. Using the time-dependent constitutive model required to set up the numerical model in such a way that the real time needed for the excavation was reproduced, which was 24 days for the whole trial tunnel, 12 days for each drift [Deane&Basset, 1995].

Results of analyses

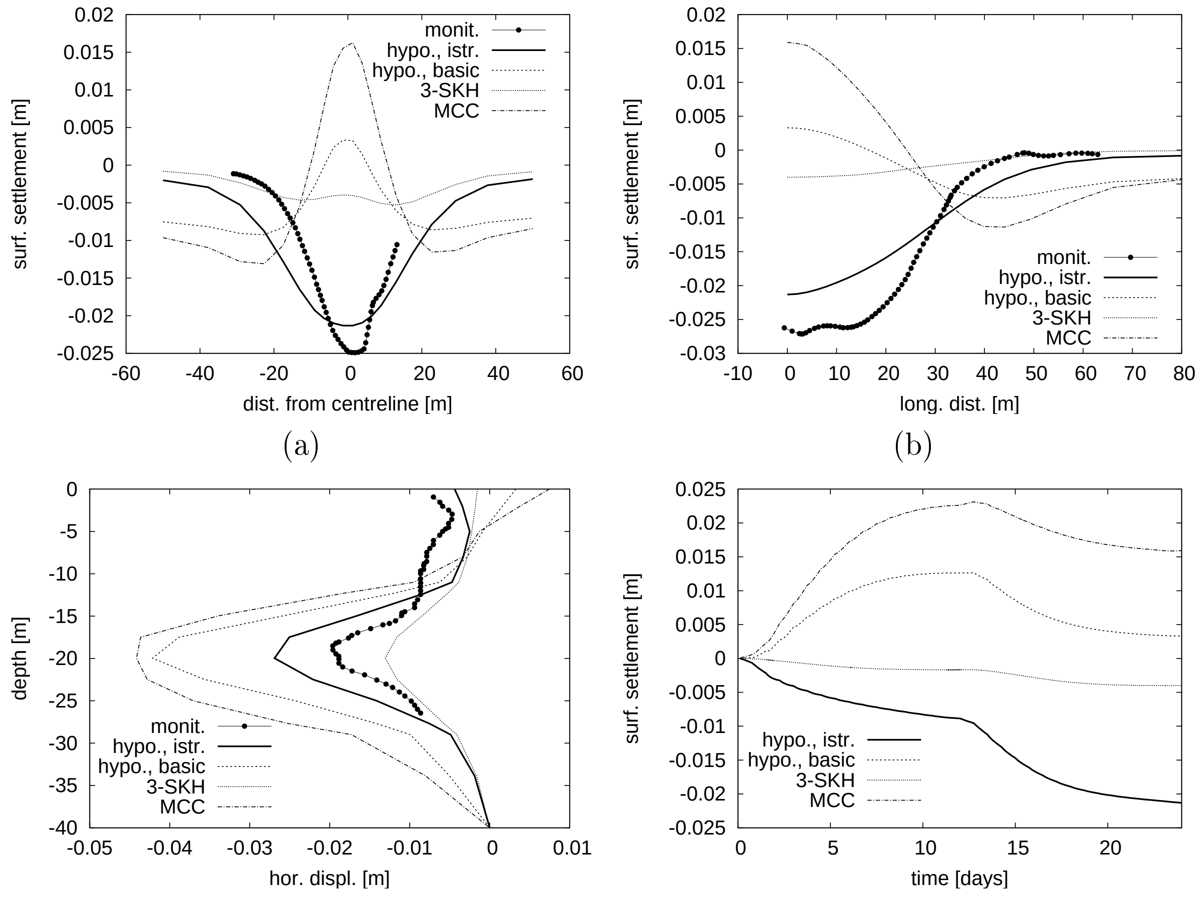
The influence of soil constitutive model on predictions is shown in Fig. 7. When possible, predictions are compared with monitoring data from [Deane&Basset, 1995, Bowers, 1997]. Figs. 7(a) and (b) show surface settlement troughs in transverse and longitudinal directions, respectively. Fig. 7c depicts horizontal displacements in the horizontal distance of 7.7 m from the tunnel centreline. Finally, Fig. 7d shows time development of surface settlements above the tunnel centreline.

The transverse settlement troughs in Fig. 7a correspond for both the simulation and experiment to the steady state conditions (no additional deformations with further face advance). As these conditions have been reached in simulations just when the tunnel face is the full tunnel length (30 m) from the monitoring point (Figs. 7b,d), the troughs are for simulations evaluated at the beginning of the modelled sequence. The monitoring section was located in the middle of the excavation sequence.

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

- The two models that are incapable of predicting stiffness in the very-small-strain range correctly – basic hypoplastic and MCC – give unrealistic predictions, with surface heave above the tunnel centreline and largest downward vertical displacement in a certain horizontal distance from the centreline (Figs. 7 a,b,d). These models also significantly overpredict horizontal displacements at the tunnel level (Fig. 7c). The other two models – enhanced hypoplastic and 3-SKH models – give more realistic predictions. Unlike the 3-SKH model, the enhanced hypoplastic model predicts reasonably also the displacement magnitude (as discussed later).

  Fig. 7 (d) shows that the surface heave predicted by "hypo., basic" and MCC models occurs exclusively during excavation of the left drift (the first 12 days in Fig.



(c) (d)

Figure 7: The influence of soil constitutive model on numerical predictions (monitoring data from [Deane&Basset, 1995])

7 d). For qualitative explanation of the deformation mechanism, lining displacement vectors during the initial stages of excavation are plotted in Fig. 8. High K_0 conditions cause large horizontal stresses and consequently large horizontal inwards displacements of the lining. The initial stiffness predicted by the "hypo., basic" and MCC models is too low and lining acts in this case as a rather rigid ring, pushing the soil upwards at the top corner. The "hypo., istr." and 3-SKH models predict a high enough initial stiffness to suppress this effect.

The overall displacement field at the end of the tunnel excavation for the four models is shown in Fig. 9. The figure nicely demonstrates the upward heave of the soil wedge above the tunnel caused by high lateral stresses predicted by the "hypo., basic" and MCC models, and the more realistic predictions by "hypo., istr." and 3-SKH models (note that displacement contours by the 3-SKH model are plotted at different scale, see legend of Fig. 9).

- When comparing simulations by the "hypo., basic" and MCC models, it is clear that "hypo., basic" capability of predicting non-linear large-strain behaviour and stiffness increase with the stress level leads to a certain improvement in predictions.

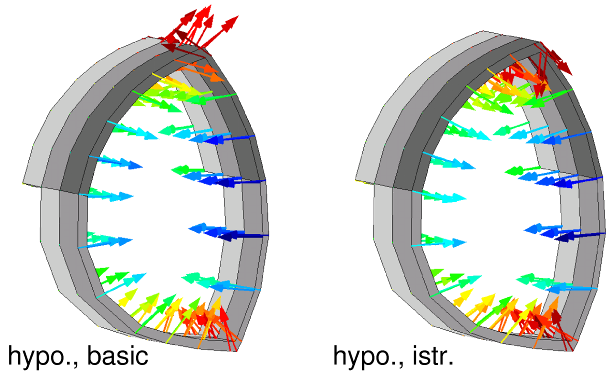


Figure 8: Displacements vectors of tunnel lining in the initial stages of excavation for basic and enhanced hypoplastic models.

- The enhanced hypoplastic model gives the most accurate predictions, both in the qualitative and quantitative way. Still, the predicted surface settlement trough (in both transversal and longitudinal directions) is wider then the observed one and horizontal displacements in the tunnel level are slightly overestimated. This is, however, a common shortcoming of predictions of deformations induced by tunneling in high K_0 environment. Further improvement would be reached by incorporation of anisotropy.

- Although the 3-SKH model gives qualitatively better predictions than the basic MCC model, it still predicts smaller surface settlements above the centreline than at a distance from it, and it underpredicts the displacement magnitude. This may be attributed to its incapability of following closely the stiffness degradation curve (Fig. 5).

4 Concluding remarks

The displacement field induced by NATM tunnelling in high K_0 environment can be predicted with reasonable accuracy without the necessity to make unrealistic assumptions about the initial conditions, boundary conditions, or the constitutive model parameters. The analyses with an advanced constitutive model (an enhanced hypoplastic model) gave satisfactory estimate for the settlement magnitude and slightly overestimated the settlement trough width.

Detailed conclusions from the present work have been outlined in the section describing the results of analyses. In the following, basic inputs needed to reach the predictions will be recapitulated. The list can serve as a guide when designing the geotechnical site investigation program in future projects in a similar environment. (1) Advanced constitutive model for soil that is capable of predicting the soil behaviour from the very-small-strain to large-strain range. The model chosen (enhanced hypoplastic model) is not complex from the end-user point of view (simple calibration procedure) and its FE

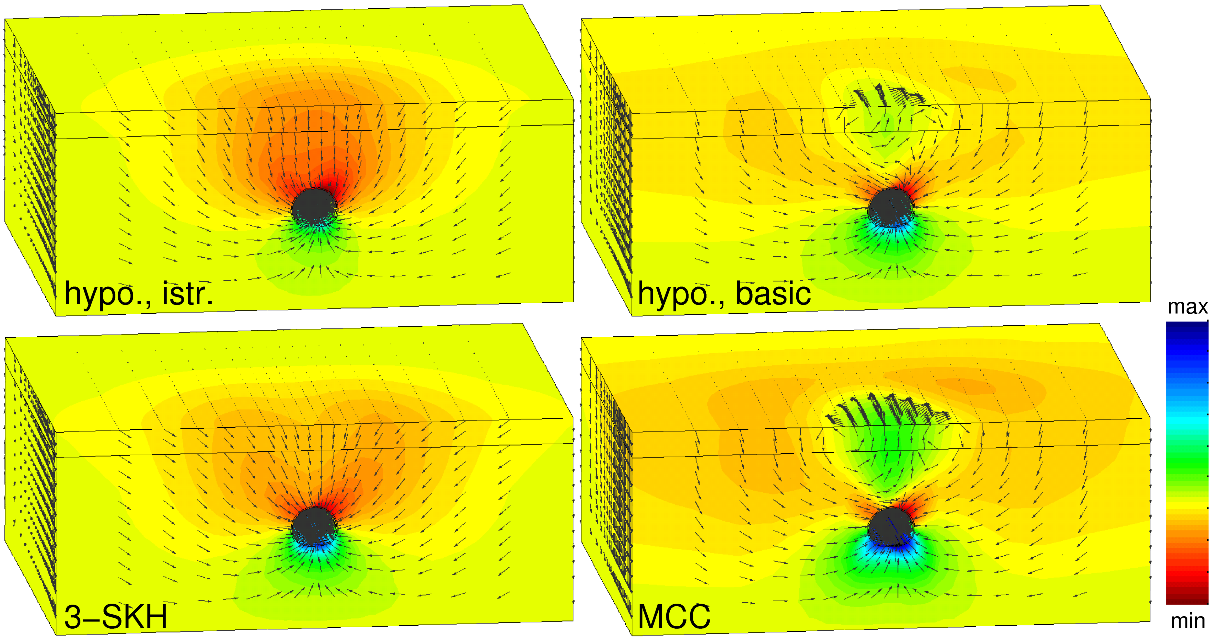


Figure 9: Predictions by different constitutive models. Contour lines show vertical displacement (scale $\min = -0.02$ m, $\max = 0.033$ m for the 3-SKH model, $\min = -0.06$ m, $\max = 0.1$ m for other models), vectors show displacement direction.

implementation is freely available. (2) Isotropic loading and unloading test and undrained shear test on a reconstituted soil. (3) Isotropic compression test on undisturbed soil to estimate the degree of soil structure; Undrained shear test on undisturbed soil with bender element and local strain measurements to estimate small-strain stiffness characteristics. (4) Field determination of K_0 and water content profiles. (5) Experiments characterising shotcrete Young modulus vs. time dependency. (6) Reasonable approximation of the excavation sequence within 3D finite element analysis.

5 Acknowledgment

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