3D simulations of a NATM tunnel in stiff clays with soil parameters optimised using monitoring data from exploratory adit

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ABSTRACT: The paper demonstrates the application of a hypoplastic model in class A predictions of a NATM tunnel in an urban environment. The tunnel, excavated in a stiff clay, is 14 m wide with 6 m to 21 m of overburden thickness. The constitutive model was calibrated using laboratory data (oedometric and triaxial tests) and the parameters were optimised using monitoring data from an exploratory drift. Based on the optimised data set, the future tunnel was simulated. After the tunnel excavation, it could be concluded that the model predicted correctly surface settlements, surface horizontal displacements, and the distribution of vertical displacements with depth. It overpredicted horizontal displacements in the vicinity of the tunnel.

1 INTRODUCTION

The main issue of tunnelling in urban environment, typically characterised by a low overburden thickness and presence of surface infrastructure, is the control of settlements induced by tunnel excavation. The first step in the design of any protective measure reducing the tunnel impact on surrounding buildings is an accurate prediction of the tunnelling-induced displacement field.

The present paper demonstrates the application of an advanced constitutive model (Mašín 2005) in predictions of a complex tunnelling problem in urban environment. The goal was to provide class A predictions of the displacement field induced by a 14 m wide road tunnel in stiff clay, with an overburden of 6 m to 21 m. The parameters of the constitutive model were calibrated on laboratory data and optimised using monitoring data from an exploratory drift. The drift was located in top heading of the future tunnel. Based on the optimised data set, class A predictions of the displacement field induced by the tunnel were performed in 2008 and early 2009. In November 2009, the full profile of the tunnel passed the simulated cross-section, which allowed us to compare the predictions with the data from the geotechnical monitoring.

2 KRÁLOVO POLE TUNNELS

The Královo Pole tunnels (often referred to as Dobrovského tunnels) form a part of the northern section of the ring road of Brno town in the Czech Republic. The tunnels consist of two parallel tubes with a separation distance of about 70 m and lengths of approximately 1250 m. The tunnel cross-section height and width are about 12 m and 14 m respectively, and the overburden thickness varies from 6 m to 21 m. The tunnels are driven in a developed urban environment (see Fig. 1). The displacement field induced by the tunnel excavation was thus an important issue the designers had to cope with.

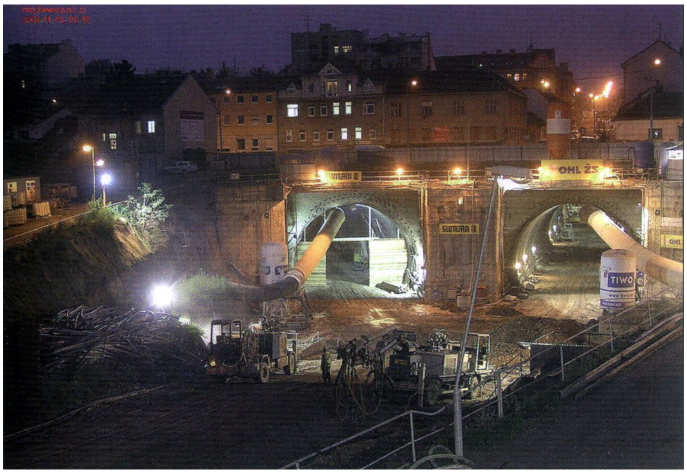

Figure 1. Temporary portals of the Královo Pole tunnels (Horák 2009).

The geological sequence in the area is shown in Fig. 2. From the stratigraphical point of view, the area is formed by Miocene marine deposits of the Carpathian fore-trough, the thickness of which reaches several hundreds meters in this location

(Pavlík et al. 2004). The top part of the overburden consists of anthropogenic materials. The natural Quaternary cover consists of loess loam and clayey loam with the thickness of 3 to 10 m. The base of the Quaternary cover is formed by a discontinuous layer of fluvial sandy gravel, often with a loamy admixture. The majority of the tunnel is driven through the Tertiary calcareous silty clay. The clays are of stiff to very stiff consistency and high plasticity.

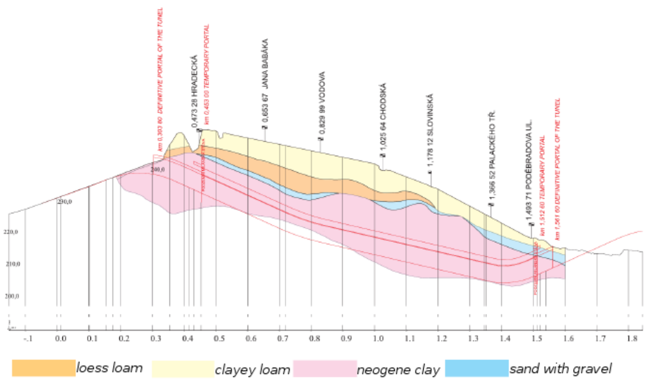

Figure 2. Longitudinal geological cross-section along the tunnels (Pavlík et al. 2004).

Before the Královo Pole project, there was only little experience with the response of the Brno clay to tunnelling. In order to clarify the geological conditions of the site, and in order to study the mechanical response of the Brno clay, a comprehensive geotechnical site investigation programme was designed, the crucial part of it being an excavation of three exploratory drifts. The drifts were triangular in cross section with the side-length of 5 m and were designed to form parts of the top headings of the future tunnels (Fig. 3). The total length of the three drifts was over 2000 m. For technological reasons, they were not driven along the complete length of the future tunnels.

Figure 3. Exploratory drifts situated in the top headings of the future Královo Pole tunnels.

The tunnels were driven by the New Austrian Tunneling Method (NATM), with sub-division of the face into six separate headings (Fig. 4). The face subdivision, and the relatively complicated excavation sequence, were adopted in order to minimise the surface settlements imposed by the tunnel.

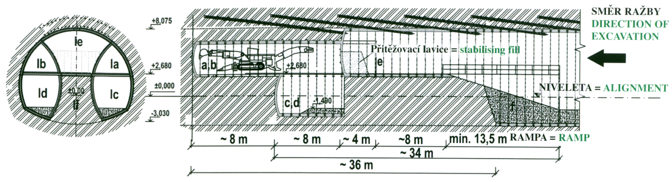

Figure 4. Sketch of the excavation sequence of the tunnel (Horák 2009).

3 LABORATORY EXPERIMENTS AND CONSTITUTIVE MODEL CALIBRATION

The behaviour of loess loams and sandy gravels was found not to influence significantly the predicted tunnel performance (Svoboda et al. 2010), the laboratory experiments thus focused on the behaviour of Brno clay. Triaxial CIUP tests and oedometric tests were performed in order to calibrate the selected constitutive model. The triaxial specimens were equipped with submersible local LVDT axial strain transducers in order to evaluate the soil stiffness in the small strain range. In addition, one specimen was equipped with bender elements to measure the soil stiffness in the very small strain range by means of propagation of shear waves. Oedometric tests have been performed on undisturbed and reconstituted specimens. The specimens were loaded up to axial pressures of 13 MPa in order to find the position of the normal compression line and in order to evaluate the apparent overconsolidation ratio, which is also used for estimation of the coefficient of earth pressure at rest K_0.

The mechanical behaviour of the Brno clay was simulated using the hypoplastic model for clays (Mašín 2005) enhanced by the concept of intergranular strains (Niemunis and Herle 1997). This model was selected to represent the advanced constitutive models, which are capable of predicting the nonlinear soil behaviour, with high stiffness at very small strains and a nonlinear decrease of stiffness with increasing strain level. The implementation of the model into various finite element programs (such as Plaxis, ABAQUS, Tochnog Professional) is freely available on the internet (Gudehus et al. 2008).

For details of the model calibration, see Svoboda et al. (2010). The parameters of the model are summarised in Table 1.

Table 1. *Brno clay parameters of the clay hypoplastic model.*

φ_c	λ^*	κ^*	N	r
19.9°	0.128	0.01	1.506	0.45
m_R	m_T	R	β_r	χ
16.75	16.75	0.0001	0.2	0.8

4 SIMULATION OF THE EXPLORATORY DRIFT

The finite element predictions of the exploratory drift and of the whole tunnel were performed using software Tochnog Professional. The geometry of the exploratory drift and the finite element mesh consisting of 4680 8-noded brick elements are shown in Fig. 5. The evaluated cross-section corresponded to the front boundary of the finite element model. It was checked that no additional displacements at the evaluated cross-section were caused by the further advance of the drift face. Steady state conditions were thus reached. The mesh density was selected to be approximately the same for the drift and full tunnel simulations (Sec. 5). The analyses were performed as undrained using penalty approach with bulk modulus of water equal to $K_w = 100$ MPa. This procedure is described in Mašín (2009). 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 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 bottom 27.7m thick stratum represent the Brno clay and it has been simulated using the hypoplastic model with parameters from Tab. 1. The overlying layers of loams and gravels were simulated using the Mohr-Coulomb model with the parameters obtained during the site investigation (Pavlík and Rupp 2003) (Table 2). The shotcrete lining was simulated using continuum elements in the 3D model. Its was modelled by a linear elasticity with time dependent stiffness calculated using an empirical relationship (Oreste 2003)

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

where E_f is the final Young modulus, α is a parameter and $t_r = 1$ day is the reference time. The same parameters as the ones adopted by Mašín (2009) were used in the simulations ($E_f = 14.5$ GPa and $\alpha = 0.14$). The simulated excavation sequence represented the one adopted on the site. An excavation step of 1.2 m was followed by the lining installation.

The initial conditions of the simulation consisted of the determination of vertical stresses, the void ratio and the coefficient of earth pressure at rest K_0. The vertical stress was calculated from the unit weight

Table 2. *Mohr-Coulomb model parameters of the layers overlying the Brno clay strata.*

	φ [°]	c [MPa]	ψ [°]	E [MPa]	ν [-]
backfill	20	10	4	10	0.35
loess	28	2	2	45	0.4
sandy gravel	30	5	8	60	0.35

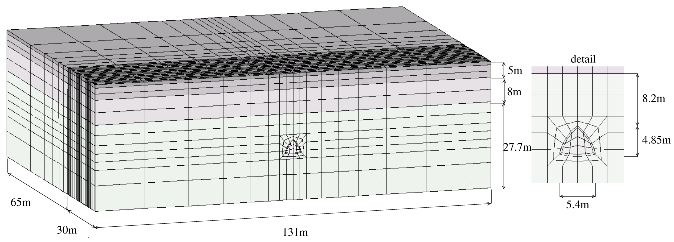

Figure 5. Finite element mesh used in the analyses of the exploratory drift.

of soil: γ=18.8 kN/m^3 for clay, 19.5 kN/m^3 for secondary loess and 19.6 kN/m^3 for sandy gravels. Water table corresponded to the Brno clay - sandy gravel interface. The initial void ratio of the Brno clay e=0.83 was derived from the undisturbed samples from both boreholes.

Because no reliable *in-situ* measurements of K_0 were available in the Brno clay massif, two extreme values of K_0 were considered in the analyses. First, the value of K_0 was determined from Mayne and Kulhawy (1982) empirical relationship:

$$K_0 = (1 - \sin\varphi_c) OCR^{\sin\varphi_c} \tag{2}$$

The overconsolidation stress of 1800 kPa was estimated on the basis of the oedometer tests on the undisturbed Brno clay samples, with the corresponding overconsolidation ratio (OCR) of 6.5, leading to K_0=1.25. The calculation of K_0 according to Eq. (2) assumes that the apparent soil overconsolidation was caused by the actual soil unloading resulting from the erosion of overlying geological layers. Creep represents the second possible interpretation of the measured overconsolidation. This interpretation would lead to the K_0 value calculated from the Jáky (1948) relationship:

$$K_0 = 1 - \sin\varphi_c \tag{3}$$

leading to $K_0 = 0.66$. K_0 of the layers overlying Brno clay was always calculated from (3) using the friction angles from Tab. 2.

The procedure of the analyses was as follows. First, the drift was simulated using the 3D finite element method. The next step was optimisation of the model parameters to account for inaccuracies of the description of soil massif based on small-size laboratory specimens. As the optimisation was CPU demanding and not feasible in 3D, an equivalent 2D model to the presented 3D model was developed. After the optimisation stage, the drift was simulated in 3D using the optimised parameter set. The 2D model was based on the load reduction method (Schikora and Fink 1982). The load reduction factor λ^d was calculated to ensure that the 3D analyses and the equivalent 2D analyses predicted as closely as possible the surface settlement troughs. The actual factors λ_d for the drift simulations were $\lambda_d = 0.50$ (for $K_0 = 1.25$) and $\lambda_d = 0.53$ (for $K_0 = 0.66$). The adequacy of the 2D representation had been demonstrated in a separate paper (Svoboda and Mašín 2010). The 3D and equivalent 2D models gave comparable predictions of the displacement fields, apart from the displacements in the very close vicinity of the tunnels.

4.1 Optimisation of the model parameters

In geotechnical practice, a common problem is that due to the size effects, sampling disturbance and limitations of experimental devices laboratory specimens do not represent the behaviour of the soil massif with sufficient accuracy. For this reason, the soil parameters calibrated by means of laboratory experiments have been corrected using an inverse analysis of the exploratory drift. The corrected parameters were then used for the class A predictions of the deformations due to the tunnel.

In the optimisation stage we focused on the shear stiffness. Namely, the parameter r controlling the large-strain shear stiffness as well as the small-strain shear stiffness was optimised. As the value of K_0 has also remarkable influence on the results, all the simulations were performed with two extreme K_0 values (as explained above). The inverse analysis has been performed using the software UCODE. In the inverse analysis, the parameter values are automatically adjusted until the model's computed results match the observed behaviour of the system (Finno and Calvello 2005). In the analyses, results of the simulations were compared with the measurement of the vertical displacements at several locations. Three locations were at the surface, where the vertical displacements were measured by means of geodetic survey. The fourth monitoring point was located just above the drift crown and it was monitored by means of an extensometer. The differences between the simulation and the monitoring data were expressed in terms of an objective function $S(b)$ (Finno and Calvello 2005) which takes the form:

$$S(b) = [y - y'(b)]^T \omega [y - y'(b)] \tag{4}$$

where b is a vector containing the values of parameters, y is a vector of observations, $y'(b)$ is a vector of the computed values corresponding to the observations and ω is the weight matrix. The weight matrix evaluates the significance of each measurement. Typically, the weight of each observation is taken as the inverse of its error variance (Finno and Calvello 2005). In the present case, with a low number of observations, however, each of the four observations is given the same weight equal to unity. UCODE performs the optimisation using by means of minimisation of the objective function $S(b)$ using the modified Gauss-Newton method.

The surface settlement troughs predicted with the original and optimised parameter sets, compared with the monitoring data, are shown in Fig. 6. Clearly, the model predicts reasonably both the settlement trough shape and magnitude already with the original parameter set. The optimisation procedure leads to a slight increase of the parameter r (Tab. 3) and a further improvement in predictions.

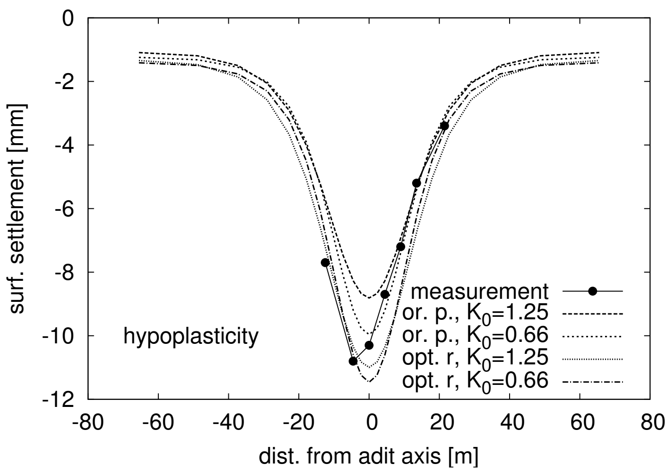

Figure 6. Surface settlement troughs due to exploratory drift predicted with the original parameter set ("or. p.") and with optimised value of the parameter r ("opt. r").

Table 3. *Original and optimised values of the parameter* r.

parameter set	r
original param.	0.45
optimised r, $K_0 = 1.25$	0.51
optimised r, $K_0 = 0.66$	0.49

5 3D SIMULATIONS OF THE KRÁLOVO POLE TUNNELS

As the last step in the investigation, the whole tunnel was simulated in 3D with the optimised parameter set. The results represent class A predictions, as they were performed in the period 2008 to early 2009, and thus before the tunnel excavation passed the simulated cross-section (November 2009).

In 3D, the finite element mesh consisting of 18 352 8-noded elements was used. The mesh and the modelled geometry are shown in Fig. 7. As in the case of the drift simulations, the evaluated cross-section was located at the front tunnel boundary. Steady-state conditions with no additional displacements with further advance of the tunnel face were reached. Other details of the analyses (drainage conditions, boundary conditions) were the same as in the exploratory drift simulations (Sec. 4). The tunnel lining was modelled using continuum elements with the thickness of 0.35 m as a linear elastic material with time dependent stiffness (as in the case of the exploratory drift). A 100 meters long simulated portion of the tunnel corresponded to the tunnel chainage 0.790-0.890 km (Fig. 2). The models considered the complex excavation sequence with the tunnel face subdivided into 6 segments (Figure 8). The excavation was performed in steps 1 to 6 (Fig. 8) with an unsupported span of 1.2 m. A constant distance of 8 m was kept between the individual faces, except the distance between the top heading and the bottom, which was 16 m.

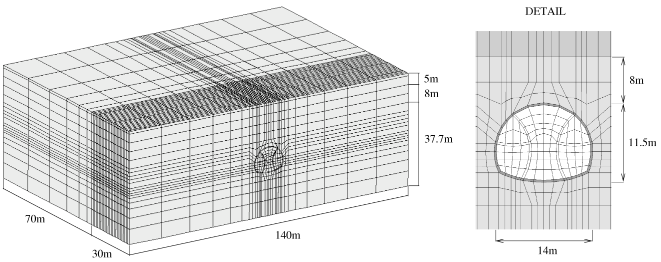

Figure 7. Finite element mesh used in the analyses of the whole tunnel.

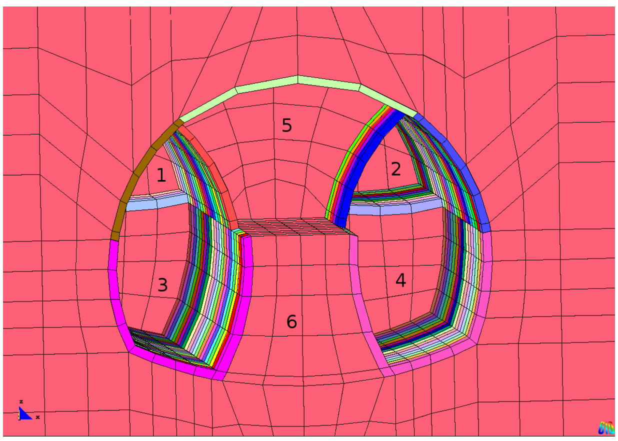

Figure 8. The excavation sequence as represented by the model.

The surface settlement troughs for both K_0 values are presented in Figure 9a. Due to the scatter of the monitoring data, several measurements at the chainages close to the simulated cross-section are presented. The agreement between the simulated and measured settlements is very good. The settlement magnitude is better predicted represented by the simulation with the higher K_0, while the trough shape is better predicted by the low K_0. Both predictions are on the safe side of the monitoring data (displacements are slightly overpredicted).

Figure 9b shows measurements of an extensometer located above the tunnel crown. The difference between the monitoring data and the simulations is approximately constant with depth, and corresponds to the slight overestimation of the surface settlements in Fig. 9a. Fig. 9b thus indicates that the hypoplastic model predicted correctly also the distribution of vertical displacements with depth, not only surface settlements.

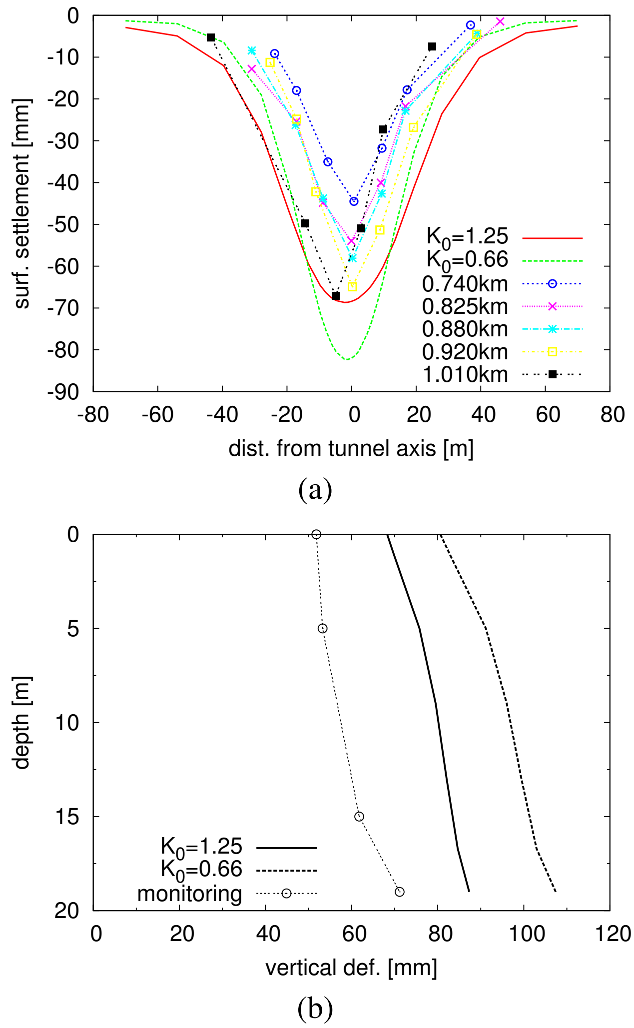

Figure 9. Surface settlement trough (a) and extensometer measurements (b). Class A predictions compared with the monitoring data.

Although the model predicted correctly the vertical displacement field, it significantly overestimated horizontal displacements in a vicinity of the tunnel in the tunnel depth. This is demonstrated in Fig. 10, showing the inclinometric measurements from an inclinometer located 3 m from the tunnel side. One of the possible reasons for this discrepancy is an absence of the small-strain stiffness anisotropy in the hypoplastic model. A similar problem was pointed out by Mašín (2009), who concluded that incorporation of the small-strain stiffness anisotropy into the hypoplastic model would improve the predicted shape of the settlement trough. This indicates a direction for the future development of the hypoplastic model. It is, however, necessary to stress out that although the horizontal displacements were overpredicted in the tunnel depth, their magnitude in the vicinity of the surface was predicted correctly. the correct predictions of the surface displacements are important in estimating the damage to the surrounding buildings.

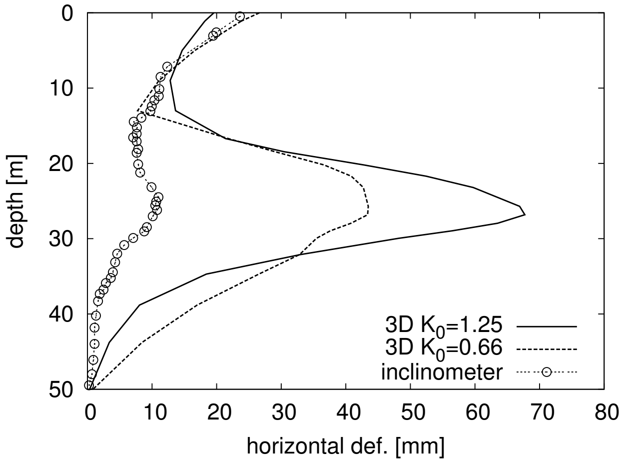

Figure 10. Inclinometric measurement, inclinometer located 3 m from the tunnel side.

6 CONCLUSIONS

It was shown that the application of an advanced soil constitutive model, in combination with quality experimental data and 3D finite element analysis, may lead to accurate forward predictions of the displacement field induced by a tunnel with low overburden thickness. The hypoplastic model for clays enhanced by the intergranular strain concept gave accurate predictions of the surface settlement, surface horizontal displacements, and the distribution of vertical displacements with depths. For both K_0 values adopted the model overpredicted the horizontal displacements in the vicinity of the tunnel.

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