3D FEM analysis of a NATM tunnel with shotcrete lining homogenization and stiffness anisotropy

Analyse 3D par la MEF du tunnel NATM avec le béton projeté homogénéisation du revêtement interne et l'anisotropie de rigidité

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ABSTRACT A stiffness anisotropy in very small strain range was implemented into a hypoplastic model for clays. Overconsolidated Brno clay was tested to obtain the anisotropic ratio of shear moduli and the NATM Královo Pole tunnel was numerically analysed using PLAXIS 3D software. To reduce the degree of uncertainty of the model, a composite steel/shotcrete primary lining homogenization was assumed. It was proven that the results of the analyses agree well with the geotechnical monitoring. In the analyses, we focused on the quantification of the earth pressure coefficient at rest K_0, which was found to be lower than 1 irrespectively of the fact that Brno clay is a stiff, overconsolidated clay.

RÉSUMÉ Une anisotropie de rigidité d'une amplitude très faible de déformation a été appliquée dans un modèle hypoplastique pour les argiles. L'argile surconsolidée de Brno a été testée pour obtenir le rapport anisotropique de modules de cisaillement, par suite le NATM du tunnel à Královo Pole a été analysé numériquement à l'aide du logiciels 3D PLAXIS. Pour minimaliser le degré d'incertitude du modèle, on a supposé une homogénéisation du revêtement interne du composite acier/béton. Il a été prouvé que les ci-mentionnées améliorations apportaient une valeur ajoutée au modèle et les résultats de l'analyse sont en accord avec la surveillance géotechnique. Le coefficient de pression des terres au repos K_0 agit comme une variable et sa valeur vraisemblable pour l'argile surconsolidée de Brno est inférieur à 1,0.

1 INTRODUCTION

In the geotechnical practice it is difficult to numerically estimate realistic displacement field in the vicinity of underground structure which is in interaction with massif. This is especially the case of predicting the NATM tunneling process with complicated face sub-division.

In this work, we simulated Královo Pole tunnel in Brno city in the Czech Republic, excavated in Brno clay ('Tegel') using the New Austrian Tunneling Method (NATM). The aim of the scientific activity was application and validation of an enhancement of the advanced constitutive hypoplastic model for clays (Mašín 2005) with the stiffness anisotropy in the very small strain range (Mašín 2014; Mašín & Rott, 2014) and the homogenization of composite steel/shotcrete primary lining (Rott, 2014). The degree of anisotropy of shear moduli α_G

$$\alpha_G = \frac{G_{pp0}}{G_{tp0}} \tag{1}$$

stands for the key parameter and was obtained from the bender element measurements carried out in the laboratory of the soil mechanics at the Faculty of Science.

2 IMPLEMENTATION OF STIFFNESS ANISOTROPY OF CLAYS IN THE VERY SMALL STRAIN RANGE

The description of the behavior of anisotropic clay is based on the theory of hypoplasticity, which is governed by the following primary equation:

$$\mathring{\boldsymbol{\sigma}} = f_s\left(\boldsymbol{\mathcal{L}} : \dot{\boldsymbol{\epsilon}} + f_d \mathbf{N}\|\dot{\boldsymbol{\epsilon}}\|\right) \tag{2}$$

where and represent the objective (Zaremba-Jaumann) stress rate and the Euler stretching tensor respectively, $\boldsymbol{L}$ and N are fourth- and second-order constitutive tensors, and f_s and f_d are two scalar factors. The model parameters correspond to the parameters of the earlier model by Mašín (2005), the model thus requires parameters φ_c (critical state friction angle), N (position of theisotropic normal compression line in the space of ln p vs. ln $(1+e)$), λ^* (slope of the isotropic normal compression line in the space of ln p vs. ln $(1+e)$), κ^* (parameter controlling volumetric response in isotropic or oedometric unloading), and ν (parameter controlling shear stiffness).

In order to predict small strain stiffness, the model has been enhanced by the intergranular strain concept by Niemunis & Herle (1997). This concept requires additional parameters. In particular, the very small strain shear modulus G_{tp0} is governed by equation

$$G_{tp0} = p_r A_g \left(\frac{p}{p_r}\right)^{n_g} \tag{3}$$

with parameters A_g and n_g and a reference pressure p_r of 1 kPa. In addition, there are parameters controlling the rate of stiffness degradation R, m_{rat}, β_r, χ. Finally, the adopted model allows for predicting the effects of stiffness anisotropy. An approach by Mašín & Rott (2014) has been followed. The most important parameter in this respect is the ratio of shear moduli in- and transverse to the plane of isotropy (already mentioned degree of anisotropy α_G), G_{pp0} and G_{tp0}. The description of the measurement of this key value is the aim of following paragraph.

The values of G_{pp0} were determined on horizontally cut specimens, and therefore after setting up in the triaxial cell the horizontal very small strain shear modulus was measured by a conventional pair of bender elements installed in the bottom and upper platens. The values of G_{tp0} were measured on different specimens cut vertically in a normal way. In all tests the back pressure of 500 kPa was applied. An example of the seismogram of test No 5 at the in-situ vertical stress of about 285 kPa is in Figure 1. Frequencies from 1 to 20 kHz were applied. At the arrival times of the shear waves were determined for frequencies of 3, 5, 7 and 9 kHz. Generally, at higher frequencies overshooting made it impossible to determine the first arrivals.

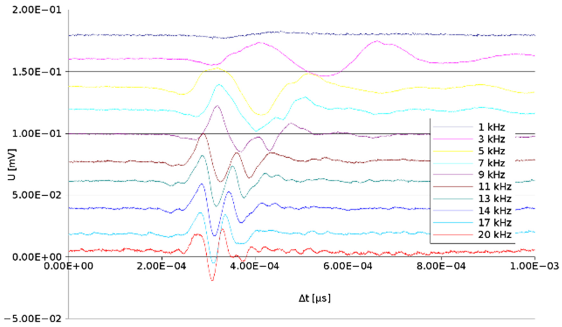

Figure 1. Seismogram of vertically travelling shear waves in specimen No 5

The measurement results are shown in Fig. 2. G_{pp0} is consistently higher than G_{tp0}. For α_G quantification, the results have been approximated by a linear fit (Fig. 2). Subsequently, the ratio α_G has been calculated from this fit as α_G=1.45.

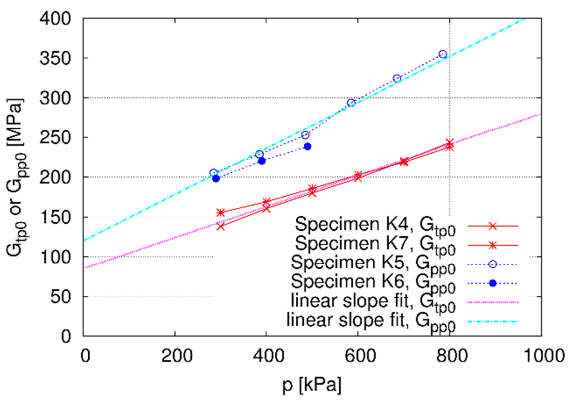

Figure 2. Results of bender element measurements of G_{pp0} and G_{tp0}.

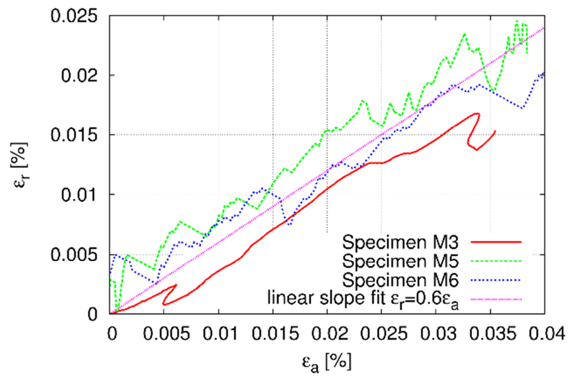

Figure 3. Relation between horizontal and vertical strain in the isotropic probe.

During triaxial tests, at the estimated in-situ mean effective stress (ca 300 kPa) isotropic probes consisting of 30 kPa loading and unloading were carried out (Fig. 3). The deformations were measured by submersible LVDTs and allowed us to quantify other anisotropy constants. For more details on experiments and their evaluation, see Rott et al. (2015).

3 HOMOGENIZATION OF STEEL/SHOTCRETE LINING

To increase the model accuracy and because of inability of the adopted FEM numerical code to assume 2 different components of primary lining, the homogenization of steel/shotcrete lining was considered (Rott, 2014). The main primary condition was conservation of static momentum, conservation of the axial and bending stiffness and modification due to the given constant value of the length of boring step. In addition, the dependence of Young modulus of SC on time is taken into the account. In the numerical model a linear elastic material was considered with time dependent SC stiffness calculated using an empirical relationship:

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

Young modulus of homogenized and modified lining $E_m(t_1, t_2)$ is determined for bending moment in the direction of tunnel axis and can be expressed as

$$E_m(t_1, t_2) = \sqrt{\frac{E_f^2\left(1-e^{-\frac{\alpha t_1}{t_r}}\right)^2 A_{NP}^3(t_1,t_2)}{12 I_{NP}(t_1,t_2) b_z^2}} \tag{5}$$

where b_z is the length of tunnel step. If one considers rectangular cross section of homogenized lining, the height of this shape $h_m(t_1, t_2)$ may be estimated from

$$h_m(t_1, t_2) = \frac{2\sqrt{3 I_{NP}(t_1,t_2)}}{\sqrt{A_{NP}(t_1,t_2)}} \tag{6}$$

Substitute moment of inertia $I_{NP}(t_1, t_2)$ and substitute cross section area $A_{NP}(t_1, t_2)$ is calculated with the usage of conversion ratios. The conversion of steel to older SC is governed by equation (7) for conversion ratio $n(t_1)$:

$$n(t_1) = \frac{E_O}{E_f\left(1-e^{-\frac{\alpha t_1}{t_r}}\right)} > 1 \tag{7}$$

and, analogically, for the conversion of younger SC to older SC the following equation (8) is valid:

$$m(t_1, t_2) = \frac{E_f\left(1-e^{-\frac{\alpha t_2}{t_r}}\right)}{E_f\left(1-e^{-\frac{\alpha t_1}{t_r}}\right)} = \frac{1-e^{-\frac{\alpha t_2}{t_r}}}{1-e^{-\frac{\alpha t_1}{t_r}}} < 1 \tag{8}$$

4 MODEL OF KRÁLOVO POLE TUNNEL AND THE RESULTS OF THE ANALYSIS

The Královo Pole tunnels (often referred to as Dobrovskeho tunnels) form an important part of the northern section (oriented SW-NE) of the ring road of Brno town in the Czech Republic. The tunnels consist of two mostly parallel tunnel tubes TT1 and TT2 with a separation distance of about 70 m and lengths of approximately 1250 m. The tunnel cross-section height and width are about 11.5 m and 14 m, respectively, and the overburden thickness varies from 6 m to 21 m. The tunnels are driven in developed urban environment. The excavation of the tunnels commenced in January 2008. The tunnels were driven by the New Austrian Tunneling Method (NATM), with sub-division of the face into six separate headings (Figure 3), excavation step was 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. Relatively complicated excavation sequences were adopted in order to minimize the surface settlements imposed by the tunnel.

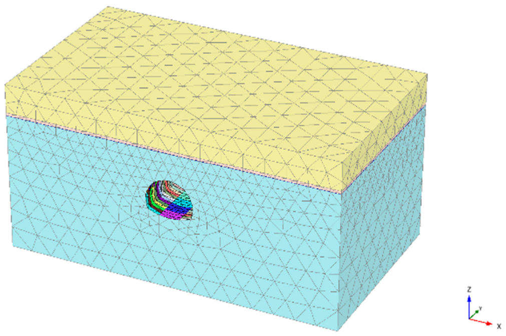

Figure 4. Model geometry of the region

3D numerical model of tunnel tube TT1 was created in PLAXIS 3D 2012 Software. Model was composed of 31,464 quadrilateral elements. The chosen constitutive material model of clay strata is represented by hypoplasticity with inherent stiffness anisotropy in the very small strain range. Simulated portion of the TT1 has 56.4 meters. Results from numerical analysis of TT1 were compared with monitoring data from inclinometer and from geodetically measured surface trough. The modeled field has dimensions 98 (width) x 50 (height) x 56.4 (length) meters. The thickness of clay overburden above the excavation is 7.9 m.The quaternary sediments in the model are represented by sand-gravel layer, sand-loess-loam and finally anthropogenic landfill which is only about 0.4 m thick. Due to its low thickness this layer was neglected. As a countermeasure, the unit weight of sand-loess-loam layer was magnified by 1 kN/m3. The overall height of overburden is therefore 17.2 m. The whole numerical analysis is composed of 76 phases and each phase represents progress of excavation by 1.2 m. The last phase is relevant for evaluation of the surface trough and the deformations were evaluated 3 m from the front side of the model. One excavation step 1.2 m long takes 8 hours. Always the first 1.2 m of excavation remains unsupported.

The measured and computed values agree relatively well. A parametric study for various pairs of K_0 – α_G (0.81 – 1.35; 0.60 – 1.70; 1.01 – 1.00) was carried out. The most reliable results seem to be for K_0 = 0.60 based solely on the tunnel simulations.

More detailed analysis presented by Rott et al. (2015) then yielded best estimate K_0 = 0.75: a remarkably low value, considering that the clay is stiff with apparent overconsolidation ratio of 7.

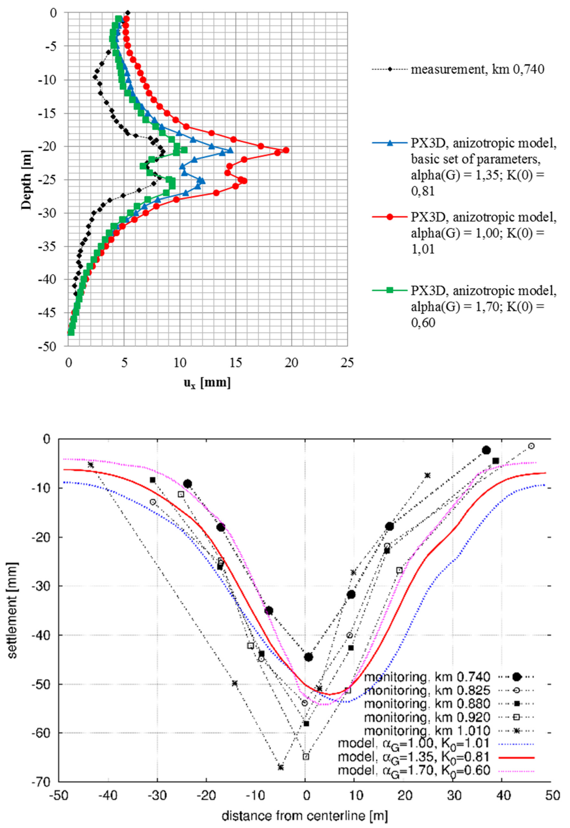

Figure 5. The comparison of measured and computed horizontal displacements in an inclinometer and vertical discplacements at the surface

5 SUMMARY AND CONCLUSION

In the article we dealt with the evaluation of numerical model of NATM Královo Pole tunnel in Brno clay. Finite element analysis used hypoplastic constitutive model for clays with the calibration of degree of anisotropy on the basis of laboratory measurement. As an added value, the composite primary lining was homogenized. It can be stated that resulting values agree well with measurement for the coefficient of earth pressure at rest lower than 1.0. More detailed analysis presented by Rott et al. (2015) then yielded best estimate $K_0 = 0.75$. This value is much lower than estimated from empirical correlations based on apparent oevrconsolidation ratio.

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