The FEM back-analysis of earth pressure coefficient at rest in Brno clay K0 with the homogenization of steel/shotcrete lining

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ABSTRACT: In the paper we estimate the earth pressure coefficient at rest by means of back-analysis of deformation measurements in underground circular unsupported cavity (constructed for the purposes of detailed geotechnical survey) in highly plastic overconsolidated clay. Because of an inability of the adopted FEM code to assume 2 different components of primary lining, the homogenization of steel/sprayed concrete ("shotcrete") lining was considered. The main primary boundary conditions were conservation of static momentum and conservation of the axial and bending stiffness. In addition, the dependence of Young modulus of shotcrete on time was also taken into the account. Our research focuses on Královo Pole tunnel in Brno city, which is one of the key parts of Brno city road ring. The backanalyses of circular cavity indicate K_0 = 0,81. This value was subsequently entered in a numerical 3D model of the whole tunnel and verified by confrontation of measured and calculated displacements.

1 INTRODUCTION

1.1 *Role of K_0 in geotechnics*

The in-situ effective stresses represent an important initial condition for geotechnical analyses, however, especially for oveconsolidated soils, the value of K_0 can estimated only with approximation. The horizontal stress is computed from the vertical stress using the coefficient of earth pressure at rest

$$K_0 = \frac{\sigma_h}{\sigma_v} \tag{1}$$

where σ_h and σ_v are effective horizontal and vertical stresses, respectively. In the case of deep foundations (friction piles), retaining structures and tunnels, K_0 influences the mechanical behaviour dramatically.

1.2 *Estimation of K_0*

Lateral earth pressure coefficient at rest K_0 is very difficult to estimate while it significantly affects the predictions of the behaviour of geotechnical structures. The strategy adopted here was to back-analyze coefficient of earth pressure at rest K_0 in overconsolidated Brno clay using as much realistic numerical model of Královo Pole tunnel as possible. The computed values of ratio of horizontal and vertical displacement u_y/u_z in unsupported adit R2 were paired with initially entered values of K_0 and the relation $K_0 - u_y/u_z$ was constructed. For given value of measured ratio an appropriate value of K_0 was deducted. Because of uncertainties in some material characteristics, a number of parametric studies and verifications using the numerical model of tunnel tube were also carried out.

2 DESCRIPTION OF LOCALITY AND BRNO CLAY

2.1 *The Carpathian fore-trough*

From the stratigraphical point of view, the area is formed by Miocene marine deposits of the Carpathian fore-trough whose depth reaches several km. 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 0.5 to 15 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. Brno Clay ("Tegel") of lower Badenian age reaches the depth of several hundred meters (300 m). The process of sedimentation took place between 16.4 to 14.8 Ma.

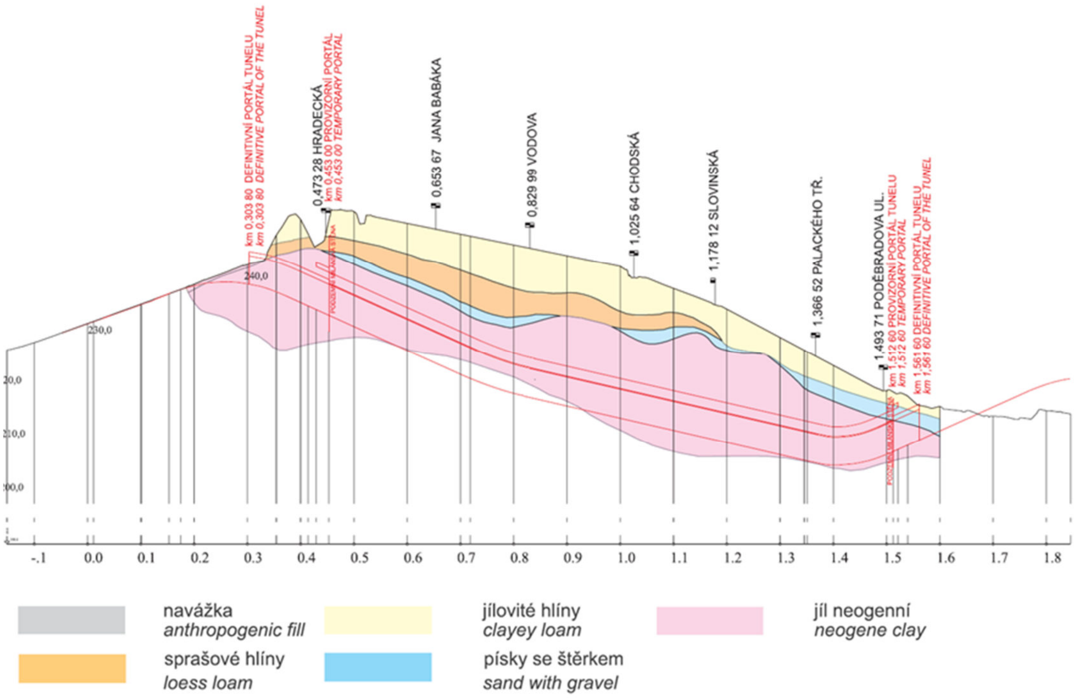

Figure 1. Longitudinal geological cross-section along the tunnels (Pavlík et al. 2004).

2.2 *Brno clay properties*

Sound Tegel has a greenish-grey color, which changes yellow-brown and reddish-brown within the zone of weathering closer to surface. Tegel exhibits stiff to very stiff consistency. The clay is overconsolidated but the height of eroded overburden is not known. The clay is tectonically faulted, the main fault planes are slicken-sided and uneven. The groundwater is mostly bound to quaternary fluvial sediments, and the collectors are typically not continuous. The clay is, however, fully water saturated. In Tegel there is about 50% of clay fraction, w_L is about 75%, I_P about 43%, the soil plots just above the A-line at the plasticity chart and its index of colloid activity I_A is about 0.9.

3 DESCRIPTION OF STRUCTURES AND NUMERICAL MODEL

3.1 *The Královo Pole tunnels (tunnel tubes TT1 and TT2)*

The Královo Pole tunnels (often referred to as Dobrovského 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. Each tube conveys 2-lane road. 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. As proposed in the project the excavation was performed in steps 1 to 6 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. The face of main tunnel tubes was subdivided and the relatively complicated excavation sequences were adopted in order to minimize the surface settlements imposed by the tunnel.

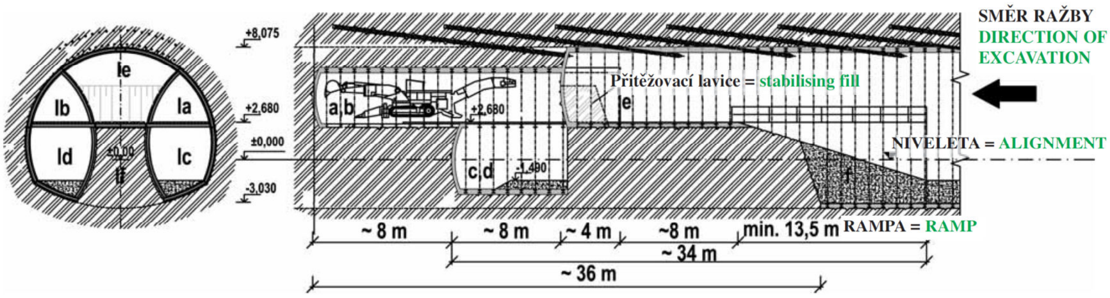

Figure 2. Sketch of the excavation sequence of the tunnel (Horák, 2009).

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 and corresponds to the tunnel chainage 0.6504 – 0.708 km. Results from numerical analysis of TT1 in chainage between km 0.6504 and 0.7068 were compared with monitoring data from inclinometer in km 0.675 and from geodetically measured surface trough in km 0,740. 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/m^3. The overall height of overburden is therefore 17.2 m as visible from overall geometry of the numerical model (figure 3). 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. Primary lining hardening was assumed using the principles of homogenization (see section 5). One excavation step 1.2 m long takes 8 hours. Always the first 1.2 m of excavation remains unsupported. The 6 partial tunnel headings (phases) are assigned as a,b, c, d, e, f as seen from the figure 1. Real excavation order according to scheme is b-a-d-c-e-f. The construction of TT2 was about 2 months in advance.

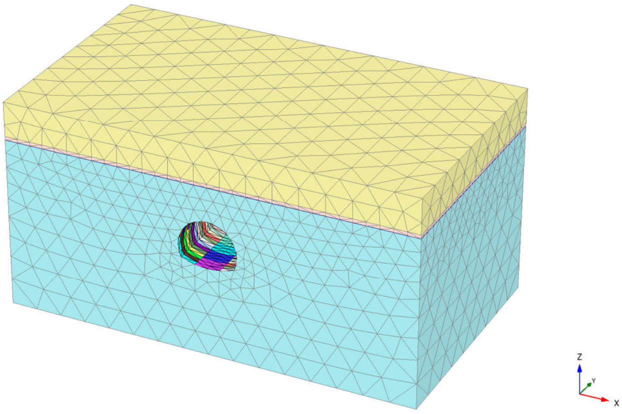

Figure 3. Model geometry of the region in the vicinity of TT1.
Height of clay overburden is ca. 8 m.

3.2 *Exploratory gallery IIB and unsupported adit R2*

Three exploratory galleries with triangular cross section and arched sides have width of 4.75 m and height of 4.5 m. With respect to TT1 and TT2 they are placed in the upper half of the profile. In routing of TT2 two equidistant exploratory galleries IIA and IIB have been excavated, where gallery IIB is one of the key structures for the back-analysis of K_0. The galery IIB was the first one, followed by the second one with the offset of several tens of meters. The length of structures is 831 m. In routing of TT1 only one exploratory gallery IA (the third overall) was excavated in the upper part of profile. The key for K_0 estimation was geotechnical monitoring in unsupported adit R2, which was chosen primarily for sufficient clay overburden and low urbanization. Four unsupported adits R1, R2, R3 and R4 were built for purposes of more detailed local exploration and all belong to exploratory gallery IIB. The unsupported adit R2 (as well as other three) is L-shaped. In each section there is one convergence profile, but only convergence profile in the part perpendicular to the gallery was taken into consideration. The part of adit R2 perpendicular to exploratory gallery IIB of Královo Pole tunnel is 5.38 m long. The diameter of circular cross-section of unsupported adit is 1.90 m. Support is made of mine steel arches in combination with steel wire meshes and acts only as a backup, with the offset of 50 mm from the excavation surface. It means that this is not in contact with soil so there is no interaction and stress redistribution. The height level difference between bottom of the adit R2 and bottom of the exploratory gallery IIB is 0.90 m. The depth of the center of the adit R2 is 22.15 m below the surface, the overburden is 16 m, out of which 6 m is quaternary cover. Figures 4a and 4b are photos of excavation faces in exploratory gallery IIB and unsupported adit R2, respectively. Chosen constitutive material model of clay strata is represented by hypoplasticity with inherent stiffness anisotropy in the very small strain range.

Figure 4a and 4b. View at the excavation face of exploratory gallery IIB (Figure 4a) and unsupported adit R2 (Figure 4b), Pavlík et al. 2004.

PLAXIS 3D Software was used. The chosen constitutive material model of clay strata is hypoplasticity with inherent stiffness anisotropy in the very small strain range. A Mohr-Coulomb model was assigned to the overburden.

Results from numerical analysis of exploratory gallery IIB and unsupported adit R2 were compared with geotechnical monitoring in the corresponding chainage (2.55 m from the junction of both structures) and time. Considering the rate of tunnel face progress (excavation of modelled portion of the exploratory gallery IIB and unsupported adit R2 took about 6 days) the analyses were modelled as undrained. The modeled section of the exploratory gallery IIB was 18 m long (figure 5b). The complete numerical analysis is composed of 28 phases and each of the phases represents progress of excavation by 1.2 m except the portion containing junction. The modeled field has dimensions 55 (width) x 37 (height) x 36 m (length) meters (figure 5a) to assure that the massif nearby the model boundaries remains intact. The quaternary sediments in model are represented by fluvial clay mould layer, then mixture of sand-loess-mould and finally anthropogenic landfill in overall height of strata of 6 m. The overall height of overburden is 22.1 m above the unsupported adit R2 and 20.4 m above the crown of exploratory gallery IIB so the thicknesses of clay overburden is 16.15 m and 14.4 m, respectively.

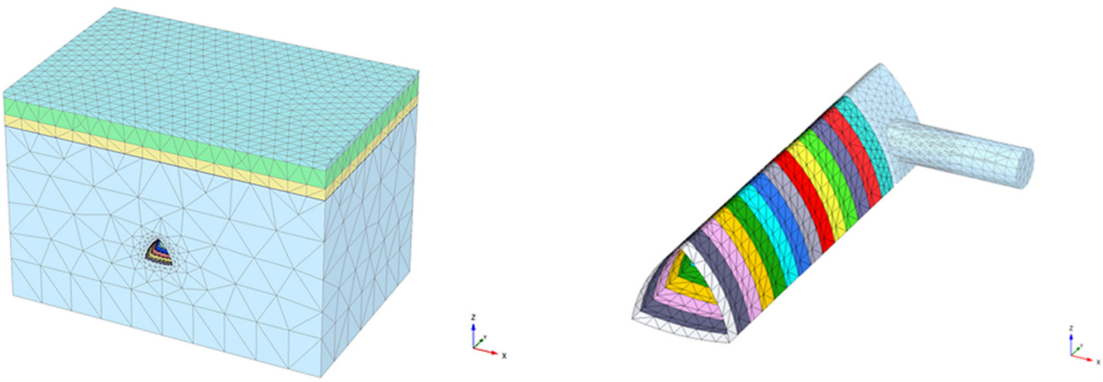

Figure 5a and 5b. Model geometry in the vicinity of exploratory gallery IIB and unsupported adit R2 (Figure 5b) and detail of the geometry of exploratory gallery IIB and junction with adit R2 (Figure 5a).

4 HYPOPLASTIC MODEL OF BRNO CLAY

4.1 *Model description*

The behavior of Brno clay has been described using hypoplastic model for clays incorporating stiffness anisotropy, developed by Mašín (2014). The model is based on the theory of hypoplasticity, which is governed by the following primary equation:

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

where $\mathring{\boldsymbol{\sigma}}$ and $\dot{\boldsymbol{\epsilon}}$ represent the objective (Zaremba-Jaumann) stress rate and the Euler stretching tensor respectively, $\boldsymbol{\mathcal{L}}$ and $\mathbf{N}$ 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), which, in turn, correspond to parameters of the Modified Cam-clay model. The model thus requires parameters ϕ_c equal to 22° (critical state friction angle), N equal to 1.51 (position of the isotropic normal compression line in the space of ln p vs. ln (1+e)), λ^* equal to 0.128 (slope of the isotropic normal compression line in the space of ln p vs. ln (1+e)), κ^* equal to 0.015 (parameter controlling volumetric response in isotropic or oedometric unloading), and ν equal to 0.33 (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

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

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 equal to 0.0001, 0.5, 0.2 and 0.8, respectively.

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 horizontal and vertical shear moduli α_G, defined as

$$\alpha_G = \frac{G_{pp0}}{G_{tp0}} \tag{4}$$

Two additional parameters characterize the dependencies of Poisson ratios and Young moduli:

$$\alpha_E = \alpha_G^{(1/x_{GE})} \tag{5}$$

$$\alpha_\nu = \alpha_G^{(1/x_{G\nu})} \tag{6}$$

Based on evaluation of an extensive experimental database, Mašín & Rott (2014) suggested default values of $x_{GE} = 0.8$ and $x_{G\nu} = 1.0$. In our analyses, these default values have been used primarily, however we also investigated sensitivity of the results on the selected parameter values.

4.2 *Model calibration*

Ratio of shear moduli α_G was measured by means of bender elements. For the purposes of the model, we considered the following value of α_G:

$$\alpha_G = 1.35 \tag{7}$$

During the calculation, the empirically estimated coefficients x_{GE} and x_{Gv} were used, however, the parametric study was also carried out. Model was further modified to predict the non-linear relation between shear modulus and mean effective stress in the small strain range:

$$G_{vh0} = p_r A \left(\frac{p}{p_r}\right)^n \tag{8}$$

where p_r is the reference stress equal to 1 kPa, p is mean effective stress, A, n are parameters equal to 5300 and 0.5, respectively. These parameters are the result of interpolation of experimental data (Figure 6). The related bender element measurements of vertical small strain shear modulus were carried out by Svoboda et al. (2010).

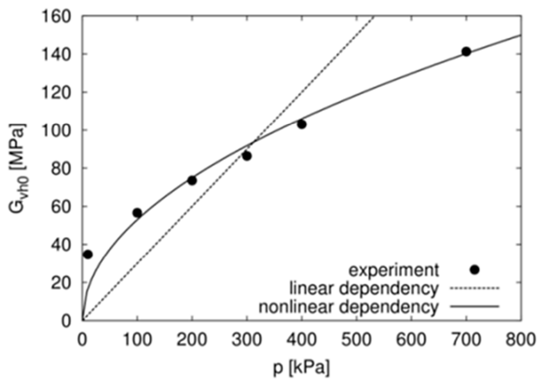

Figure 6. Calibration of vertical shear modulus in the very small strain range (Svoboda et al. 2010).

5 HOMOGENIZATION OF PRIMARY LINING

5.1 *Introduction*

The main tunnel tubes and exploratory galleries are supported by primary lining. This brings another uncertainty into the numerical model, since we do not exactly know time proportions and time-dependent material characteristics. To increase the model accuracy and because of inability of the adopted FEM numerical code to assume two different components of the primary lining, the homogenization of steel/sprayed concrete (shotcrete, further denoted as 'SC') 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 given constant value of the length of excavation step. In addition, the dependence of Young modulus of SC on time is taken into account. Královo Pole tunnels and exploratory galleries have primary lining composed of massive steel profile and SC applied in two layers. Then, in fact, we have three-material composite – older SC, younger SC and mine steel profile (exploratory gallery IIB) and "HEBREX" profile respectively (TT1 and TT2).

5.2 *Description*

In the numerical model, linear elastic material with time dependent SC stiffness was used, with parameters calculated using an empirical relationship where E_f =18 GPa is the final Young modulus, α is a parameter equal to 0.14 and t_r = 1 is the reference time.

$$E_B = E_f\left(1 - e^{-\frac{\alpha t}{t_r}}\right) \tag{9}$$

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

$$E_m(t_1,t_2) = \sqrt{\frac{E_f^2\left(1-e^{-\frac{\alpha t_1}{t_r}}\right)^2 A_{NP}^3(t_1,t_2)}{12 I_{NP}(t_1,t_2) b_z^2}} \tag{10}$$

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

$$h_m(t_1,t_2) = \frac{2\sqrt{3 I_{NP}}(t_1,t_2)}{\sqrt{A_{NP}(t_1,t_2)}} \tag{11}$$

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 (12) for conversion ratio $n(t_1)$:

$$n(t_1) = \frac{E_O}{E_f\left(1-e^{-\frac{\alpha t_1}{t_r}}\right)} > 1 \tag{12}$$

and, analogically, for the conversion of younger SC to older SC the following equation (13) is valid:

$$m(t_1,t_2) = \frac{E_f\left(1-e^{-\frac{\alpha t_2}{t_r}}\right)}{E_f\left(1-e^{-\frac{\alpha t_1}{t_r}}\right)} = \frac{1-e^{-\frac{\alpha t_2}{t_r}}}{1-e^{-\frac{\alpha t_1}{t_r}}} < 1 \tag{13}$$

These equations are valid with the assumption of the following main simplifications:

a) The overall stiffness (bending, axial and shear) does not involve SC cover of steel profile.
b) We assume full static interaction of 2 layers of SC.

For the primary lining of exploratory gallery IIB (consisting of 1 rolled mine steel beam, 2 layers of SC, overall static thickness 100 mm, E_f =18 GPa, time difference $t_1 - t_2$ = 1day, α = 0.14 per one excavation step with the length b_z = 1.2 m) the resulting values for various time t_1 are summarized in following Table 1:

Table 1. Resulting stiffness characteristics of homogenized and modified primary lining.

t_1	$E_m(t_1, t_2)$	$h_m(t_1, t_2)$	$E_m I_m (t_1, t_2)$
days	MPa	m	MNm^2
1	5153	0.123	0.796
2	7114	0.117	0.941
5	11733	0.109	1.268
10	16277	0.105	1.581
15	18551	0.104	1.736
20	19683	0.103	1.813
25	20318	0.103	1.851

Figures 7a and 7b show the relation $t_1 - h_m(t_1, t_2)$ for primary lining of the exploratory gallery IIB and main tunnel tubes, respectively. Due to coupled effect of homogenization (conversion of Steel/SC and SC/SC) and modification (to satisfy the condition of constant width of tunnel step) the progress of the static thickness $h_m(t_1, t_2)$ seems to be surprising. Since the steel profile is in contact with the massif, the initial value of the above-mentioned static characteristics is non-zero.

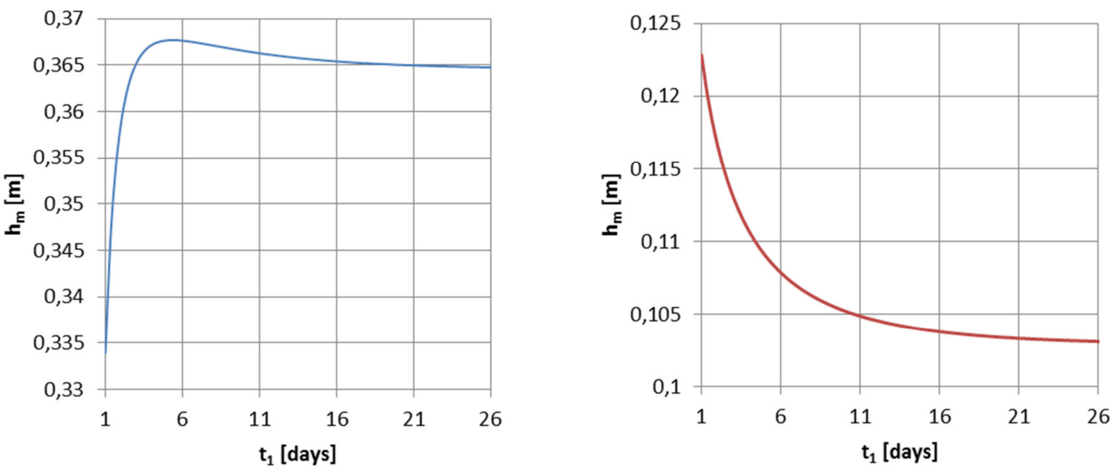

Figure 7a and 7b. Progress of $h_m(t_1, t_2)$ in time for primary lining of TT1 (Figure 7a) and progress of $h_m(t_1, t_2)$ in time for primary lining of exploratory gallery IIB (Figure 7b).

6 RESULTS OF BACK-ANALYSIS OF K_0

The first step in numerical modeling was evaluation of the likely value of K_0 on the basis of back-analysis of exploratory gallery IIB and unsupported adit R2 according to the key convergence ratio $P = u_y/u_z = 1{,}25$. In the next phase the influence of each of the parameters α_G, x_{GE}, x_{Gv} was studied. The second step was to verify K_0 by means of numerical analysis of the complete Královo Pole tunnel tube TT1.

The dependence of convergence ratio on K_0 is obvious from Figure 8a and is very significant. The measured value $u_y/u_z = 1{,}25$ is exactly matched for $K_0 = 0{,}81$, which is considerably lower than value obtained by Mašín & Novák (2013) in the analyses with the hypoplastic model without anisotropy and without presence of exploratory gallery IIB.

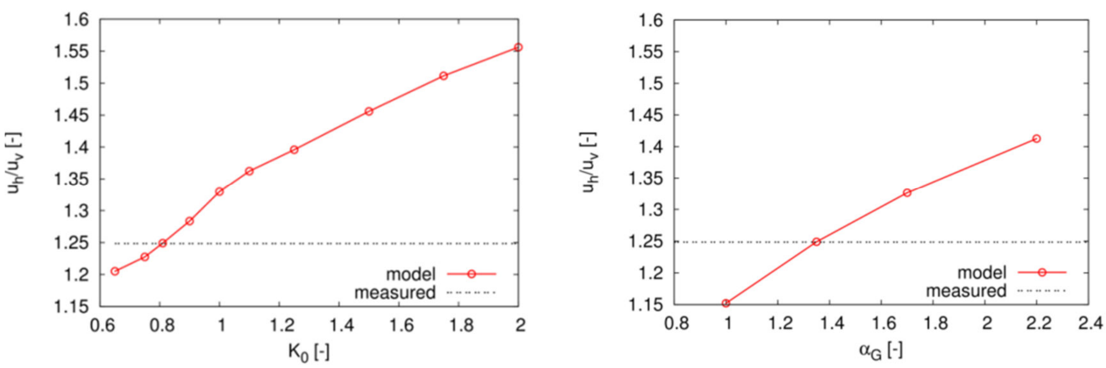

Figure 8a and 8b. Resulting relation between K_0 and u_h/u_v (u_y/u_z in model, Figure 8a) and relation between α_G and u_h/u_v (u_y/u_z in model, Figure 8b).

It is apparent that the predicted ratio u_y/u_z is significantly affected by α_G as seen from Figure 8b. Fortunately, this value is easiest to be found out in laboratory. Increasing of α_G (increasing of degree of anisotropy) leads to increase of ratio u_y/u_z. Apart from the shear stiffness, anisotropy affects also the stress path and therefore change of effective stress around excavation under undrained conditions, which leads to the observed dependency of K_0 and α_G.

Since we have relations $u_y/u_z - K_0$ and $u_y/u_z - \alpha_G$, it is necessary to carry out an additional numerical calculation to obtain the dependence $K_0 - \alpha_G$. Therefore, for given boundary values of α_G =1.00 and α_G =1.70 (this is assumed as a maximum value for Brno clay) the corresponding values of K_0 are obtained. The search for K_0 corresponding to α_G =1.00 and α_G =1.70 and condition u_y/u_z = 1.25 was done by interpolation between 2 values of u_y/u_z. The resulting values are K_0 = 1.01 and K_0 = 0.60, respectively.

7 VERIFICATION OF K_0 USING SIMULATIONS OF THE COMPLETE TUNNEL

The real construction of TT2 was about 2 months in advance so the surface trough of TT1 is believed to be affected. Nevertheless, the affection is relatively small due to the distance between tunnel tubes axes. One of the main advantages of selected portion of TT1 is the absence of exploratory gallery IA. Numerical model does not fully respect the real time distances since they vary from date to date. However, the difference is relatively small, so the result is believed to be the same as through entering the mentioned average value. First of all, an analysis with α_G =1.35 and K_0 = 0.81 was performed, then an alternative pairs of α_G and K_0 consistent with the condition u_y/u_z = 1.25 (found on the basis of circular adit simulations) were entered. The purpose is to demonstrate that the "default" back-analysed combination of α_G=1,35 and K_0 = 0,81 is the most probable. The results of all analysed cases are summarized in following Table 2 and shown in Figures 9a and 9b.

Table 2. Resulting horizontal deformations from the verification analysis.

Case	α_G	K_0	$u_{x,max}$
	-	-	(mm)
1	1.35	0,81	14,5
2	1.00	1,01	19,5
3	1.70	0,60	10,5

The equidistant position of graphs is the verification that geotechnical structure is seated in correct depth. The surface settlement trough depth (Figure 9b) is almost unaffected and vary between 50 – 54 mm. The crucial reason is probably the influence of very stiff primary lining on the results of TT1 simulations which does not allow for significant bending due to the change of loading. All values are acceptable and realistic in comparison with the monitoring results in chainage 0.740, which in maximum yields 44 mm. We can state that variation of α_G and K_0 has only negligible influence on the surface trough. The surface trough for the combination α_G = 1.70; $K_0 = 0{,}60$ is narrower than for other cases which is typical for anisotropic stiffness. Here the effect is further strengthened by the low value of K_0. Surface settlement trough is slightly non-symmetric. The asymmetry is caused partly by the tunnel headings offset and also by the fact that the tunnel tube TT2 was bored 2 months later. Even though the distance between the tunnel tubes is around 70 m, it is believed that terrain surface troughs of both tubes certainly influence each other. In contrast with the surface settlement trough, horizontal displacements in the tunnel vicinity vary relatively significantly (Figure 9a). The reason comes from the assumption of shear moduli of anisotropic clay. Theoretical shear modulus in the horizontal direction at zero strain is calculated as $G_{hh0} = G_{tp0}\alpha_G$ which means the change of stiffness in the horizontal direction only. For the "default" back-analyzed combination α_G = 1.35 and K_0 = 0.81 the maximum obtained value is 14.5 mm. The monitoring data show 8.5 mm. The other results 19.5 mm and 10.5 mm belong to combination α_G = 1.00 and K_0 = 1.01 and α_G = 1.70 and K_0 = 0.60, respectively. It is now possible to say that the model was working properly and the obtained numbers are logical because the increase of α_G leads to a decrease of displacements and contrary, the increase of K_0 on the contrary leads to the increase of displacements.

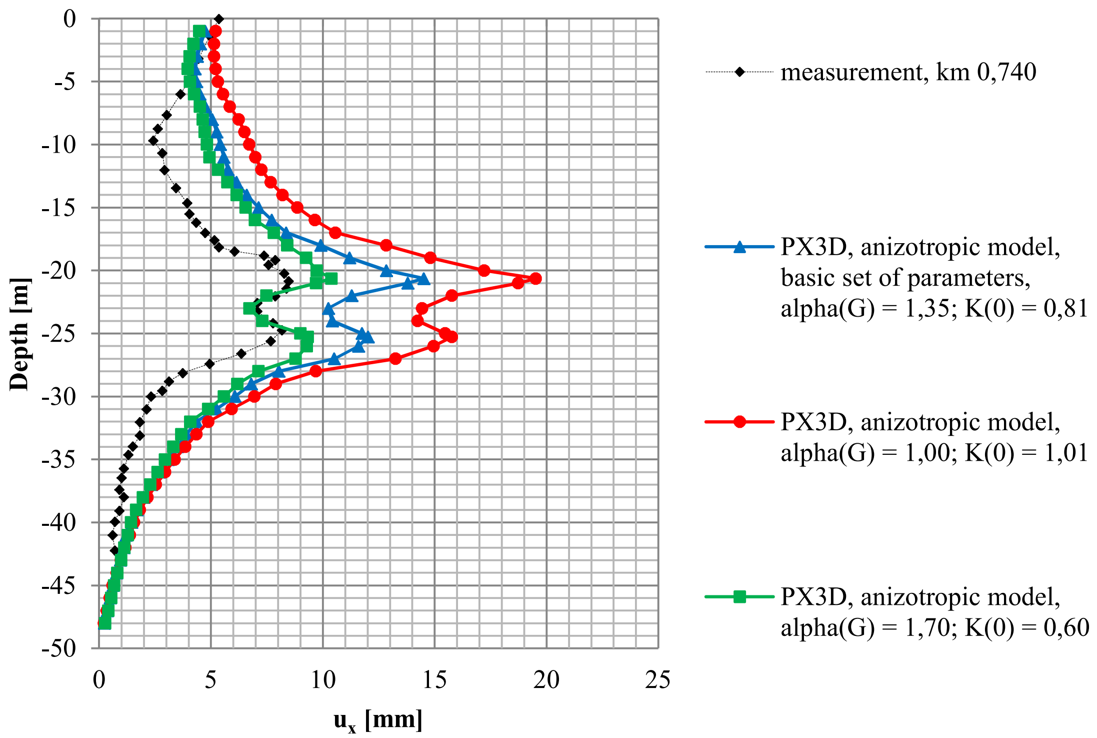

Figure 9a. Modelled horizontal displacements for all 3 cases.

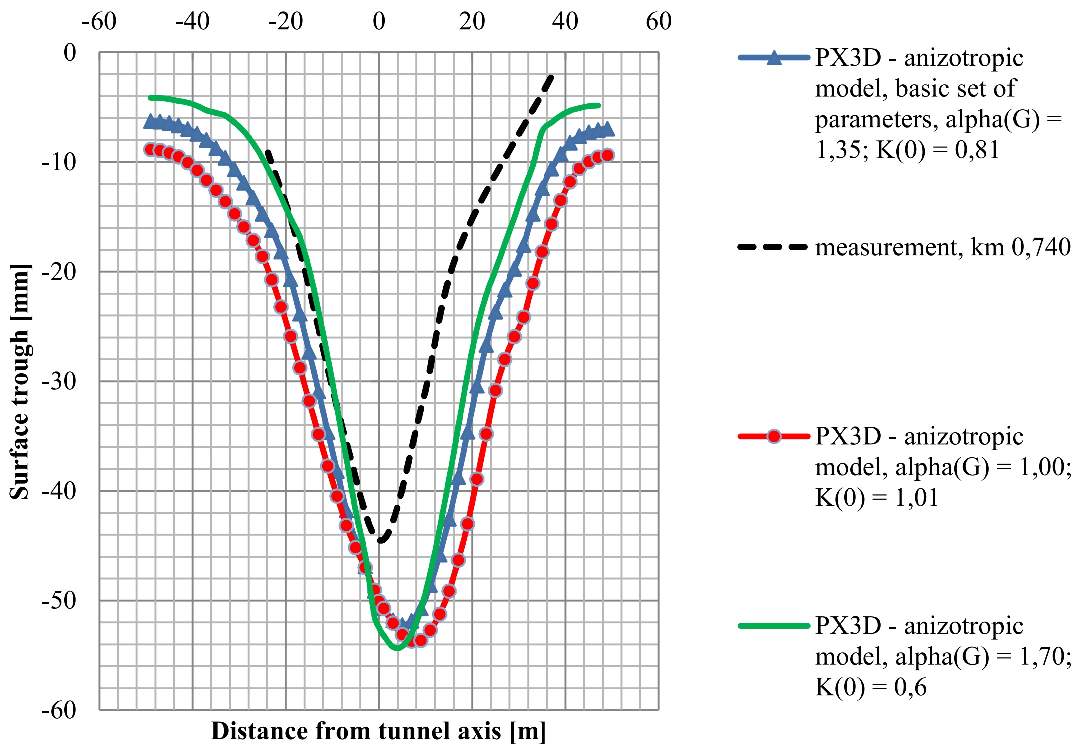

Figure 9b. Modelled surface settlement trough for all 3 cases.

8 SUMMARY AND CONCLUSIONS

The paper deals with an estimation of lateral earth pressure at rest in over-consolidated Brno clay. An indirect method of numerical simulation of boring process of Královo Pole tunnel was used, where the obtained values are fitted to geotechnical monitoring data through parametric study. We simulated Brno clay using a hypoplastic model with implemented inherent small strain stiffness anisotropy. The homogenization of primary lining was adopted to represent SC lining. The obtained values agree well with measurements. It was also proved that the variability of the degree of anisotropy of shear moduli α_G affects the results significantly, whereas the effect of parameters x_{GE} and $x_{G\nu}$ is small. The resulting lateral earth pressure coefficient at rest was $K_0 = 0.81$. Note that this value is valid for circular adit depth only.

9 ACKNOWLEDGEMENT

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