Comparison of displacement field predicted by 2D and 3D finite element modelling of shallow NATM tunnels in clays

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Abstract

The 2D load-reduction method for simulating NATM tunnels using plane strain finite elements was in the paper evaluated by comparison with fully 3D simulations. Three real shallow tunnels in urban environment in different stiff clays were simulated. The soil behaviour was described by an advanced non-linear soil constitutive model based on the hypoplasticity theory. Time-dependent behaviour of shotcrete lining was considered in 3D simulations, whereas constant final stiffness was used in the plane strain analyses. The 2D analyses were thus controlled by a single parameter that accounts for 3D effects. It was shown that for an optimum value of this parameter the displacement field predicted by the 2D method agrees well with the 3D simulations. In some cases only, a discrepancy was observed in a close vicinity of the tunnel. The controlling parameter was, however, found to be dependent on the problem simulated (for the same material) and also on the material properties (for the same tunneling problem). Considering the material properties, the very small strain shear modulus was found to be more influential than the large strain shear modulus. The initial K_0 stress state did not influence the controlling parameter substantially.

KeyWords: finite element analysis; load reduction method; constitutive models; clays

1 Introduction

3D numerical analysis becomes an increasingly affordable tool for predicting deformations and stress redistribution induced by tunnelling. As tunnel excavation is clearly a three dimensional problem, considering the third dimension should intuitively lead to more accurate predictions. However, simplified procedures that allow us to consider 3D effects within a simplified 2D plane strain analysis are still popular in geotechnical design [32, 57, 34].

One of the reasons may be the fact that the set up of the full 3D model is inevitably, regardless the computer power, a time consuming process and it might not be feasible at preliminary design stages or for less demanding tunneling problems. Second, a perhaps surprising reason, is the fact that the available research studies are not conclusive in demonstration that the accuracy of predictions achievable in practical applications of full 3D methods is higher than those of properly used 2D analyses with indirect incorporation of 3D effects (see [14, 29, 55]). This is because the 3D simulations require description of such aspects as the excavation sequence, lining installation procedure, or time-dependent behavior of shotcrete, which may be difficult to incorporate in a numerical model accurately. On the other hand, 2D methods typically require specification of just a few or only one parameter (denoted in the following as λ^d), which integrates the influence of all of the aforementioned factors that need to be considered in 3D analyses. Obviously, it is not possible to find a direct empirical relationship between λ^d and all the modelling aspects that enter the 3D simulations. Thus, in practical applications, the parameter λ^d is often merely estimated. Either based on experience with previous simulations of tunnels in similar environment, if one needs to perform forward predictions of tunnel behaviour, or it is adjusted on the basis of monitoring results, if the observational method [44] is applied. This allows, in a simple way, to overcome the difficulties of accurate modelling of all the relevant factors entering the 3D simulations.

At this point, however, it must be emphasized that the appealing simplicity of 2D methods may have dangerous consequences if it is used improperly. Particularly the second approach, i.e. calibrating λ^d based on monitoring results, is popular as it is often possible to tune the model using a single empirical parameter such that some monitored aspect is correctly reproduced. Disadvantage of this approach is that it hides eventual inaccuracies in description of the mechanical behaviour of the soil massif. It has been shown in a number of studies that selection of constitutive model and calibration procedure of its parameters have a substantial effect on predictions of tunneling problems (see, for example, [36, 11, 18, 1, 14]). As a consequence, tuning λ^d to reproduce correctly one aspect of the observed behaviour (e.g. prediction of the magnitude of surface settlements) will indeed lead to inaccuracies in predictions of other aspects (overall displacement field or lining internal forces, as examples), if the soil massif behaviour is not reproduced correctly (see, e.g., [11]). Also, this approach does not provide any information on the suitability of the 2D method itself to reproduce 3D effects.

Regardless advantages and shortcomings of the 2D methods from the point of view of practical applications and as a matter of fact, before each from a number of existing 2D methods accounting for 3D effects is applied in practice, its predictive capabilities should be properly evaluated. As explained, comparison with monitoring results does not provide any information on the suitability of the 2D method, due to the uncertainties in the description of the mechanical behaviour of the ground. A way to proper evaluation of the 2D method thus lies in comparison of its predictions with predictions by equivalent fully 3D analyses. Unfortunately, such studies are not common in the technical literature dealing with numerical simulations of tunneling problems (several available studies are summarised in Sec. 2).

It must also be pointed out that if the aim of an analysis is the evaluation of the tunnel face stability, appropriate 3D-modelling is required and irreplaceable [15, 10, 33, 58, 30, 40]. Evaluation of the tunnel face stability is, however, outside the scope of the present paper.

2 Methods accounting for 3D effects within plane strain simulation procedure and their validation

A number of 2D methods that can be used to account for 3D effects within finite element analysis framework have been summarised by Leca et al. [32] and Karakus [26].

Probably the most popular method is the so-called load reduction method [52, 39]. In fact, this method is a finite-element utilisation of the analytical convergence-confinement method introduced by Panet and Guenot [43] with primary application to tunnel lining design. Panet and Guenot [43] demonstrated that the 3D ground response to tunneling could be analysed with a plane strain approach, provided a fictitious pressure σ_r^f was introduced on the tunnel surface in the 2D model (Fig. 1). This pressure could be derived from the initial stress in the ground σ_r^0 from

$$\sigma_r^f = (1 - \lambda)\sigma_0^f \tag{1}$$

where λ is the stress release coefficient. The ground reaction curve, also termed ”convergence curve”, reflects the relationship between the amount of convergence, C, and the ground pressure σ_r^f. The lining is nstalled at some distance d from the tunnel face, in which some tunnel convergence C_d has already taken place. Additional convergence tends to increase lining loads, which is reflected by the ”confinement curve”. The state of equilibrium E is reached when the lining forces equal to the ground pressure. If the tunneling is considered as time-independent process, i.e. if such aspects as pore pressure dissipation, creep and time-dependency of lining properties are neglected, the load reduction method requires specification of only one parameter - the value of the stress release coefficient corresponding to a moment of lining installation, denoted as λ_d.

[Figure 1 about here.]

In principle similar to the load reduction method is the volume loss method by Potts and Zdravkovic [47] and progressive softening method by Swoboda [54]. Specific approaches were also developed for shield driven tunnels, such as the gap method (Rowe et al., [51]). In addition to these ”classical” methods, several more advanced methods have been developed and evaluated more recently. Among the most notable examples belong the hypothetical modulus of elasticity (HME) soft lining method (Powel et al., [48]), applied for example by Karakus and Fowell [27, 28]. An extension of the load reduction method accounting for the time evolution of both mechanical unloading and drainage conditions induced by advancing excavation face was proposed by Callari [6]. In this method, time is explicitly incorporated into the evolution equation for the load reduction factor λ. Pore pressure conditions are treated separately, allowing to represent pore pressure dissipation in permeable saturated ground. Although Callari [6] incorporated time explicitly into the method, time-dependent behaviour of shotcrete lining was not considered (only unsupported tunnel was analysed). Bernaud and Rousset [3] pointed out that the lining installation modifies the tunnel wall convergence even in the yet unsupported portion in advance of the face -the higher the stiffness of the lining, the lower the tunnel convergence. Therefore, if the conventional load reduction method is calibrated to reach specified final deformations without considering low stiffness of the shotcrete lining just after its installation, it implies an overestimation of the wall displacement at the time of lining installation, which in turn leads to an underestimation of the final load on the lining. Based on axisymmetric analyses with a simple constitutive law with Tresca criterion, Bernaud and Rousset [3] introduced a method accounting for a lining stiffness, named ”new implicit method”. Oreste [42] and Graziani et al. [19] further developed this method to také into account time-dependent shotcrete lining behaviour. Both the studies considered axisymmetric simulations of deep tunnels. Gonzales-Nicieza et al. [17] developed a procedure to estimate radial displacements and consequently support pressures of non-circular tunnels at different depths based on axisymmetric load reduction method simulations.

As already indicated, only few authors studied the suitability of the 2D methods based on comparison with fully 3D simulations. Callari and Cassini [7] evaluated the extended load reduction method method by Callari [6] using 3D results from Callari and Cassini [8]. The ground was described by a basic Drucker-Prager elasto-plastic model, unsupported tunnel was considered only. Fully coupled consolidation analyses [4] were performed. The authors obtained a good agreement between settlement troughs predicted by the 2D and 3D methods, the only appreciable difference was in simulation of a tunnel with a slow advance rate in a saturated soil, where the 2D analysis gave approximately 10% higher settlements. The accuracy of predictions of horizontal displacements with depth and overall displacement field around the tunnel has not been presented.

Karakus and Fowell [29] compared 3D and 2D analyses of the Heathrow express trial tunnel [9]. 2D analyses were performed using the HME soft lining method. The analyses were undrained with material properties characterised by a Mohr-Coulomb constitutive model. Both the analyses were tuned to obtain the monitored surface settlement trough. In 2D, proper HME values were specified. The authors noted that the 3D analyses underestimated significantly the surface settlement if realistic excavation sequence was adopted. The monitored surface settlements were then predicted with unrealistic size of an excavation step. Both the 2D and 3D analyses finally produced the same size and a shape of the surface settlement trough. The overall displacement field around the tunnel was not studied.

A detailed evaluation of the load reduction method based on 3D simulations has been presented by Doležalová [11]. A NATM tunnel in a partly weathered jointed shale was simulated under drained conditions. Two constitutive models were used in the analyses – Mohr-Coulomb model and path dependent non-linear elastic perfectly plastic model [12]. It was observed that different values of the stress release factor λ_d are required to match surface settlements and crown deflections respectively, which supported conclusions by Vogt et al. [56]. Moreover, the value of the stress release factor was found to depend both on the constitutive model chosen and on the value of the coefficient of earth pressure at rest K_0.

The influence of different tunneling aspects (such as ground stiffness and strength, K_0 and cutting length) was investigated by Möller and Vermeer [39]. They used linear elastic and elastoplastic analyses with Mohr-Coulomb model. For the cases studied, the λ_d factor depended substantially on the ground stiffness and strength and on the cutting length. The influence of $K_0$was less pronounced. Möller and Vermeer [39] also noticed that significantly different values of λ_d were required to fit correctly ground displacements, lining normal forces and lining bending moments.

This review shows that proper evaluation of 2D methods is not common throughout scientific literature. To check whether 2D methods can fully account for 3D effects, the accuracy of predictions of the overall displacement field, not only surface settlement trough, must be studied. In thepresent paper, we focus on the following questions:

- whether λ_d depends on the problem geometry, i.e. whether the same λ_d can be used for predictions of different tunnels in the same material.

- to what extent λ_d depends on material properties, i.e. whether the same λ_d can be used for predictions of a tunnel advancing through different geological materials.

Unlike in the previous works, the problem has been studied using an advanced non-linear constitutive model for the ground behaviour. The load-reduction method has been be used throughout this study.

3 3D simulations of case studies analysed

3D finite element models of three case histories will be used as a basis for the evaluation of the 2D load reduction method. All of the cases represent shallow NATM tunnels excavated in stiff clays in urban environment. Results of the 3D analyses were thoroughly checked with respect to monitoring data, to demonstrate realistic representation of the three-dimensional effects.

Soil properties were described by means of an advanced nonlinear constitutive model for finegrained soils, a hypoplastic constitutive model for clays by Mašín [35] enhanced by the intergranular strain concept by Niemunis and Herle [41]. This model has been shown to represent accurately the soil behaviour from the very small strain range to large strain range, including high quasielastic stiffness in the very small strain range and its non-linear decrease with increasing strain level [37, 21, 22]. A finite element implementation of this model is freely available on the web [20]. The basic version of the hypoplastic model requires five parameters, whose physical interpretation corresponds to the parameters of the Modified Cam-Clay model [50]: N, λ^*, κ^*, φ_c and r. The parameters N and λ^* define the position and the slope of the isotropic normal compression line (NCL) within the ln p vs. ln(1 + e) representation, where p is the effective mean stress and e is the void ratio. Parameter κ^* controls the slope of the isotropic unloading line. Parameter φ_c is the critical state friction angle. The parameter r controls the soil shear stiffness.

The basic hypoplastic model predicts the soil behaviour in the medium to large strain range. In order to predict the high initial (very-small strain) stiffness, its decrease with straining, and the effects of recent stress (deformation) history [2], the model needs to be enhanced by the intergranular strain concept [41]. The concept requires additional five parameters (m_R, m_T, R, β_r and χ). The parameters m_R and m_T influence the initial (very-small-strain) shear modulus through the equation [35].

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

The parameter R controls the size of the elastic range and the remaining parameters β_r and χ control the ate of the stiffness degradation.

Finite element simulations were performed using the software Tochnog Professional [49]. In all cases, undrained analyses were performed. The water bulk modulus K_w was reduced with respect to its true value to account for consolidation effects. Realistic excavation sequence as applied in the field was simulated. The lining behaviour was described by a linear elastic model with
time-dependent Young modulus following an exponential expression from [46, 42]. Continuum elements, rather than shell elements, were used to simulate tunnel lining. For a detailed description of the analysis procedures see [36].

3.1 Heathrow express trial tunnel

The Heathrow express trial tunnel [9], a NATM tunnel built to test the effectiveness of the shotcrete lining method in stiff clay, has since become a classical example for evaluation of different numerical tools [55, 28]. In this work, 3D analyses described in detail by Mašín [36] are used as a basis for evaluation of the load reduction method. The tunnel had been excavated in London Clay. London Clay is typically firm to stiff, becoming hard with depth, overconsolidated, inorganic, blue to gray Eocene marine silty clay. The plasticity of London Clay is generally high or very high. The hypoplastic constitutive model was calibrated using high quality experimental data on London Clay by Gasparre [16]. All the simulations were performed with a constitutive model calibrated solely on the basis of laboratory experiments, without tuning material parameters to obtain monitored deformations. High K_0 values varying with depth, as measured in situ by Hight et al. [23], were considered. The finite element mesh and the modelled geometry is shown in Fig. 2.

[Figure 2 about here.]

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. 3 ([28, 5]).

[Figure 3 about here.]

Mašín [36] performed a parametric study aimed at clarification of the influence of material parameters on the predicted results. Figure 4 shows shear stiffness degradation curves for undrained shear triaxial tests on natural samples of London Clay simulated using the hypoplastic model with different values of material parameters. As is clear from Fig. 4, the parameter m_R controls the initial stiffness in the very small strain range, while the parameter r controls the large-strain stiffness (constant initial stiffness was imposed in the analyses while varying parameter r). Values mR = 9 and r = 0,5 represent the best the experimental data.

[Figure 4 about here.]

Surface settlement troughs predicted by the 3D model are shown in Fig. 5. With parameters calibrated solely on the basis of laboratory experiments, the model is capable of reproducing the settlement magnitude, while it slightly overestimates the settlement trough width. Although the influence of the large strain stiffness (parameter r) appears to be insignificant in Fig. 4b, it has a substantial effect on the predicted settlement magnitude (Fig. 5b).

[Figure 5 about here.]

3.2 Dobrovskeho exploratory adit

The second case study analysed is an exploratory adit of the Dobrovskeho tunnels, which were excavated in Brno, Czech Republic. These tunnels form the northern part of the large city ring road. The tunnels consist of two tunnel tubes with lengths 1.2 km with height of about 12 m, a section width of about 14 m and a full face area over 140 m^2. Both the tunnels are led parallel at a distance of 70 m and are being excavated by the NATM with the tunnel face subdivided into 6 parts. The overburden ranges between 6 and 21 m. For exploration purposes, three adits were excavated. The exploratory adits had approximately triangular cross sections with side length 5 m and were situated in the tunnel top headings (Fig. 6). The subsoil in which the tunnels are excavated consists of Brno Clay. Brno Clay is Tertiary (Miocene) limy, calcareous silty stiff clay. In the upper part the clay is tinted rusty-brown due to limonitic solutions penetrating through discontinuity systems, the fresh clay is of green-grey color. The natural cover of the Brno Clay deposits is represented by Quaternary loess loams, clayey loams and sandy gravel.

[Figure 6 about here.]

The full 3D numerical model of the exploratory adit has been developed by Svoboda et al. [53]. The parameters of the hypoplastic model for clays were calibrated on the basis of quality laboratory experiments that included measurements of small strain stiffness characteristics using local LVDT strain transducers and bender elements. The finite element mesh and the model geometry are shown in Fig. 7. The adit was excavated full-face, with the unsupported span of 1.2 m.

[Figure 7 about here.]

As no measurements of the coefficient of earth pressure at rest K_0 have been performed on the site, the simulations were performed with two different extreme values of K_0. One considers the apparent overconsolidation of the soil deposit caused by mechanical unloading. K_0 is then calculated using an approach proposed by Mayne and Kulhawy [38] leading to $K_0 = 1.25$. The second approach assumes that overconsolidation is apparent and it is caused by creep phenomena. The K_0 value then corresponds to a normally consolidated soil and it can be obtained from Jáky [25] formula, leading to $K_0 = 0.66$. Surface settlement troughs predicted by the hypoplastic model for the two K_0 values are shown in Fig. 8a. The analysis with $K_0 = 0.66$ simulates the monitored behaviour better, but in general the K_0 value does not have a substantial effect on the settlement trough. Again, realistic predictions were obtained with the constitutive model calibrated solely on the basis of laboratory experimental data. Additional analyses were performed with variable parameters *r* (large strain stiffness) and m_R (small strain stiffness) for $K_0 = 1.25$. The influence of these characteristics on the small strain stiffness is similar to the one from Fig. 4. Fig. 8b shows that both small and large strain stiffness influence the settlement trough predictions.

[Figure 8 about here.]

3.3 Dobrovskeho tunnel

In addition to the simulations of the exploratory adit of Dobrovskeho tunnel, a full 3D model of the whole tunnel has been created. The model considered the rather complex excavation sequence followed at the site. The geometry and finite element mesh of the 3D model is shown in Fig. 9, the excavation sequence is detailed in Fig. 10, and Fig. 11 shows the excavation sequence as adopted in the finite element model. The excavation was performed in steps 1-6 (Fig. 11) 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 9 about here.]

[Figure 10 about here.]

[Figure 11 about here.]

Two cases were in the analyses considered. One with original model parameters calibrated on the basis of laboratory experiments, the second with parameters optimised based on monitoring results from exploratory adit (see [53] for details). Only simulations with the original parameter set are considered in this paper, so that direct comparison with simulations of the exploratory adit is possible. The simulations represent ”class A” predictions [31] of deformations due to the tunnel, as the tunnel had not been built by the time the authors were performing the simulations. The predicted surface settlement troughs for the two K_0 states are in Fig. 12. Fig. 12 also include monitoring results obtained after the tunnel excavation.

[Figure 12 about here.]

4 2D analyses by the load reduction method

To study the applicability of the load reduction method, 2D equivalents of all the 3D models presented have been prepared. A basic version of the load reduction method was adopted, i.e. time dependency of the lining stiffness, which has been considered in the 3D analyses, was not modelled. In 2D, truss-beam elements were used to represent the lining, whereas in 3D, the lining was modelled using continuum elements.

The load reduction method controlling parameter λ_d was calibrated to ensure that the 2D and 3D analyses predicted as closely as possible the *surface settlement troughs* (the overall displacement fields were studied subsequently). In order to prevent subjectivity of the λ_d determination, it was calibrated using a software tool specifically devised for optimisation and inverse modelling UCODE [45]. For other applications of this software in geotechnical engineering optimisation see [13]. The following procedure was applied. The vertical surface displacements computed by the load reduction method in different distances from the tunnel axis (approx. 20 locations) were used to assemble a ”simulation vector” y_0, whereas displacements obtained by the 3D analyses formed an "observation vector" y. The difference was quantified by means of a weighted least-squares objective function *S(b)* expressed as

$$S(b) = [y - y'(b)]^T w [y - y'(b)] \tag{3}$$

where *b* is a vector containing values of parameters to be estimated (in our case a single parameter λ_d) and is a 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 [13]. In the present case, however, each of the four observations is assigned the same weight equal to unity. Minimisation of the objective function *S(b)* was accomplished by UCODE with the modified Gauss-Newton method.

5 Summary of the 2D analyses and discussion of results

Altogether 12 simulations have been performed and evaluated. They correspond to the 3D simulations described in Sec. 3. The results are summarised in Tab. 1, which gives values of theparameter λ_d corresponding to the minimum value of the objective function *S(b)*. In addition, relation between very small strain and large strain shear moduli used in simulations (G_0(simul.) and G_{ls}(simul.) respectively) and their original values calibrated using laboratory experiments (G_0 and G_{ls} respectively) is indicated. The able also indicates the initial K_0 conditions.

[Table 1 about here.]

5.1 Dependency of λ_d on different studied factors

The following observations may be summarised based on results presented in Tab. 1.

- λ_d depends on the assumed material parameters, i.e. on the soil type. The very small strain shear modulus G_0 influences $\_d$ remarkably. Interestingly, λ_d does not appear to be influenced significantly by the soil behaviour in the large strain range. Varying the very small strain shear modulus G_0 imposed changes of λ_d of the order of ± 0.1 in comparison with the original values, while varying G_{ls} had only slight effect on λ_d of the order of ± 0.03 at maximum. This result might appear surprising, as both G_0 and G_{ls} were shown to have substantial effect on the predicted displacements, both for the Heathrow express trial tunnel (Fig. 5) and Dobrovskeho exploratory adit (Fig. 8).

  A consequence of this observation is that a change of geological conditions during excavation of a single tunnel might require appropriate modification of λ_d values used in the simulations.

- K_0 does not appear to have substantial effect on $\lambda_{d.}$
- For the same soil type, the tunnel size and geometry influences significantly the appropriate values of λ_d. In the case of the Dobrovskeho study, $\lambda_d \approx 0.5$ was found for the exploratory adit, whereas $\lambda_d \approx 0.3$ for the whole tunnel. Thus, if λ_d found on the basis of results of an exploratory adit simulations was used for predictions of the full tunnel response, it would lead to an overestimation of the tunnel deformations. This is demonstrated in Fig. 13, in which the Dobrovskeho tunnel simulations are repeated with λ_d calibrated based on simulations of exploratory adit. These simulations lead to approximately 35% larger surface settlements.

[Figure 13 about here.]

5.2 Accuracy of the 2D predictions of displacement fields

The accuracy of the 2D predictions was studied on the basis of analyses of the three case histories with the original parameter values (and $K_0 = 1.25$ for Dobrovskeho case studies). Figure 14 shows predictions by the 2D method and full 3D method for the three case studies analysed. Figure 14a shows the surface settlement troughs, Fig. 14b gives horizontal displacements from an inclinometer located approximately

1D from the tunnel boundary, and Fig. 14c presents vertical displacements from an extensometer located above the tunnel axis.

The surface settlement troughs by the 2D and 3D methods match very well. An overall agreement could have been expected, as λ_d was calibrated with the intention to match the magnitude of surface settlements as accurately as possible, but it is interesting to observe that also the settlement trough shape is predicted accurately by the 2D method. Plots of variation of vertical and horizontal displacements with depth show good agreement for the Heathrow express case. For both Dobrovskeho adit and Dobrovskeho tunnel, the 2D method underestimates the displacements within a distance of approximately 1 tunnel diameter from a tunnel. The predictions match well outside this region.

[Figure 14 about here.]

The overall field of vertical displacements is shown in Fig. 15. An overall agreement of 2D and 3D method is good. The most notable discrepancy is in the predictions of the exploratory adit. The 2D method predicts higher vertical displacements above the sides of the adit then above its axis. This is caused by high K_0 value adopted in the Dobrovskeho case study analyses presented in this section ($K_0 = 1.25$). In the 3D analyses, this effect is not that significant and the method predicts more reasonable shape of the vertical displacement field in a close vicinity of the adit.

[Figure 15 about here.]

6 Concluding remarks

The 2D load reduction method for simulating NATM tunnels using plane strain finite element method was in the paper evaluated by comparison with fully 3D simulations of three different case histories. The paper focused on the predictions of displacement field, rather then on predictions of the tunnel lining forces or tunnel face stability. The 3D simulations considered the time-dependent behaviour of shotcrete lining, whereas the 2D method not. It was shown that for an optimum value of the parameter λ_d the displacement field predicted by the 2D method agrees well with the 3D simulations. In some cases only, a discrepancy was observed in a close vicinity of the tunnel. The parameter λ_d was found to be dependent on the problem simulated (for the same material) and also on the material properties (for the same tunneling problem). Considering material properties, the very small strain shear modulus was found to be more influential on the λ_d value than the large strain shear modulus. The initial K_0 stress state was not found to influence λ_d substantially.

One of the consequences of the results obtained is that a change in geological conditions during excavation of a single tunnel might require appropriate modification of λ_d values used in the simulations. Also, the λ_d value found on the basis of monitoring of an exploratory adit (or different tunnel in the same environment) cannot be simply used for simulations of the new tunnel. However, if the λ_d value is chosen appropriately, the load reduction method can relatively successfully account for the 3D effects. Nevertheless, some discrepancy may be expected in the close vicinity of the tunnel and in predictions of the tunnel convergence.

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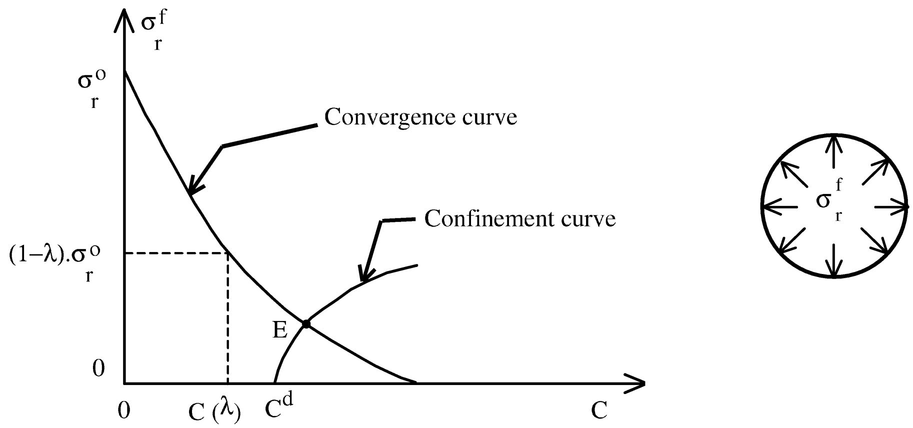
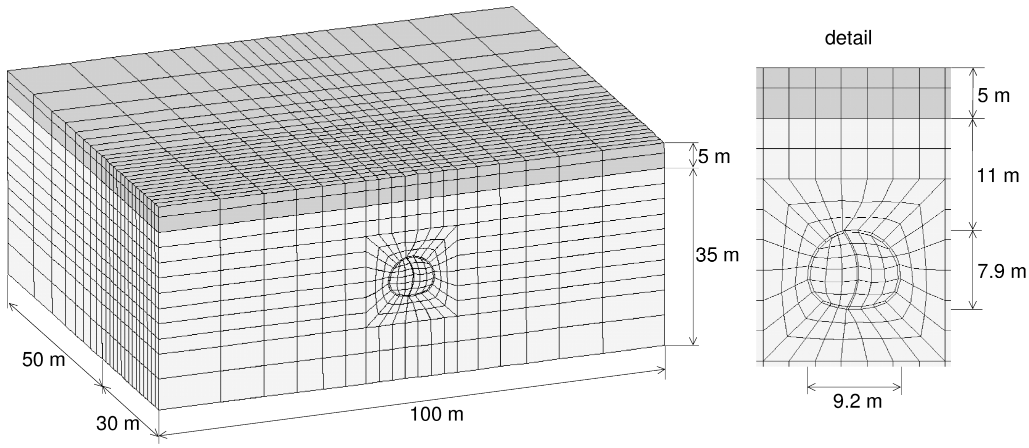


Figure 1: Main principles of the convergence-confinement method (from Leca et al., 2000)



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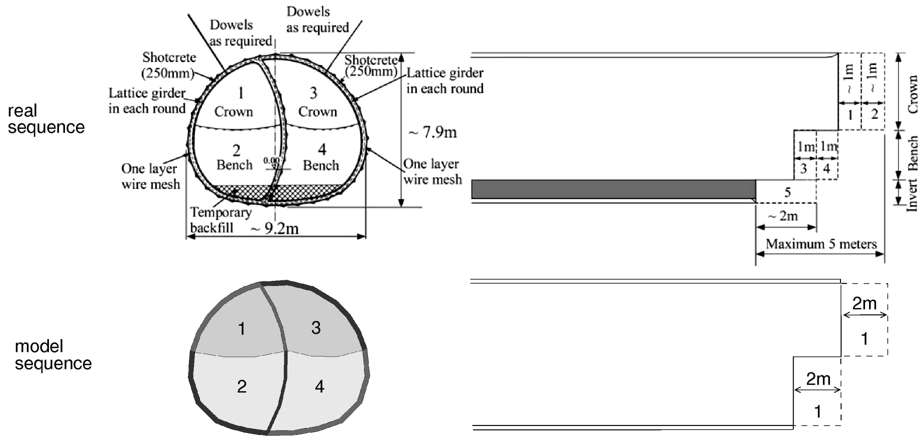

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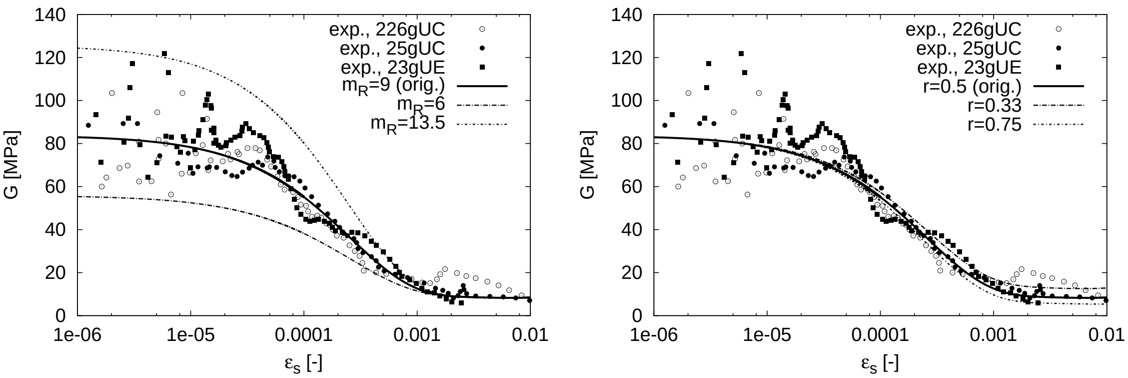

(a) (b)

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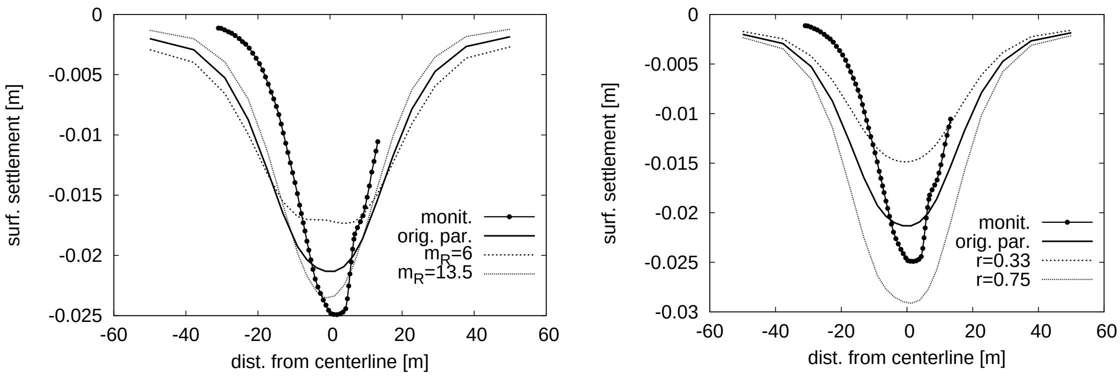

(a) (b)

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(a) (b)

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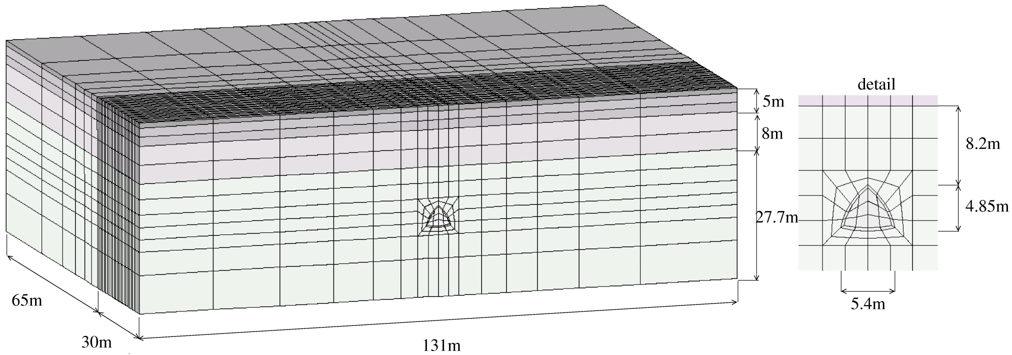

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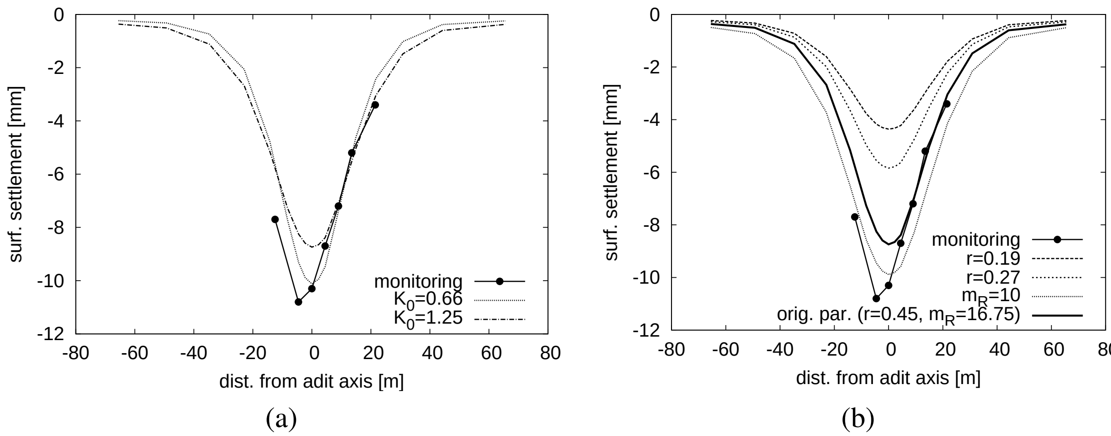

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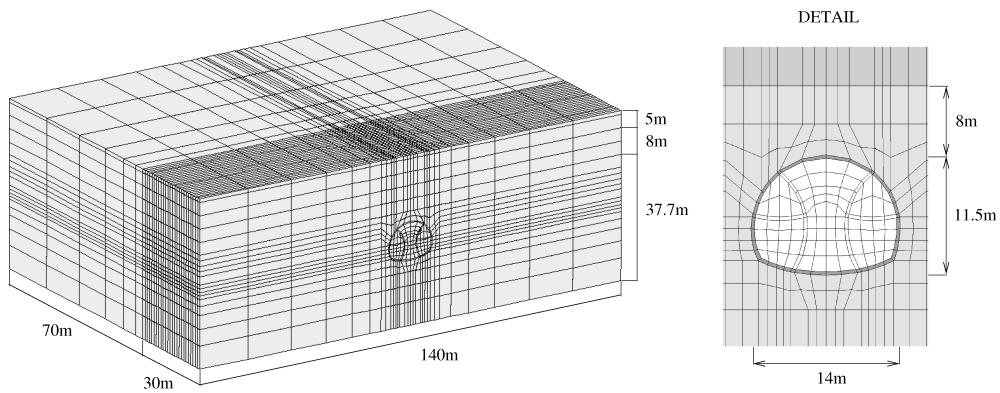

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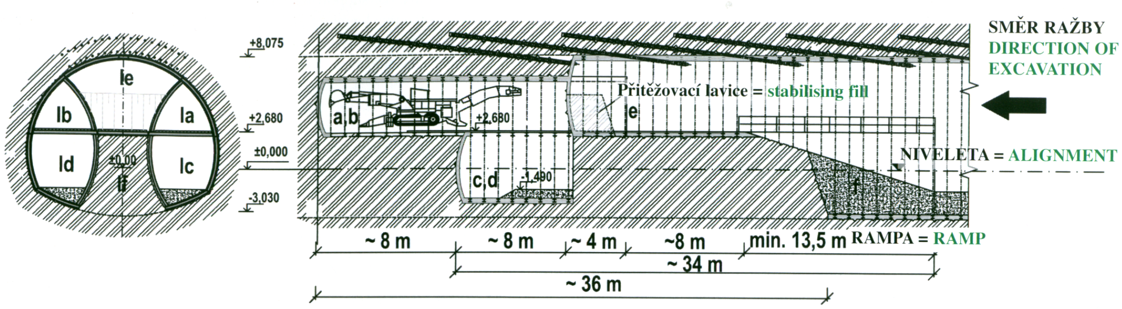

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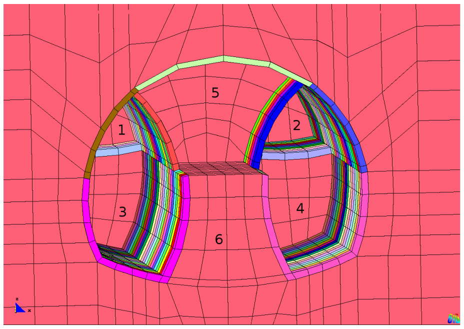

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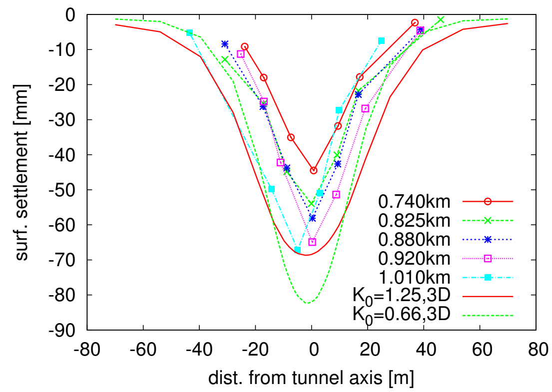

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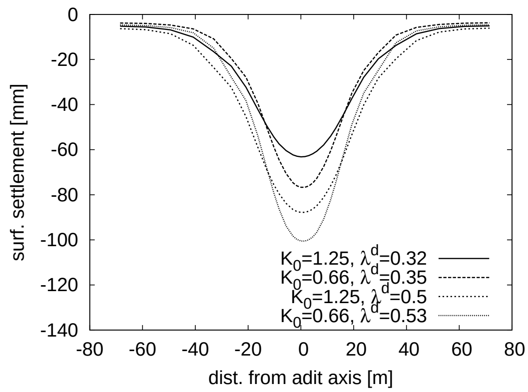

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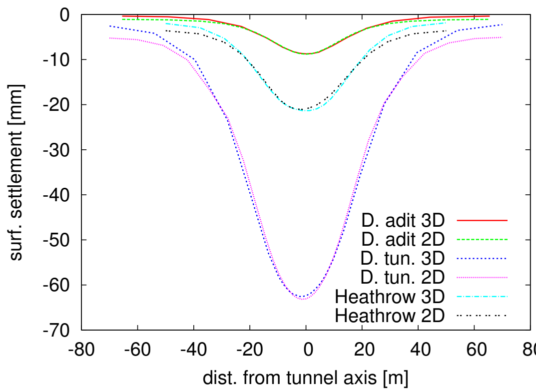
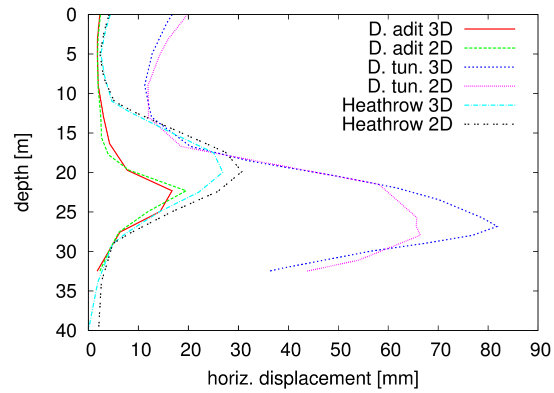


(a)



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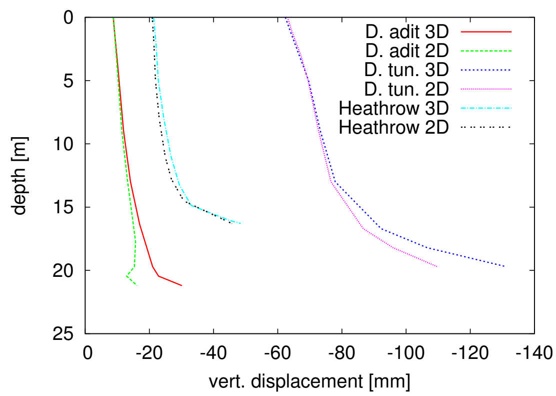

(c)

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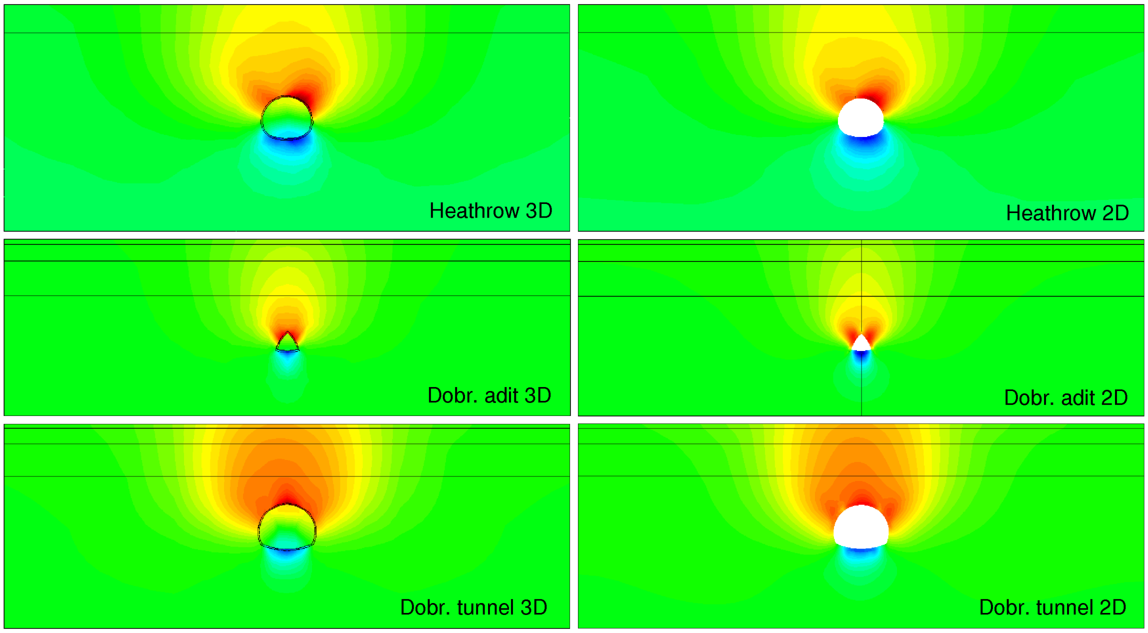

Figure 15: Qualitative comparison of vertical displacement field predicted by the 3D and 2D methods for the three case studies analysed (the same color scale for corresponding 2D and 3D analyses).

Table 1: *Summary of the CCM simulations*

Case study	$G_0(simul.)$	$G_{ls}(simul.)$	K_0	λ^d
Heathrow	$= G_0$	$= G_{ls}$	variable	0.56
Heathrow	$= G_0$	$> G_{ls}$	variable	0.59
Heathrow	$= G_0$	$< G_{ls}$	variable	0.53
Heathrow	$< G_0$	$= G_{ls}$	variable	0.48
Heathrow	$> G_0$	$= G_{ls}$	variable	0.65
Dobr. - adit	$= G_0$	$= G_{ls}$	1.25	0.50
Dobr. - adit	$= G_0$	$\gg G_{ls}$	1.25	0.52
Dobr. - adit	$= G_0$	$> G_{ls}$	1.25	0.53
Dobr. - adit	$< G_0$	$= G_{ls}$	1.25	0.40
Dobr. - adit	$= G_0$	$= G_{ls}$	0.66	0.53
Dobr. - tunnel	$= G_0$	$= G_{ls}$	1.25	0.32
Dobr. - tunnel	$= G_0$	$= G_{ls}$	0.66	0.35