In situ stresses and their role in the geotechnical analyses – a state of the art

The in situ effective stresses represent an important initial condition for geotechnical analyses. Typically, the horizontal stress is computed from the vertical stress using the coefficient of earth pressure at rest $K_0=\sigma_h'/\sigma_v'$, 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. Franzius et al. (2005) made a direct investigation into the influence of K_0 conditions in 3D finite element analysis of a tunneling problem using K_0=1.5 and K_0 =0.5. The unrealistically low K_0 value for London Clay led to better predictions: the normalised settlement trough was narrower and deeper. In absolute values, however, low K_0 caused overprediction of surface settlements by a factor of 4. With K_0=1.5 the predicted trough was too wide and vertical displacements were underpredicted by the factor of 4. Similar conclusions were drawn by Doležalová (2002) reporting a narrowing of the settlement trough and increasing of the vertical settlements in absolute terms on decreasing the K_0 value from 1.5 to 0.5 .

For normally consolidated soils the estimation of horizontal stresses is not a major problem. Jaky's equation in its usual simplified form of $K_{0nc}=1-\sin\varphi_c'$ may be used in determining the K_{0nc} for normally consolidated soils (Jaky, 1948; φ_c' is the critical state friction angle). There is a lot of experimental evidence throughout the literature that the Jaky formula represents the at rest coefficient of normally consolidated soils well provided the critical state effective friction angle φ_c' is used (Mesri and Hayat, 1993; Mayne and Kulhawy, 1982).

For overconsolidated clays however neither a general formula nor a generally applicable experimental procedure for determining the initial stress are available to date. In summarising the knowledge about the mechanical behaviour and characterisation of a typical example of overconsolidated clays – the Tertiary London Clay, which has been a subject of very intensive research for many decades, Hight et al. (2003) noted: „Still the most difficult parameter to determine for the London Clay is K_0“.

Determination of horizontal stresses (K_0)

Two categories of methods for determining K_0 can be distinguished: direct methods, in which the horizontal stress is measured directly, and indirect methods. The examples of the indirect methods are the correlations based on the OCR, use of horizontally oriented specimens in the laboratory oedometer, or the use of capillary pressure to compute the horizontal effective stress (Feda, 1978). Hamouche et al. (1995) classified three categories of the methods: in situ, laboratory and correlation ones. In situ methods were divided further into the intrusive and nonintrusive ones. Hydraulic fracturing, total pressure cells and dilatometers represented the intrusive methods, during which no precaution can be made to avoid disturbance. The self boring pressuremeter was believed the only nonintrusive in situ method available. The laboratory methods make use of either the triaxial device with a radial strain transducer, or the oedometer capable of measuring the horizontal stress. A typical correlation method is an experimentally based modification of the Jaky formula.

Direct measurements

Horizontal stress in clay are most often determined by a self-boring pressuremeter (e.g., 'Camkometer' - Wroth and Hughes, 1973), by the flat dilatometer (Marchetti, 1980), or different types of pushed-in spade-shaped pressure cells (e.g., Tedd and Charles, 1981; Handy et al, 1982). The use of push-in instruments in stiff clays is often questioned due to possible problems with the installation and due to the soil disturbance. The latter reason together with the possibility of imperfect fit in the bore hole seems to have disqualified the Menard-type pressuremeter in stiff clays. A good agreement of K_0 values obtained by push-in spade-shaped pressure cells and Camkometer for London Clay was reported by Tedd and Charles (1981), the 'spade' producing a smaller scatter and better reproducibility. Hamouche et al (1995) reported results by Marchetti dilatometer consistent with those obtained with the self boring pressuremeter in overconsolidated sensitive Canadian clays.

A hydraulic fracturing technique for clays for measuring the horizontal total stress was developed by Bjerrum and Andersen (1972). It is considered either direct (Feda, 1978) or indirect method (Lefebvre et al., 1991). The method is based on measuring the stress at closing of a vertical crack that had previously been formed by pressurised water. The method can hardly be used under the condition of $K_0 > 1$ as a horizontal crack would be formed instead of the vertical one, and „...the method will just measure the weight of the overburden...“ (Bjerrum and Andersen, 1972; Hight, D., 2009 –

personal communication). A recent 2D numerical study by Wang et al (2009) also considers horizontal cracks being formed in the case of $K_0>1$, i.e. in overconsolidated clays. However, Lefebvre et al. (1991) using methylene blue tracer in studying the shapes of clay fracturing reported vertical cracks formed in overconsolidated clays of $K_0>1$. The measured K_0 values were higher than when approximated by the established K_0-OCR correlations (by Mayne and Kulhawy, 1982 - see next section). A similar conclusion was made by Hamouche et al (1995), who also found that horizontal pressure determined by fracturing corresponded well to the self boring pressuremeter and Marchetti dilatometer results.

Indirect measurements

Skempton (1961) made use of four ways of determining the capillary pressure of the undisturbed samples in the laboratory: direct and indirect measurement of the load preventing swelling, analysis of the undrained strength measured in the triaxial device, and measurement of pore water suction in the triaxial specimen. The averaged capillary pressure from the four methods was used to compute the effective horizontal stress from the equation $p_{capillary}=\sigma_{vertical}'(K_0-A_{swelling}(K_0-1))$, where the pore pressure coefficient $A_{swelling}$ was determined in the triaxial apparatus.

Burland and Maswoswe (1982) used the method in supporting the use of direct measurements of horizontal stresses in London clay. Figure 1 shows that their suction based results agreed well with the Camkometer self boring pressuremeter and the push-in 'spade' by Tedd and Charles (1981).

A modernised version of the Skempton's procedure makes use of the „suction probe“ capable of direct measurement of capillary suctions within undisturbed samples taken by a thin walled samplers (Hight et al., 2003; Hight, D., 2009 – personal communication). However the only up-to-date alternative in London clay projects seems to estimate K_0 on the basis of lift-off pressures measured in self-boring pressuremeter tests, although interpretation remains controversial (Hight et al, 2003). The profiles of K_0 obtained by suction measurements (suction probe and filter paper technique) and self boring pressuremeter are in Figure 2.

Doran et al. (2000) studied the changes of pore pressures and effective stresses in the laboratory specimens on sampling and preparation. They concluded that using isotropic elasticity in the 'suction method' results in underestimating the determined K_0.

The correlation methods for determination of K_0 are represented by the Jaky formula for normally consolidated soils and by its extensions to cope with the overconsolidated soils in the form of $K_{0oc}=(1-\sin\varphi_c')\times(OCR)^{\alpha}$. The most common alternative for the exponent is $\alpha=\sin\varphi$ (Mayne and Kulhawy, 1982), or $\alpha=0.5$(Meyerhof, 1976). Some studies indicated $\alpha \approx 1.0$ (Lefebvre, 1991; Hamouche, 1995 - see above). Using such correlations however neglects other effects than the stress history (unloading), for example creep and cementation that might have developed in the soil in situ, and may lead to

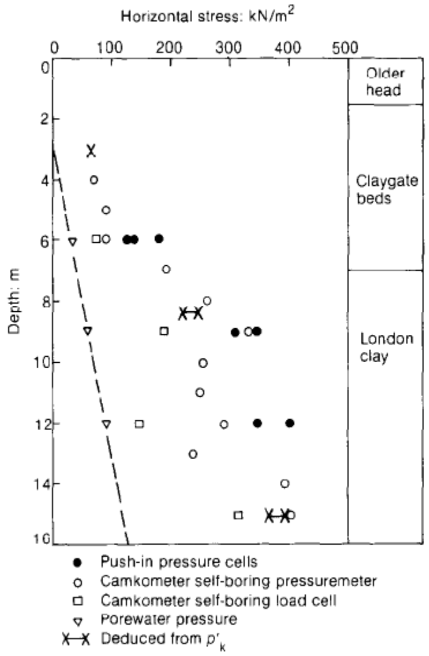

Figure 1 Comparison of direct methods and suction measurements (Burland and Maswoswe, 1982).

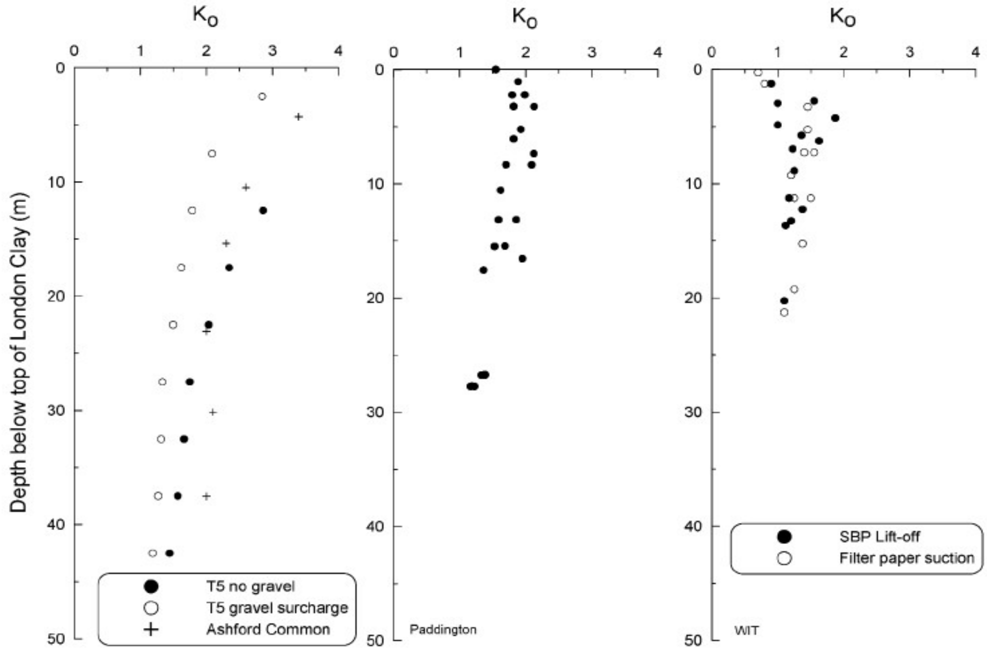

Figure 2 K_0 profiles in London Clay by different methods (Hight et al., 2003).

erroneous values of K_0.

An experimental determination using the advanced triaxial instrumentation (stress path testing, local LVDT gauges mounted on the specimens etc.) was suggested by Garga and Khan (1991) and Sivakumar et al (2009). The latter proposed and experimentally confirmed a new expression $K_{0oc}=1/\eta(1-(1-\eta K_{0nc})OCR^{(1-\chi)})$, which takes account of OCR (parameter χ is determined by 1-D and isotropic compression tests on undisturbed specimens) and of anisotropy (parameter η is determined from a CIUP test). K_{0nc} can be determined, for example, by Jaky's formula.

Doležalová et al. (1975 in Feda, 1978) made use of the displacements measured after unloading the massif by means of a gallery. The deformation parameters of the rock were determined by independent in situ testing and then the FEM was used to simulate elimination of the monitored displacements of the gallery. The stresses necessary for the simulation were considered the in situ stresses in the massif. A similar approach using an advanced hypoplastic model and advanced determination and calibration of the parameters is also planned as one of the methods within the proposed project.

The review shows that in determining initial stresses in overconsolidated clays a single method can hardly be sufficient. The best way seems taking good quality samples (thin wall sampler) and measuring suctions, and comparing the result with a direct measurements, for which push-in spade-shaped pressure cells or self boring pressuremeter seem most promising. If available, convergence measurements of a underground cavity (gallery) evaluated using a numerical model with an advanced anisotropic constitutive model is believed the best method.

Current local case histories – Brno Clay

In the Czech Republic, Neogen overconsolidated clays of Southern Moravia (Brno Clay) and the Miocene clays overlying the brown coal seams in the North-West Bohemia may serve as typical examples of soils where K_0 cannot be determined by the simple methods suitable for normally consolidated soils. Further, both the Tertiary clays are commonly encountered in geotechnical projects, and two important tunnelling projects have taken place in the particular soils recently – the Březno railway tunnel near Chomutov and Královo Pole motorway tunnels in Brno. However the determination of the in-situ K_0 for both the major tunnelling projects during the site investigation was far from satisfactory.

In Brno an attempt was made to measure in situ stresses by hydraulic fracturing, by measurements of convergence in a test gallery of a circular cross section, and by a dilatometer measurements („contraction-meter probe"; Pavlík et al, 2004). The hydraulic fracturing technique failed due to difficulties firstly in producing the sufficient pressures, secondly in determining the orientation of the created (if any) cracks (Staš, 2002). The convergence measurements in the test gallery on the other hand are believed extremely valuable data, and the proposed project suggests their interpretation by means of an advanced newly developed anisotropic constitutive model using FEM. The dilatometer method adopted in the site investigation was aimed at developing a new device and no data were obtain to be used in the design (Pavlík et al, 2004).

Significance of K_0 for numerical modelling

The quality of geotechnical predictions is substantially influenced also by the constitutive model used in the numerical analysis. The influence is pronounced in a soil massif with high K_0 value, when the ability of a model to predict the high initial very-small-strain stiffness and its non-linear decrease with straining is crucial. In the following text such models are denoted 'small-strain nonlinear', as opposed to the 'small-strain linear' models, which lack the above capabilities.

Several studies gave a direct comparison between predictions of tunneling problems by using "small-strain linear" and "small-strain nonlinear" models. Addenbrooke et al. (1997) performed two dimensional FE analyses of a tunnel in London Clay with K_0=1.5 by small-strain linear and nonlinear elastic perfectly plastic models. The nonlinear models, which were calibrated to fit the laboratory determined decay of soil stiffness, gave better results in comparison with linear models, although the predicted surface settlement trough was still shallower and wider than the measured one. It was concluded that "unrealistic soil stiffness was required to achieve an improved prediction with K_0>1." Similar results were reported by Gunn (1993) and the necessity to model small-strain nonlinearity has been accepted by many others (Dasari et al. 1996; Franzius et al. 2005, Grammatikopoulou et al. 2002; Yazdchi et al. 2006).

Another aspect controlling the predictions is soil anisotropy. Direct investigations into the influence of soil anisotropy were presented by Addenbrooke et al. (1997), Gunn (1993), and Franzius et al. (2005). In all cases, small-strain nonlinear models were used and, in all cases, it was concluded that incorporation of soil anisotropy (higher stiffness in horizontal than in vertical direction) improved the predictions by narrowing and deepening the settlement trough. Obviously, an advanced anisotropic model is needed for correct evaluation of K_0 from the above convergence measurements (Doležalová et al., 1975; Pavlík et al., 2004).

The influence of the constitutive model on predictions of the Heathrow Express trial tunnel in London clay was presented by Mašín (2009), who compared two different constitutive models. The first one was Modified Cam clay model, which incorporates the influence of the initial void ratio on the soil behaviour, and it is thus more advanced than the most commonly used Mohr-Coulomb model. The Modified Cam clay however does not predict the high very-small-strain stiffness and its non-linear decrease with straining. The second model was an advanced nonlinear hypoplastic model for clay (Mašín, 2005) enhanced by the intergranular strain concept (Niemunis and Herle, 1997). Figure 3 shows very high initial K_0 values (data by Hight et al., 2007). Qualitative predictions of the displacement fields are in Figure 4. The hypoplastic model gives a realistic displacement field with maximum of surface settlements above the tunnel centre-line, while Modified Cam clay leads to the unrealistic vertical heave above the tunnel and downwards displacements in a distance from the tunnel centre-line. The quantitative comparison of the predictions is in Figure 5. The hypoplastic model overpredicts the width of the surface settlement trough, but the magnitude of displacements and the overall shape of the settlement trough is predicted relatively correctly. On the contrary, the Modified Cam-clay model gives unrealistic predictions with an inverse shape of the settlement trough.

The above case history of Královo Pole has been modelled by Svoboda et al. (2010). They provided a class A (Lambe, 1973) predictions of the displacement field induced by the 14 m wide road tunnel excavated in a stiff clay deposit with an overburden of 6 m to 21 m. The advanced hypoplastic model was calibrated on laboratory data and its parameters were optimised using the monitoring data from an exploratory drift excavated in advance of the future tunnel. Based on the optimised data set, class A predictions of the displacement field due to the tunnel were performed in 2008 and early 2009. In November 2009, the tunnel excavation passed the simulated cross-section, which allowed us to compare the predictions with the actual data from the geotechnical monitoring.

One of the important problems of the simulations was the unknown value of K_0. There were no reliable in situ measurements of K_0 available (see above) and therefore two bound values of K_0 were considered in the analyses. First the value of K_0 according to the Mayne and Kulhawy (1982) relationship. From the oedometer test on undisturbed Brno clay the overconsolidation stress of 1800 kPa was estimated, leading to OCR of 6.5 and to K_0 =1.25. In this procedure the assumption was made that the overconsolidation was caused by the actual soil unloading (erosion). Creep was the other

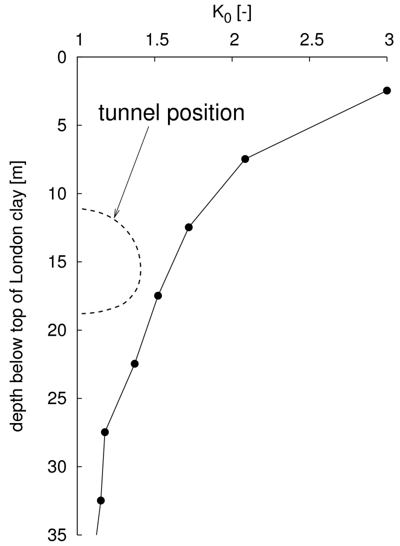

Figure 3 The initial K_0 state for the Heathrow Express trial tunnel (Mašín, 2009).

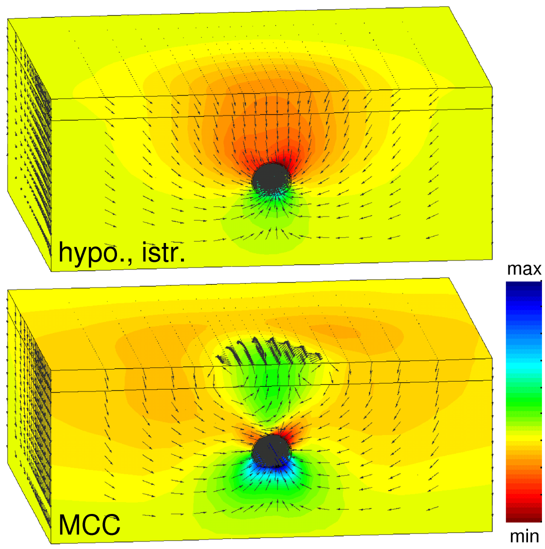

Figure 4 Displacement field due to Heathrow Express tunnel. Predictions by hypoplastic model (hypo., istr.) and by Modified Cam clay model - MCC (Mašín, 2009).

possible interpretation of the - apparent - overconsolidation, which would lead to the value of K_0=0.66 according to the Jaky relationship (for normally consolidated soil).

The assumed value of K_0 influenced the calculated results substantially. Figure 6 shows the surface settlement trough, as predicted by the hypoplastic model in advance of the tunnel excavation. After the tunnel excavation, the predictions were compared with the monitoring data, also shown in Figure 6. The hypoplastic model provided very accurate predictions of the surface settlement trough. An important uncertainty entering the simulations was the K_0 value.

The K_0 value influences significantly also the distribution of horizontal displacements with depth. Figure 7 shows the predictions for the two K_0 values compared with monitoring data. Two observations can be made. First, the results depend substantially on the assumed K_0 value. Second, the hypoplastic model overpredicts significantly the magnitude of horizontal displacements with depth. This discrepancy is caused by inability of the model to predict anisotropy of the very-small-strain stiffness. Both issues - specification of the initial K_0 state and incorporation of anisotropy into the hypoplastic model - are in scope of this grant proposal.

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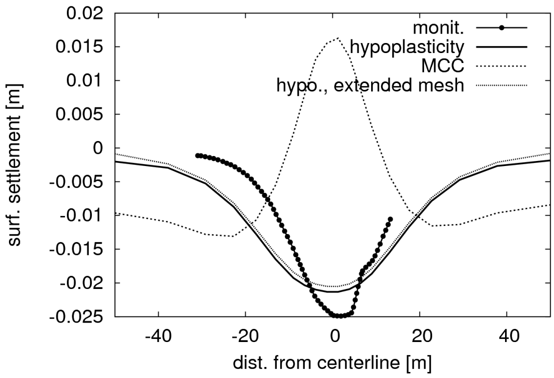

Figure 5 Surface settlement trough from the Heathrow Express case study, predictions by two constitutive models.

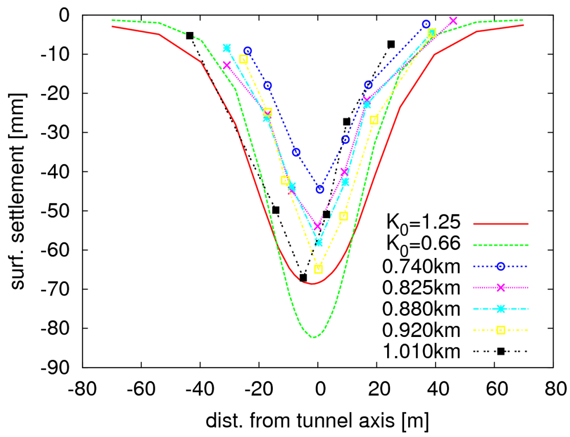

Figure 6 Predictions of settlement trough of Královo Pole Tunnel for two K_0 values, compared with monitoring data (Svoboda et al., 2010).

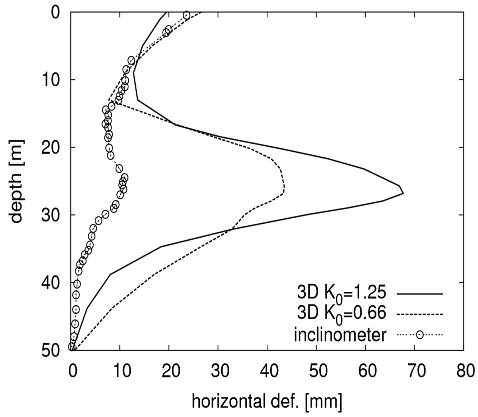

Figure 7 Predictions of horizontal displacements of Královo Pole Tunnel for two K_0 values, (Svoboda et al., 2010).

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