Methods of determination of K_0 in overconsolidated clay

Méthodes de détermination de K_0 dans une argile surconsolidée

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ABSTRACT: In situ effective stresses, expressed usually by the at rest coefficient K_0, influence the mechanical behaviour and thus any geotechnical analysis. In normally consolidated soils K_0 can be computed according to the Jáky formula. For overconsolidated clays however neither a general formula nor a general experimental procedure are available. The paper summarizes briefly the available methods and then presents some methods in more detail. First the use of flat dilatometer is discussed. Further, K_0 is determined by back analysing the convergence of a circular test gallery. Finite element analysis using the hypoplastic constitutive model to represent the clay behaviour is adopted in the analyses.

RÉSUMÉ : Les contraintes effectives in situ, exprimées habituellement à travers le coefficient des terres au repos K_0, ont une influence sur le comportement mécanique des sols, et donc, sur toute analyse géotechnique. Dans les sols normalement consolidés, K_0 peut être calculé à partir de la formule de Jáky. Pour les argiles surconsolidées cependant, il n'existe ni formule générale, ni procédure expérimentale reconnue pour évaluer ce paramètre. La communication présentée résume brièvement les méthodes actuellement disponibles, puis présente quelques méthodes plus en détail. D'abord, l'utilisation du dilatomètre plat et d'une cellule de pression en forme de pelle est discutée. En outre, K_0 est déterminé à partir de l'analyse en retour de la convergence d'une galerie d'essai circulaire. Une approche en éléments finis, basée sur l'utilisation d'une loi de comportement hypoplastique pour modéliser le comportement des argiles, est adoptée dans l'analyse.

KEYWORDS: clay, earth pressure at rest, horizontal stress, anisotropy, hypoplasticity, tunnelling.

MOTS-CLÉS : argile, pression des terres au repos, contrainte horizontale, anisotropie, hypoplasticité, tunnel

1 INTRODUCTION

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.

For normally consolidated soils the estimation of horizontal stresses is not a major problem. Jáky'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 (Jáky, 1948; φ_c' is the critical state friction angle). There is a lot of experimental evidence throughout the literature that the Jáky 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“.

1.1 *Direct methods of K_0 determination*

Horizontal stress in clay is most often determined by selfboring 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). 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 borehole 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). 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). 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). A similar conclusion was made by Hamouche et al. (1995), who also found that the horizontal pressure determined by fracturing corresponded well to the self boring pressuremeter and Marchetti dilatometer results.

1.2 *Indirect methods of K_0 determination*

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, and the pore pressure coefficient was determined in the triaxial apparatus.

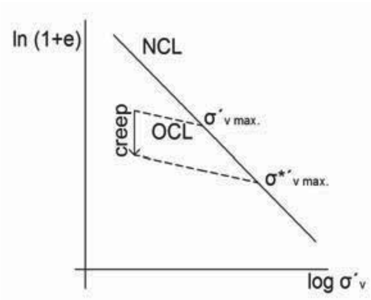

Figure 1. The influence of creep on $\sigma'_{v\,max}$. position at oedometer test.

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

The current 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 (e.g., Hight et al., 2003). 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 K_0. The only up-to-date alternative in London clay projects seems to be 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 correlation methods for determination of K_0 are represented by the Jáky 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 et al., 1991; Hamouche et al., 1995). 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 erroneous estimation of the values of K_0. Creep moves the position of the real maximal vertical stress to the position of an apparent maximal vertical stress (Fig. 1). The oedometer test is a common technique for evaluating $\sigma'_{v\ max}$.. Due to creep however the test produces a pseudo-overconsolidation value of σ_{vmax}*' instead of the present overconsolidation pressure σ_{vmax}'. Hence, both the OCR and $K_{0,OC}$ values determined by the correlations and not considering creep (ageing) are overestimated.

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 Jáky'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 is presented further.

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 Marchetti dilatometer (DMT), 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.

2 GEOLOGY AND CHARACTERISTICS OF CLAY INVESTIGATED

Different methods were used to evaluate K_0 of clay from Brno, Czech Republic. The investigated calcitic silty Brno Clay (“Tegel”) of Miocene (lower Badenian) age belongs to the Neogene of Carpathian foredeep, and reaches the depth of several hundred metres. Sound Tegel has a greenish-grey colour, which changes to yellow-brown to reddish-brown colour at the zone of weathering closer to surface. According to X-Ray analysis there is a substantial percentage of CaO (ca 20%) and the main minerals are kaolinite (ca 23%) and illite (22%), calcite (20%), quartz (17%), chlorite (up to 10%) and feldspar (Boháč et al., 1995). Tegel exhibits stiff to very stiff consistency. The clay is overconsolidated but the height of eroded overburden is not known. Above the Miocene clay there are Quartenary gravels overlain by loess loam. The clay is tectonically faulted. The groundwater is mostly bound to Quartenary fluvial sediments, and the collectors are typically not continous. However the clay is 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 is about 0.9.

3 MARCHETTI DILATOMETER MEASUREMENTS

At the site the current phreatic water table was 4.7 metres under the surface and top layer of about 5.5 metres was excavated some 30 years ago. This generated negative pore water pressures, which have not fully dissipated yet. At the current depth of 11.7 metres the pore pressure of -32 kPa was measured (after dissipation of excess pore pressures caused by the sounding) by a push-in spade pressure cell. The present vertical effective stress in the depth of 11.7 metres calculated from the soil unit weight and pore water pressure was σ_v' = 185 kPa.

The K_0 was measured using Marchetti (1980) flat dilatometer. The measured K_D according to Marchetti (1980) was 8.0 and K_0 derived using the empirical equation $K_0 = (K_D/1.5)^{0,47} - 0.6$ was $K_0 = 1.6$. This is substantially higher than K_0 determined from oedometric yield point and the empirical correlation of Mayne and Kulhawy (1982) K_0 =1.2.

4 NUMERICAL ANALYSIS OF MARCHETTI DILATOMETER

An attempt was made to explain this discrepancy by numerical modelling of the flat dilatometer penetration into the soil. For the numerical analysis the hypoplastic model (Mašín, 2005) was used in combination with the intergranular strain concept (Niemunis and Herle, 1997). The model predicts nonlinear stiffness depending on the strain level. The input value of K_0 of 1.2 was considered. Both the calibration and the parameters for the hypoplastic model were taken from Svoboda et al. (2010) and Mašín (2012). The parameters are summarised in Table 1.

Table 1. Parameters of the hypoplastic model

φ_c	λ^*	κ^*	N	r
22°	0.128	0.015	1.51	0.45
m_R	m_T	R	β_r	χ
16,75	8.375	1.e-4	0.2	0.8

The numerical analysis was carried out using Plaxis 2D finite element code. The modelling sequence involved three phases:

1. Generation of the initial stress condition with K_0 = 1.2,
2. Excavation of the 5.5 metres thick layer in order to reach the measured pore water pressure of -32 kPa at the depth of 11.7 metres. Consolidation time was varied using the consolidation analysis until the measured excess pore water pressure was obtained.
3. The installation of the dilatometer was simulated in a simplified manner using two approaches. In the first one, displacement was prescribed at the left boundary of the model, as depicted in Fig. 2. The second analysis involved prescribed load. The dilatometer was 200 millimetres high and 14 millimetres wide (7 mm horizontal displacement was considered in the model thanks to its symmetry) and it was installed in the depth of 11.6 – 11.8 metres. In the analyses, load/displacement was evaluated in the centre of the dilatometer. These phases employed a plastic undrained analysis.

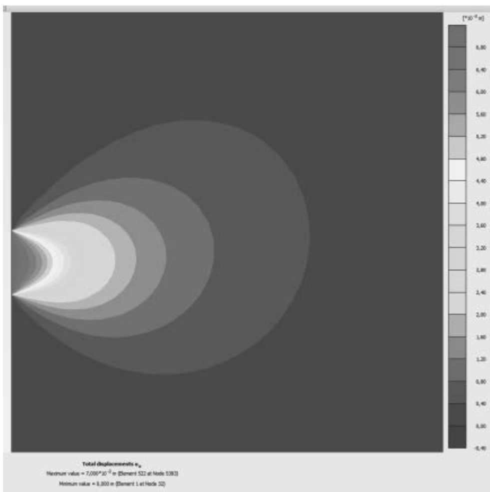

Figure 2. Distribution of horizontal displacements calculated by the hypoplastic simulation of Marchetti (1980) dilatometer.

The calculated coefficient K_D was 4.51 for the load controlled analysis and 4.06 for the displacement controlled analysis, which leads to K_0 equal 1.07 and 1.00 respectively. This preliminary analysis thus indicated slight underprediction of K_0 using Marchetti (1980) empirical equation. Limitations of the model need, however, be considered. In particular the simplified geometry and limitations of the adopted constitutive model, which does not allow for an explicit consideration of inherent stiffness anisotropy. To overcome this limitation, a new anisotropic version of the hypoplastic model is currently being developed.

5 BACKANALYSIS OF CIRCULAR ADIT

In the second numerical study presented, the K_0 coefficient is evaluated by means of backanalysis of convergence measurements within a circular exploratory adit. The adit was excavated as part of a geotechnical site investigation preceeding the excavation of Královo Pole Tunnels in Brno (see Svoboda et al., 2010).

The adit was located 26 m below the ground level, and its diameter was 1,9 m. Its geometry is shown in Fig. 3. The adit was protected by a steel net and rolled steel arches. These were installed for safety reasons only, and the support was never in full contact with the cavity wall. The monitored convergence of the cavity is thus assumed to be representative of the displacement of an unsupported massif. Its convergence was monitored by means of push-rod dilatometer in four different directions (vertical, horizontal and two sections inclined at 45 degrees).

Figure 3. Circular adit used in backanalyses of the earth pressure coefficient at rest K_0 (Pavlík et al., 2004).

The adit has been simulated in 2D and 3D using finite element method. The model properties were taken over from Svoboda et al. (2010). Hypoplastic model parameters are in Tab. 1. In the analyses, it was assumed that the massif properties were known. The initial value of K_0 was varied by a trial-and-error procedure until the model correctly reproduced the measured ratio of horizontal and vertical convergence of the adit. The analyses were performed under undrained conditions.

The analyses were performed using the softwares PLAXIS 2D and PLAXIS 3D. The 2D analyses adopted the load reduction method (see Svoboda and Mašín, 2011). In these analyses, the load reduction factor was varied to achieve the monitored displacement magnitude, and the coefficient K_0 was adjusted to reproduce the ratio of displacements in horizontal and vertical directions.

Geometry assumed in the 3D analyses is in Fig. 4. No effort was made to vary model properties to reach the exact monitored displacement magnitude. As in 2D analyses, K_0 was backanalysed to fit the displacement ratio. To represent the real excavation and monitoring procedure, displacements were reset in simulations once the adit face passed the monitored section. They are thus not biased by the pre-convergence displacements, which are not registered by the rod dilatometers. Evaluation of horizontal displacements in the monitored section is demonstrated in Fig. 4.

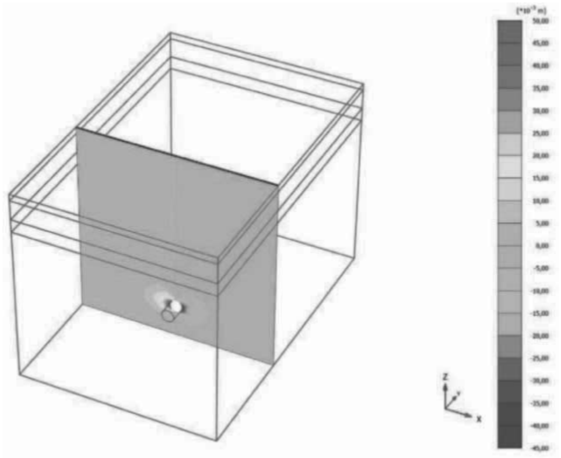

Figure 4. 3D model geometry and predictions of horizontal displacements.

Results of backanalyses are summarized in Table 2. It is clear that the 2D and 3D analyses were consistent in the estimation of K_0 (1.37 and 1.45 respectively). The 3D analyses overpredicted the displacement magnitude.

Table 2. Results of numerical backanalysis of circular exploratory adit.

	monitoring	2D model	3D model
horiz. conv. (u_h) [mm]	19.8	19.8	33.4
vert. conv. (u_v) [mm]	15.9	15.4	26.1
Ratio u_h /u_v	1.25	1.25	1.28
K_0	-	1.37	1.45

Similarly to Sec. 4, it is expected that the results obtained are negatively influenced by inaccurate representation of soil anisotropy using the hypoplastic model. The development of the new model is ongoing.

6 CONCLUSIONS

In the paper, we summarized different methods for evaluation of the earth pressure coefficient at rest K_0. Due to limitations of different methods, it is always advisable to combine different approaches based on laboratory investigation, field measurements and numerical analysis. Several results of the K_0 evaluation of the Brno Clay were presented, in all cases leading to K_0 higher than unity. These analyses are preliminary and they will be adjusted in the forthcoming work.

7 ACKNOWLEDGEMENTS

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