# Modelling of cementation bonds in clay – laboratory and numerical model

# Modélisation des liens de cémentation dans l'argile – modèle de laboratoire et modèle numérique

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## ABSTRACT

Artificial cementation bonds were created in a model clayey material by adding 4% of Portland cement to kaolin clay. The mechanical behaviour of the model material was compared with the behaviour of the pure reconstituted kaolin clay. Submersible LVDT´s and bender elements were used in identifying the elastic stiffness and the destructuration of the specimens. The triaxial CIUP tests were carried out in determining the shear strength. The aim of the laboratory study was to obtain a complete picture of the mechanical behaviour of the model material, which can be used for calibration, evaluation and further development of constitutive models.

Based on laboratory experiments, a new expression that relates very small strain shear modulus to the current degree of bonding, overconsolidation and mean stress was developed. This expression was incorporated into a hypoplastic model for clays with meta-stable structure enhanced by the intergranular strain concept. It is shown that the modified model predicts the behaviour of cemented soil in the very small strain range with better accuracy than the original model, and that both the original and modified models provide satisfactory predictions in the medium to large-strain range.

## RÉSUMÉ

Les liens de cémentation artificielle dans un matériau argileux modèle ont été créés en ajoutant 4 % de ciment de portland à de l´argile kaolinique. Le comportement mécanique du matériau modèle a été comparé avec celui de l´argile pure reconstituée. Pour l´identification de la dureté élastique et la déstructuration des échantillons, des LVDTs submersibles et des LVDTs submersibles et des piézocéramiques transducteurs ont été utilisés. Afin de déterminer la résistance au cisaillement, des triaxiaux CIUP ont été effectués, le but de l´étude de laboratoire étant l´obtention d´une image complète du comportement mécanique du matériau modèle, ce qui permettra ensuite de calibrer, d´évaluer et de développer davantage des modèles de comportement.

Les essais de laboratoire ont permis de proposer une nouvelle formule établissant un rapport entre le module de cisaillement très petit et le degré de liaison, la surconsolidation et la contrainte moyenne. Cette formule a été intégrée dans un modèle hypoplastique pour les argiles avec une structure métastable rehaussée par le concept de déformation intergranulaire. On démontre qu´en comparaison avec le modèle d´origine, le modèle modifié fournit une prédiction plus exacte du comportement du sol cémenté dans le domaine des très petits déplacements. Les deux modèles prédisent de manière satisfaisante le comportement dans le domaine des moyens à grands déplacements.

Keywords : cemented clay, compression, shear strength, stiffness, hypoplastic model

## 1 INTRODUCTION

Mechanical behaviour of natural clays is significantly affected by the structure (combination of fabric and bonding) formed during and after sedimentation. Testing of intact natural clays showed decrease of compressibility and increase of strength due to the effects of structure (Burland 1990; Leroueil & Vaughan 1990; Feda 1995; Cotecchia & Chandler 1997).

Typically the bonding is brittle and diminishes during sampling and specimen preparation. Artificial cementation is therefore considered as a good option to examine the mechanical behaviour of cemented clays. In this work, a mixture of kaolin clay with Portland cement was chosen as a suitable material to represent bonding in natural clays (Feda 2002; Horpibulsuk et al. 2004; Puppala et al. 2006).

Laboratory experiments were used as a basis for a calibration of the hypoplastic constitutive model for clays with meta-stable structure (Mašín 2007), enhanced by the intergranular strain concept (Niemunis & Herle 1997). A new expression relating shear modulus to mean stress and current degree of bonding is proposed.

## 2 SAMPLE PREPARATION

The soil specimens were prepared using commercially available kaolin clay (Lasselsberger, Kaznějov, CZ, minimum 20% of grains smaller than 0.002 mm, maximum 0.005% of grains larger than 0.2 mm, liquid limit 60-70%). The kaolin clay powder was mixed with distilled water at water content of 70%. After homogenization (24 hours), 4% of Portland cement was added (with respect to the dry mass of kaolin clay).

The homogeneous paste, formed by thorough mixing, was $K_0$-consolidated in the oedometer rings or in a high press in the case of triaxial specimens. The oedometer tests were carried out after 14 days of curing with permanent watering at vertical stress of approximately 0.5 kPa. The triaxial specimens were put into the triaxial cells after 3 days of curing at vertical stress of 5 kPa.

## 3 TEST RESULTS AND ANALYSIS

### 3.1 *Compression*

A part of the laboratory study was concerned with one-dimensional and isotropic compression tests of pure and cemented kaolin clay (Fig. 1). The maximum vertical stress reached up to 7 MPa and 16 MPa during the oedometer tests of pure kaolin and cemented specimens, respectively. The triaxial specimens were consolidated either in steps or by continuous compression up to 1500 kPa.

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The influence of cementation is shown mainly by the low one-dimensional and isotropic compressibility of cemented specimens until a threshold stress level. Subsequently, after surpassing the threshold, the compressibility increases owing to the process of debonding. After complete debonding the normal consolidation lines of originally cemented specimens should converge for both types of compression to the normal compression lines of pure kaolin clay. However, the maximum applied stresses were not high enough to confirm the phenomenon.

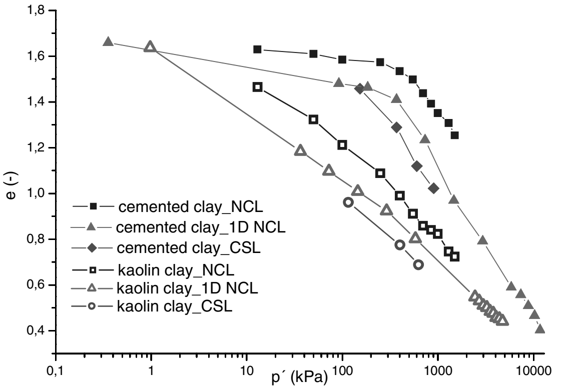

Figure 1. Behaviour of reconstituted cemented and pure kaolin clay during isotropic and one-dimensional compression and critical state lines

### 3.2 *Shear strength*

Shear strength of both materials was measured by triaxial CIUP tests. Figure 2 shows the obtained stress paths. The cemented material is characterised by higher values of the friction angle in the whole applied stress range and by cohesion of 48.5 kPa, obtained by the linearization of the strength envelope in the stress range from 100 to 600 kPa. The progressive debonding with the increasing mean effective stress makes the failure envelope of the cemented material nonlinear.

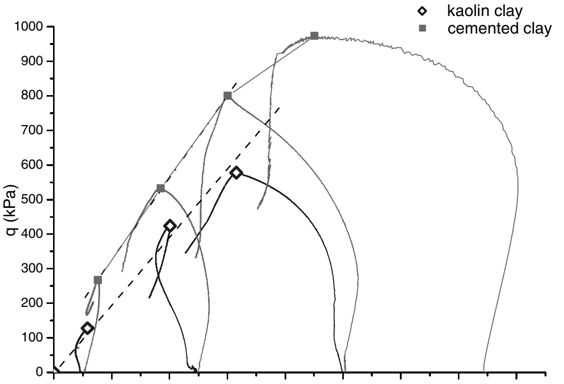

Figure 2. Stress paths and failure envelopes of reconstituted cemented and pure kaolin clay obtained from CIUP tests

### 3.3 *Stiffness at small strains*

Yielding and destructuration were identified from the development of shear stiffness, measured by submersible LVDT´s and by propagation of shear waves through the specimens (bender elements). Both types of measurements were carried out in the triaxial cell. During isotropic compression/consolidation measurements of shear stiffness were carried out at several stress levels. The LVDT´s were used during the deviator stress probes at standard total stress path $\Delta q/\Delta p = 3$. The change of the deviator stress was limited to 20 kPa.

In Figure 3 the shear stiffness of reconstituted normally consolidated pure kaolin clay is dependent on the mean effective stress in the very small strain range. On the contrary, the very-small-strain shear stiffness of the cemented kaolin clay is stress independent until a threshold. The destructuration after the threshold results in the stress dependency of the shear stiffness similar to the shear stiffness behaviour of pure kaolin clay. The shear stiffness of cemented material is higher in the whole measured stress range.

Moreover, the results in the Figure 3 reflect strain dependency of shear stiffness. At the strains of 0.006% the stiffness is lower than at the strains of 0.001% (results from LVDT´s). The maximum shear stiffness was obtained at very small strains (below 0.001%) from the propagation of shear waves through the specimens.

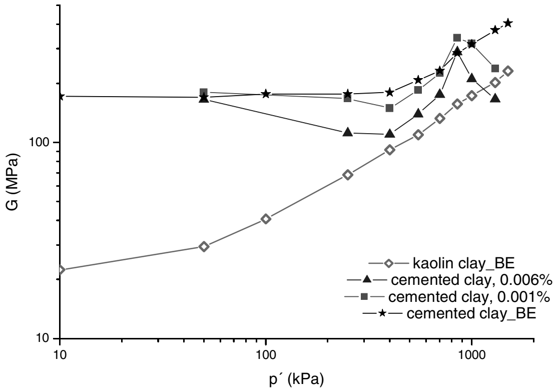

Figure 3. Behaviour of small-strain shear modulus versus mean effective stress measured using bender elements for cemented and pure kaolin clay and using LVDT´s for cemented clay (values at strain 0.006% and 0.001%)

## 4 CONSTITUTIVE MODEL

### 4.1 *Calibration of model parameters*

The hypoplastic constitutive model for clays with meta-stable structure (Mašín 2007), based on hypoplastic constitutive model for clays (Mašín 2005), was chosen to model the mechanical behaviour of the cemented kaolin clay. First, the reference constitutive model was calibrated for reconstituted pure kaolin clay using the data obtained from the isotropic compression tests (Fig. 4) and from the shear tests. The data from the isotropic compression allowed the calibration of parameters N, $\lambda^*$, $\kappa^*$. The parameters N and $\lambda^*$ define the position and the slope of the isotropic virgin compression line. The parameter $\kappa^*$ determines the slope of the swelling line. From the shear tests parameters $\varphi_c$ and r were calibrated. The parameter $\varphi_c$ is the critical state friction angle, the parameter r controls the shear modulus. The same values of these five parameters were used in simulations of both pure and cemented kaolin clay.

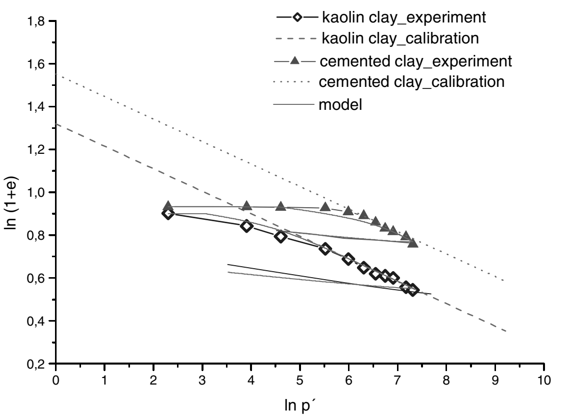

Figure 4. The calibration of the model based on the isotropic compression tests

The concept of sensitivity (Cotecchia & Chandler 2000) was integrated in the model through the use of parameters k, A and $s_f$ . The three parameters allow modelling of destructuration with increasing strain, from the phase of the initial resistance of the structure to compression and shear strain to progressive debonding. The comparison of experimental data and model predictions for isotropic compression behaviour are in Figure 4, the stress-strain diagrams are in Figure 5. The localization of strain to the shear surface results in the difference between the post-peak parts of the stress-strain diagrams (model vs. experiment).

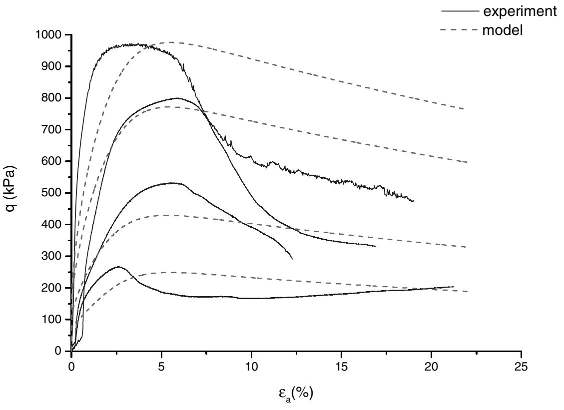

Figure 5. The calibration of the model based on the shear tests of cemented clay

For both pure and cemented kaolin clay the model was further enhanced with intergranular strain concept (Niemunis & Herle 1997). The concept improves the model prediction of the nonlinear behaviour of soils in the very small strain range, originally with five additional parameters, R, $m_R$, $m_T$, $\beta_r$ and $\chi$. The parameters R, $\beta_r$ and $\chi$ define the decrease in stiffness with increasing strain, the parameters $m_R$ and $m_T$ control the initial stiffness. The model was modified in order to catch the dependency of the initial stiffness on stress, overconsolidation and current state of structure. The parameters $m_R$ and $m_T$ were replaced by parameters $A_G$, n, m and l. The details of the model modification are discussed in the following paragraph. The other obtained hypoplastic constants are shown in Table 1.

Table 1.The parameters of hypoplastic constitutive model for clay with cementation bonds

| N | $\lambda^*$ | $\kappa^*$ | $\varphi_c$ | r | k | A | $s_f$ | R | $\beta_r$ | $\chi$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1.32 | 0.105 | 0.014 | 27.5 | 0.45 | 0.25 | 0.4 | 1 | 0.00012 | 1.05 | 3 |

## 4.2 *Prediction of stiffness at very small strains*

The results of laboratory experiments supplied enough information to propose an expression relating shear stiffness to stress and current state of structure.

Viggiani & Atkinson (1995) proved that shear modulus of clay can be expressed by equation (1), where $G_0$ is initial shear modulus, $p'$ is mean effective stress, $p_r$ is reference pressure (usually taken as 1 kPa), $p_p'$ is the effective stress at the intersection of a swelling line with the normal compression line, thus the ratio $p_p'/p'$ defines overconsolidation ratio. The dimensionless parameters $A_G$, n and m depend on the properties of the soil.

$$\frac{G_0}{p_r} = A_G \left(\frac{p'}{p_r}\right)^n \left(\frac{p_p'}{p'}\right)^m \tag{1}$$

In the case of the cemented kaolin clay the compression line lies above the normal consolidation line (NCL) of pure kaolin clay (Fig. 4). The apparent overconsolidation ratio can be calculated as the ratio $p_e'/p'$, where $p_e'$ is the Hvorslev equivalent pressure determined on the NCL of the cemented kaolin clay, corresponding to the given void ratio e. Further enhancement of the equation (1) was proposed on the basis of the presented laboratory experiments, and it relates the shear stiffness to the changes of the metastable structure. The effect of structure was included by the sensitivity s, the state variable of the model defined as the ratio of the Hvorslev equivalent pressures on the normal compression lines of cemented and pure kaolin clay. The equation (1) was modified in the following way:

$$\frac{G_0}{p_r} = A_G \left(\frac{p'}{p_r}\right)^n \left(\frac{p_e'}{p'}\right)^m \left(\frac{s}{s_f}\right)^l \tag{2}$$

where $s_f$ is the final sensitivity which corresponds to the complete degradation of all cementation bonds.

The proposed equation was verified based on the behaviour of the shear stiffness versus mean effective stress obtained from the measurements of shear wave velocities (Fig. 6). The application of the same parameters $A_G$, n and m for the calculation of the initial stiffness of pure and cemented kaolin clay gives results, which are in a good agreement with the measured data. The calibrated values of the parameters are presented in Table 2.

Table 2. The parameters of hypoplastic constitutive model controlling stiffness at very small strains for clay with cementation bonds

| $A_G$ | n | m | L |
|---|---|---|---|
| 1020 | 0.73 | 0.65 | 0.24 |

The equation (2) was used for modification of the hypoplastic model. The parameters $A_G$, n, m and l replaced the original paremeters controlling the initial stiffness $m_R$ and $m_T$. Mašín (2005) derived a formula for calculation of the initial shear modulus $G_0$ predicted by the hypoplastic model:

$$G_0 = \frac{m_R}{r\lambda^*} p \tag{3}$$

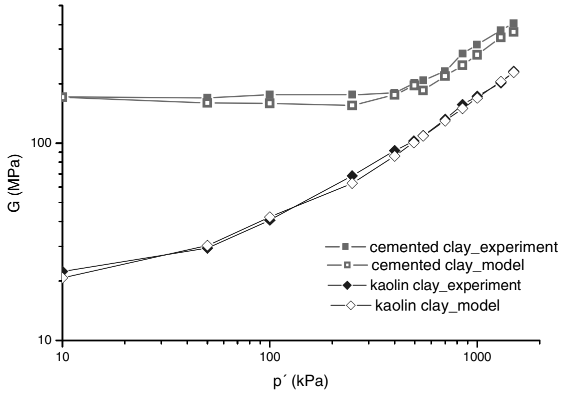

Figure 6. Shear stiffness versus mean effective stress – model and experiment (using bender elements)

Equation (3) can be combined with Equation (2) in such a way that the parameter m$_R$ of the original model becomes a variable calculated from:

$$m_R = r\lambda^* A_G \left(\frac{p'}{p_r}\right)^{(n-1)} \left(\frac{p_e'}{p'}\right)^m \left(\frac{s}{s_f}\right)^l \tag{4}$$

The final model is effective in reproducing the stiffness at very small strains (Fig. 6), the decrease of stiffness with increasing strain (Fig.7) and also the behaviour in large strain range (Fig. 4, 5).

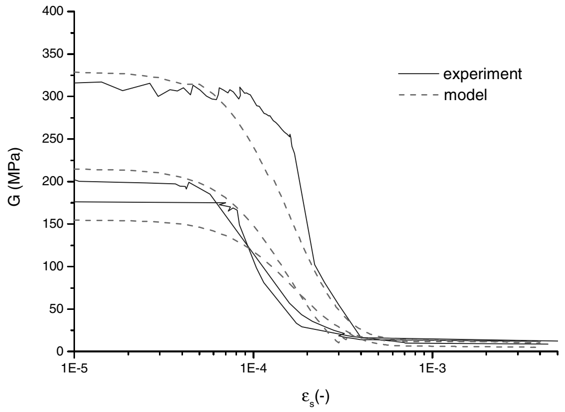

Figure 7. Dependency of shear stiffness on shear strain

## 5 CONCLUSIONS

The mixture of kaolin clay with 4% of Portland cement was found as a suitable material for a laboratory simulation of cementation bonds in fine-grained soil. It was shown that the large-strain behaviour observed is comparable to the behaviour known from experiments on natural clays.

Measurements by bender elements and local LVDT transducers indicate a significant influence of structure on G$_0$. The influence can be quantified by a new formula relating G$_0$ to the mean stress, overconsolidation and structure.

The equation was incorporated into the hypoplastic model for metastable structure. The modified model simulates accurately the mechanical behaviour of cemented clay in the isotropic and one-dimensional compression and in triaxial shear, from the very-small to large strain range.

## ACKNOWLEDGMENTS

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## REFERENCES

Burland, J.B. 1990. On the compressibility and shear strength of natural clays. *Géotechnique* 40(3): 329-378.

Cotecchia, F. & Chandler, R.J. 1997. The influence of structure on the pre-failure behaviour of a natural clay. *Géotechnique* 47(3): 523-544.

Cotecchia, F. & Chandler, R.J. 2000. A general framework for the mechanical behaviour of clays. *Géotechnique* 50(4): 431-447.

Feda, J. 1995. Behaviour of a cemented clay. *Can. Geotech. J.* 32: 899-904.

Feda, J. 2002: Simulation of behaviour of undisturbed specimens of soil. *Geotechnika* 1: 3-5.(in Czech)

Horpibulsuk, S., Bergado, D.T. & Lorenzo, G.A. 2004. Compressibility of cement-admixed clays at high water content. *Géotechnique* 54(2): 151-154.

Leroueil, S. & Vaughan, P.R. 1990. The general and congruent effects of structure in natural soils and weak rocks. *Géotechnique* 40(3): 467-488.

Mašín, D. 2005. A hypoplastic constitutive model for clays. *International Journal for Numerical and Analytical Methods in Geomechanics* 29(4): 311-336.

Mašín, D. 2007. A hypoplastic constitutive model for clays with meta-stable structure. *Can. Geotech. J.* 44: 363-375.

Niemunis, A., Herle, I. 1997. Hypoplastic model for cohesionless soils with elastic strain range. *Mech. of Cohes.-Frict. Mater.* 2: 279 – 299.

Puppala, A.J., Kadam, R., Madhyannapu, R.S. & Hoyos, L.R. 2006. Small-strain shear moduli of chemically stabilized sulfate-bearing cohesive soils. *J. Geotech. Geoenviron. Eng.* 132(3): 322-336.

Viggiani, G. & Atkinson, J.H. 1995. Stiffness of fine-grained soil at very small strains. *Géotechnique* 45(2): 249-265.