Properties of double porosity clayfills and suitable constitutive models
Les équations constitutives de sols de la double porosité

D. Mašín, V. Herbstová & J. Boháč
Department of Engineering Geology, Charles University, Prague, Czech Republic

ABSTRACT

In the Czech Republic, double porosity soils exhibiting large total porosity are abundant in the clayfills of open-pit coal mines. Numerical analyses of such soils require unconventional procedures. The paper presents an approach to the constitutive modelling of double porosity materials. Application of the framework is demonstrated by an enhancement of the Modified Cam clay model. The results of laboratory experiments that have been carried out in the triaxial apparatus are used in calibrating the reference MCC model. The performance of the enhanced model is demonstrated by a parametric study.

RÉSUMÉ

En République Tchèque, les sols à double porosité présentant une porosité totale importante sont fréquents dans les argiles de remplissage de mines de charbon à ciel ouvert. La modélisation numérique du comportement mécanique de tels sols nécessite des procédures non conventionnelles. Cet article présente une approche à la modélisation constitutive des matériaux a double porosité. Cet approche est ici appliqué afin d'améliorer le modèle Modified Cam clay (MCC). Le modèle MCC de référence a été calé à partir de résultats expérimentaux de laboratoire obtenus lors d'essais triaxiaux. On présente les résultats d'une étude paramétrique qui montrent l'amélioration obtenue grâce à l'approche proposée.

1 INTRODUCTION

In the North-Western Bohemia the large-scale open-cast coal mining has resulted in depositing the clayey overburden in non-engineered landfills. The landfills reach depths of several tens of metres, exceptionally even over 100 metres, and occupy about 100 km^2 of land. There is an increasing demand for developing the landfills and also for building traffic infrastructure on them. The site investigation for a motorway that is crossing the landfills was described by Boháč and Škopek (2000), Škopek and Boháč (2004) and by Kurka and Novotná (2003) who also attempted numerical modelling of the landfills' behaviour. Dykast et al. (2003) described long-term deformations of a 130 metres deep landfill supporting a railway, road and also a small river in a steel pipeline.

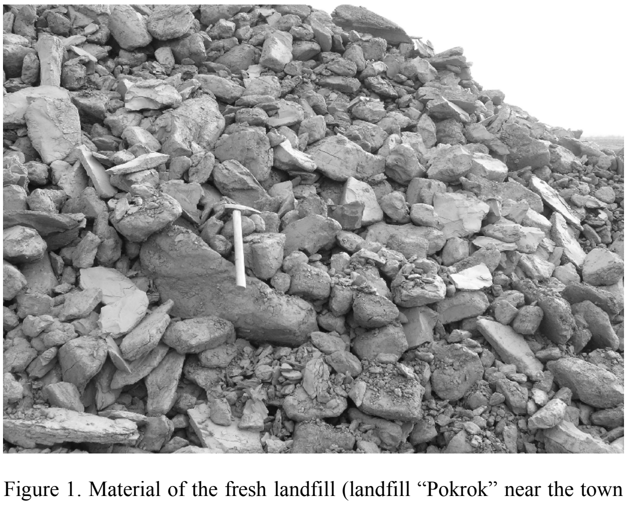

Figure 1. Material of the fresh landfill (landfill "Pokrok" near the town of Bílina, North Bohemia).

2 SOIL PROPERTIES

The clayey overburden soils have been end-dumped without any compaction in the form of blocks and lumps ranging in size from millimetres up to about 0.5 metre. Intragranular porosity of the original overburden is typically ca 40% and the initial intergranular porosity arising from the filling reaches up to ca 45%. The fresh landfills are therefore formed by fragmentary material of high total porosity of up to ca 70% (Fig. 1). Gradually, mainly due to the influence of stress and water (climate, precipitation), the material of originally high intergranular porosity transforms back into more homogeneous soil. Fig. 2 shows the landfill material after about 10 years from filling. Even under very low overburden pressure the material became relatively homogeneous again.

The properties of the undisturbed overburden were studied for example by Feda et al (1995). The sediments mostly consisted of kaolinitic and illitic clays with variable amount of

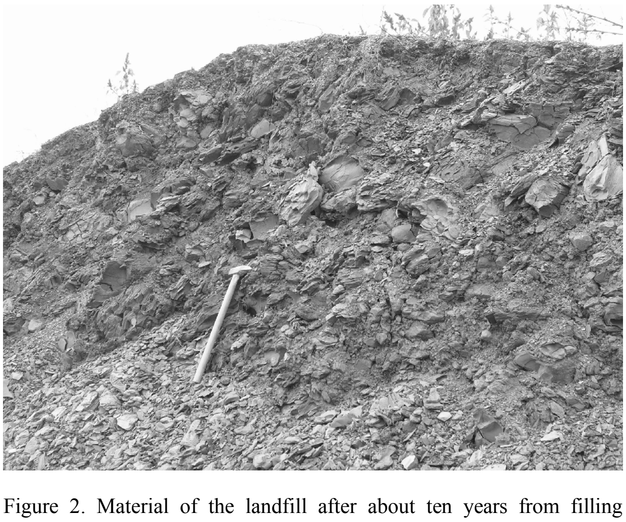

Figure 2. Material of the landfill after about ten years from filling (landfill "Pokrok" near the town of Bílina, North Bohemia).

montmorillonite (up to about 25%). There was no sand fraction present, and practically no layering was found. Plasticity index was typically of about 40% (up to about 100% in the case of high amount of montmorillonite) and liquid limit ranged from 60% to 90%.

When submerged in the laboratory, the specimens from the present sites decompose quickly into fine grained particles. No cementation of the undisturbed overburden was detected. Specimens from some sites however decomposed into angular fragments of the size of millimetres to centimetres, which indicated variable existence of water resistant structural bonds (Feda et al., 1995).

The properties of the material of the landfills were studied for example by Feda et al (1994), Boháč and Škopek (2000) and Boháč et al. (2003). Feda (1998) analysed the individual mechanisms associated with the transformation of the 'granular' material of the fresh landfills back into the fine grained soil. From the studies it follows that the most important feature of the clayfills is their double porosity. Further the behaviour of the upper part of the fills is influenced by the partial saturation and by continuous air phase.

There were several attempts to model the behaviour of the landfills numerically. Doležalová and Kořán (2002) used DEM in 2D in determining the properties of a “synthetic” numerical material of the fragmented claystone. They claimed a good match between the deformation parameters derived in the laboratory and by the numerical modelling by PFC^{2D}. Appropriate continuum-based numerical analyses have been limited due to the lack of constitutive models that would be able to catch the features of the double porosity material.

3 FRAMEWORK FOR CONSTITUTIVE MODELLING OF DOUBLE POROSITY MATERIALS

The proposed approach to the continuum-mechanics constitutive modelling of double-porosity materials is based on the critical state soil mechanics. It is similar to concepts proposed by Lagioia and Nova (1995) for cemented granular materials, Nova et al. (2003), who included effects of chemical degradation of inter-particle bonds, Cecconi et al. (2002) for materials with crushable grains, and, e.g., Rouiana and Muir Wood (2000) for structured clays. Large amount of experimental data on the mechanical behaviour of natural clays was compiled by Cotecchia and Chandler (2000). Their “sensitivity framework” is used as a basis for developments presented in this paper.

The approach is demonstrated in Fig. 3, which shows the results of an ideal isotropic compression test on a double-porosity material and the “reference” material (material of the clay lumps). Due to its internal structure, the double-porosity material may exist at higher overall void ratios, than the reference material. The shaded area in Fig. 3 represents the contribution of the intergranular porosity. As confirmed by field observations summarised in Sec. 2, plastic straining of the double-porosity material causes gradual closure of inter-granular voids (Fig. 3), and the mechanical behaviour of the composite soil then starts to be governed by the mechanical properties of the reference material.

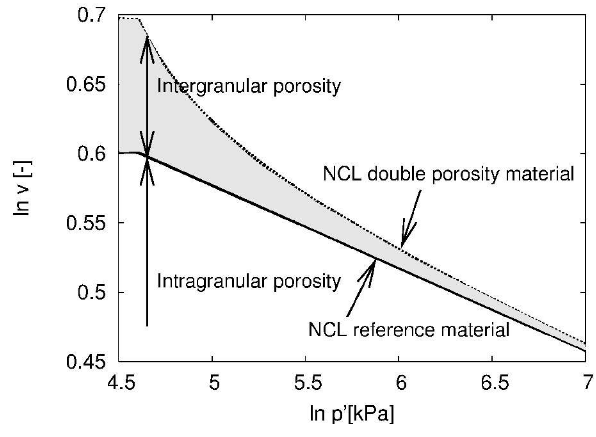

Figure 3. Ideal isotropic compression test on a double-porosity material and a reference material.

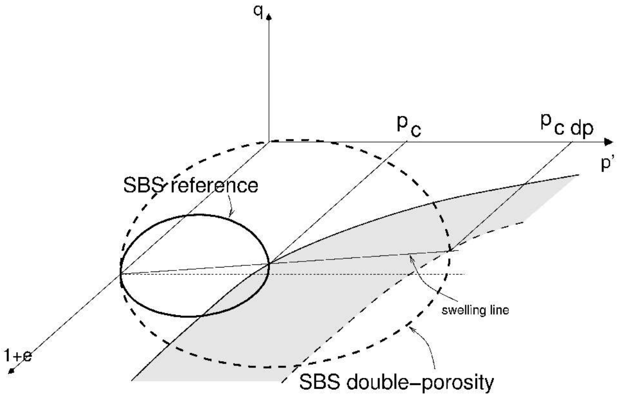

Figure 4. SBS of double-porosity and reference materials

The described behaviour may be generalised for the general stress-porosity space, as shown in Fig. 4. In terms of the critical state soil mechanics, the double-porosity material has a larger state boundary surface (SBS), defined as the boundary of all possible states of a soil in the stress-porosity space, than the reference material. The size of the SBS of the double-porosity material is not constant and depends on the additional state variables, which represent current degree of structuring.

Because detailed information on the shape of the SBS of the double-porosity material are not available yet, we assume geometric similarity of the SBS of the reference and double-porosity materials. Following Cotecchia and Chandler (2000), the size of SBS of a double-porosity material is determined by a single state variable, sensitivity (s), defined as the ratio of the sizes of SBS of the double-porosity and reference materials:

$$s=\frac{p_{c\,dp}}{p_c} \tag{1}$$

where p_c and $p_{c\,dp}$ are defined in Fig. 4.

4 CONSTITUTIVE MODEL

4.1 *Reference constitutive model*

Modified Cam clay (MCC) model (Roscoe and Burland, 1968) has been chosen as the constitutive model for the reference material (material of clay lumps). Since several versions of this model are described throughout literature, it is useful to point out some details of the mathematical formulation of the model adopted in the present research:

Normal compression and swelling lines are assumed to be linear in the lnp' : ln(1+e) space (Butterfield, 1979). Isotropic normal compression line has the following formulation:

$$\ln(1+e)=N-\lambda^{*}\ln p' \tag{2}$$

where e is void ratio, p' is mean stress and N and λ^* are material parameters.

Elastic stiffness matrix is defined by constant shear modulus G and bulk modulus K given by

$$K=\frac{p'}{\kappa^{*}} \tag{3}$$

Swelling index κ^* is a material constant. The shape of the yield and plastic potential surfaces and the isotropic hardening law corresponds to the standard formulation of the MCC model given, e.g., in Muir Wood (2004). The model requires five constitutive parameters, namely M, N, λ^*, κ^* and G, the state of the soil is defined by Cauchy stress σ' and void ratio e.

MCC model has been chosen as the reference model, since it is simple to calibrate and predicts qualitatively correctly stress-dilatancy behaviour of fine grained soils, which is important for the present work. It must be, however, recognized that the model is unable to predict such important phenomena of the soil behaviour as stiffness nonlinearity of overconsolidated soil, high elastic stiffness, the influence of the stress level on elastic shear modulus, influence of recent history of deformation etc.

4.2 *Constitutive model for double-porosity material*

MCC model has been enhanced by the effects of double-porosity structure in a similar way, as proposed for structured clays by Liu and Carter (2002) and Baudet and Stallebrass (2004).

Isotropic normal compression line of the "Structured Modified Cam clay" (SMCC) model reads

$$\ln(1+e)=N+(\lambda^*-\kappa^*)\ln s-\lambda^*\ln p' \tag{4}$$

Sensitivity s is the only additional state variable (see Sec. 3). Its evolution is governed by the following equation (Rouiana and Muir Wood, 2000):

$$\dot{s}=\frac{-k}{\lambda^*-\kappa^*}(s-1)\dot{\epsilon}^d \tag{5}$$

where k is a model parameter controlling the rate of degradation of the intergranular porosity and $\dot{\epsilon}^d$ is the rate of damage strain, defined by an equation similar to Rouiana and Muir Wood (2000):

$$\dot{\epsilon}^d=\sqrt{(\dot{\epsilon}_v^p)^2+\frac{A}{1-A}(\dot{\epsilon}_s^p)^2} \tag{6}$$

where $\dot{\epsilon}_v^p$ is plastic volumetric strain, $\dot{\epsilon}_s^p$ is plastic shear strain and A is a parameter which defines relative importance of plastic volumetric and shear strains on degradation of the intergranular porosity. For most soils $0<A<0.5$, with $A=0.5$ the formulation by Baudet and Stallebrass (2004) is recovered.

SMCC model requires two additional parameters (k and A) and one additional state variable (s), as compared to the reference constitutive equation.

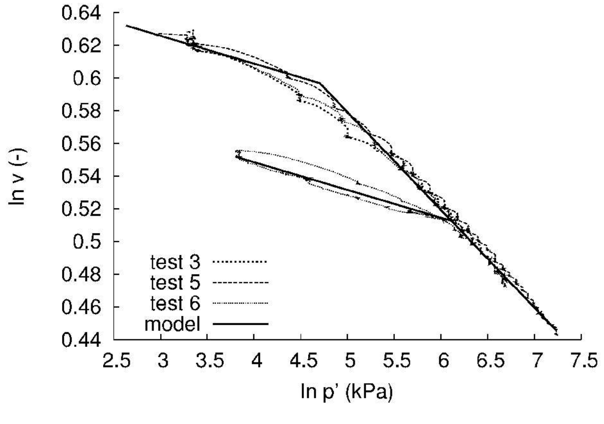

Figure 5. Isotropic loading and unloading test results with model predictions

5 LABORATORY EXPERIMENTS ON THE REFERENCE MATERIAL

Undisturbed samples were taken from the clayfill about 20 years old (clayfill "5 květen" near the town Ústí nad Labem, North Bohemia). The tested kaolinitic-illitic clay is of high plasticity, I_p = 29-34%, w_L = 58-62%. Figure 5 shows the compressibility curves of the specimens obtained in isotropic compression.

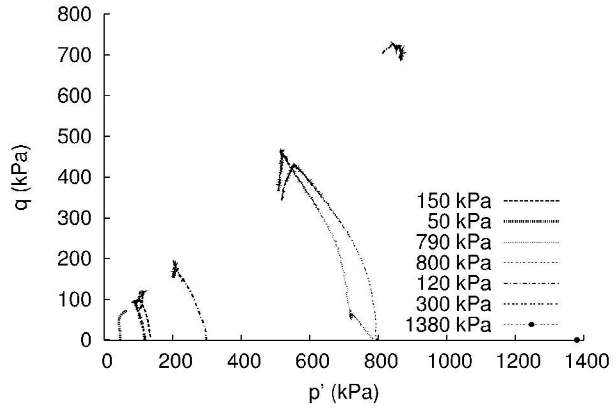

Figure 6. Stress paths of the triaxial tests.

Stress paths obtained from undrained triaxial tests on undisturbed specimens are in Fig. 6. More details about experimental investigation of the reference material are given in Herbstová et al. (2005).

6 CALIBRATION OF THE REFERENCE CONSTITUTIVE MODEL

Reference constitutive model (MCC) was calibrated using laboratory experiments on the material of clay lumps described in Sec. 5.

Parameters N, λ^* and κ^* were calibrated using isotropic loading and unloading tests (Fig. 5), parameter M was calculated from the critical state friction angle in triaxial compression.

The calibration of the parameter G is rather subjective, due to the shortcomings of the MCC model summarised in Sec. 4.1. The value of parameter G was found by means of a parametric study, calibration curves are shown in Fig. 7. Discrepancy again stems from the shortcomings of the MCC model (Sec. 4.1).

All parameters of the reference constitutive model are summarised in Tab. 1.

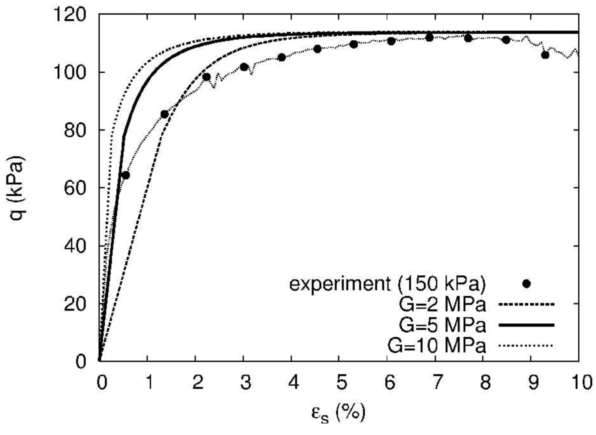

Figure 7. Parametric study performed for calibration of parameter G

Table 1. Parameters of the MCC model for the reference material

G	M	λ^*	κ^*	N
5 MPa	1.07	0.0598	0.017	0.878

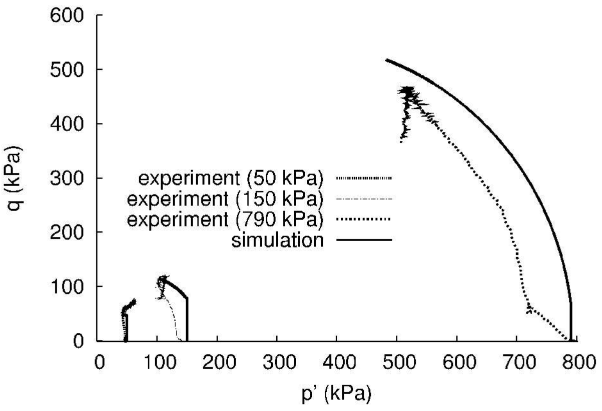

Figure 8. Experimetntal and simulated stress paths on undrained shear tests on reference material

The stress paths of three undrained triaxial tests on reference material are shown in Fig. 8. Predictions by the MCC model are also included in Fig. 8.

7 PERFORMANCE OF THE MODEL FOR DOUBLE-POROSITY MATERIALS

SMCC model requires specification of two additional material parameters, A and k. Before detailed laboratory data on double-porosity materials are available only qualitative performance of the SMCC model can be demonstrated.

Fig. 9. shows the influence of parameter k on the shape of the isotropic normal compression line of the material with intergranular porosity. The initial value of sensitivity has been set to s=5, the value of parameter A does not influence model predictions for tests with zero plastic volumetric strains (see Eq. (6)).

8 CONCLUSIONS

The paper presents a continuum approach to constitutive modelling of double-porosity materials, developed to predict the behaviour of clayey landfills in North-Western Bohemia. The double-porosity material has a structure, caused by inter-granular voids, which is progresively lost due to plastic straining and weathering. Structure degradation due to plastic straining was incorporated into the constitutive model for double-porosity materials, which is based on the Modified Cam clay model.

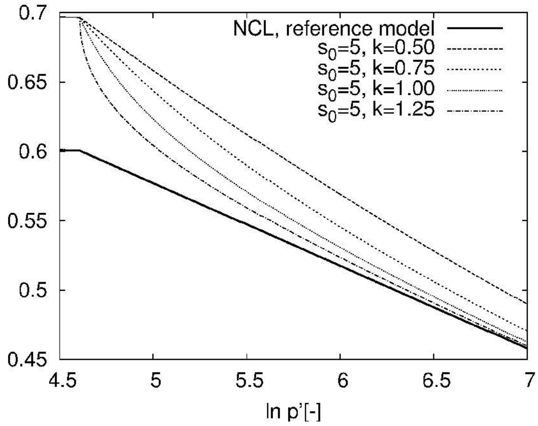

Figure 9. Qualitative influence of parameter k on predicted isotropic normal compression line of soil with double-porosity structure

Modified Cam clay model was calibrated using experimental data on the “reference” material of clay lumps. A good qualitative agreement between observed and predicted response was demonstrated. Performance of the model for double-porosity materials was studied by simulating an ideal isotropic compression test with different values of model parameters. Their calibration with respect to experimental data on double-porosity materials is the subject for future work.

ACKNOWLEDGEMENT

The autors are grateful for the financial support by the research grants GAUK 331/B-GEO/PřF and GAČR 205/03/1467.

REFERENCES

Baudet, B. and Stallebrass, S.E. 2004. A constitutive model for structured clays. *Geotechnique* 54, No 4, 269-278.

Boháč, J., Herbstová, V., Kořán, P. and Vogel, T. 2003. Properties of unsaturated colliery clayfills in North-Western Bohemia. *Proc. 13ECSMGE*, Prague, Vol. 1, 33-36.

Boháč, J. and Škopek, J. 2000. A motorway over clayey fills of open pit coal mines. *Geoeng 2000*, Melbourne.

Butterfield, R. 1979. A natural compression law for soils (an advance on e-ln p'). *Geotechnique* 29, No 4, 469-480.

Cecconi, M., DeSimone, A., Tamagnini, C. and Viggiani, G.M.B. 2002. A constitutive model for granular materials with grain crushing and its application to a pyroclastic soil. *Int. J. Num. Anal. Meth. Geomech.* 26, 1531-1560.

Cotecchia, F. and Chandler, J. 2000. A general framework for the mechanical behaviour of clays. *Geotechnique* 50, No 4, 523-544.

Doležalová, M. and Kořán, P. 2002. Micromechanical modeling of a dump material. *The International Journal of Geomechanics*. Vol. 2, No. 1, 47-74.

Dykast, I:, Pěgřímek, R., Pichler, E., Řehoř, M., Havlíček, M and Vaníček, I. 2003. Ervenice corridor – 130 m high spoil heap from clayey material – with transport infrastructure on its surface. *Proc. 13 ECSMGE*, Vol. 4, 57 – 76.

Feda, J. 1998. Fragmentary clay – a difficult waste material. *Engineering Geology* 51, 77-88.

Feda, J., Herle, I. Kurka, J. and Zavoral, J. 1995. Neogene clays and claystones of Western Bohemia. *Proc. 10ECSMFE*, Vol. 7, 69-74.

Feda, J., Herštus, J., Herle, I and Šťastný, J. 1994. Landfills of waste clayfill material. *Proc. 13ICSMFE*, New Delhi, 1623-1628.

Herbstová, V., Mašín, D., and Boháč, J. 2005. Parameters for non-engineered colliery clayfills. *Proc. Int. Conf. on Problematic Soils GEOPROB 2005*. Famagusta, Cyprus (in preparation).

Kurka, J. and Novotná, I. 2003. The D8 motorway over old tailing dam and spoil heaps near Ústí nad Labem. *Proc. 13 ECSMGE*, Vol. 4, 131 – 146.

Lagioia, R. and Nova, R. (1995) An experimental and theoretical study of a behaviour of calcarenite in triaxial compression. *Geotechnique* 40, No. 4, 633-648

Liu, M.D. and Carter, J.P. 2002. A structured Cam Clay model. *Can. Geotech. J.* 39, 1313-1332.

Muir Wood, D. 2004. *Geotechnical modelling*. Spon Press, London, 488p.

Nova, R., Castellanza, R. and Tamagnini, C. 2003. A constitutive model for bonded geomaterials subject to mechanical and/or chemical degradation. *Int. J. Num. Anal. Meth. Geomech.* 27, 705-732.

Roscoe, K.H. and Burland, J. 1968. On the generalised stress-strain behaviour of wet clay. Pp 535-609 in Heyman, J. and Leckie, F. A. (Eds), *Engineering plasticity*, Cambridge, CUP.

Rouiana, M. and Muir Wood, D. 2000. A kinematic hardening constitutive model for clays with loss of structure. *Geotechnique* 50, No 2, 153-164.

Škopek, J. and Boháč, J. 2004. Site investigation by trial embankments. *Proc. ISC-2*, Porto 2004, Vol. 2, Millpress, Rotterdam, 1571-1574.