Proceedings of International Conference on Problematic Soils, 25-27 May 2005, Eastern Mediterranean University, Famagusta, N. Cyprus

Parameters for non-engineered colliery clayfills

V. Herbstová[1], D. Mašín[1], J. Boháč[1]

[1]Department of Engineering Geology, Charles University, Prague, Czech Republic

Abstract

The material of the recent clayey landfills is suggested to be analysed by a constitutive model based on the Modified Cam clay enhanced by the sensitivity framework. Such a model allows for modelling the degradation of the intergranular porosity of the double porosity soils. Older clayfills however are supposed to be described well by the Modified Cam clay model, possibly with considering creep behaviour. The paper presents triaxial and oedometer laboratory tests on undisturbed saturated samples of the clayfill material, used for calibration of the constitutive model.

Keywords: *clayfill, double porosity, constitutive model, parameters*

1 Introduction

In the North-Western part of the Czech Republic, the open-cast brown coal mining resulted in large non-engineered landfills of the overburden Tertiary clays and claystones. In total almost 100 km^2 of land are covered by the landfills of different age, at different stage of reclamation. Due to the demand for land the landfills are often considered for development. The typical height of the landfills to be developed is about 30 to 50 metres.

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 montmorillonite (up to about 25%). No sand fraction and no layering was identified. Plasticity index was typically 40%, reaching however 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 some sites decomposed into angular fragments of the size of millimetres to centimetres, which indicated existence of some water resistant structural bonds. However, the soils dealt with in the present study decompose relatively quickly into fine grained particles. No cementation of the undisturbed overburden was detected.

During the open-cast mining the excavated overburden is dumped both outside and inside the excavations in the form of clayey fragments ranging in size from centimetres to tens of centimetres (Figure 1). 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% (Figure 1). Gradually, mainly due to the influence of stress and climate (precipitation), the material transforms back into more homogeneous soil.

Figure 1. Freshly filled overburden material

Freshly dumped overburden material can be seen in Figure 1 ("Pokrok" landfill, near the town of Bílina). In the background the material after about 10 years from filling can be seen. Figure 2 shows a detail of an excavation in the landfill material after about 15 years from filling. Even under low overburden pressure the material became relatively homogeneous again.

Limited information is available on the properties of the material of the landfills (e.g., Feda et al, 1994 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, including collapse and creep. 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. Herštus (1999) reported measuring the depth of the continuous air phase material by injecting compressed air into the landfills and concluded that the depth varied at different landfills between 8 to 22 metres. The unsaturated state of the soils has not however been considered in analyses to date. Further to insitu CPT and laboratory oedometer testing the deformation properties were investigated by in situ trial loading of the landfills. For example Škopek and Boháč (2004) have analysed settlements of monitored trial embankments and have detected substantial creep deformation in the landfill material in situ.

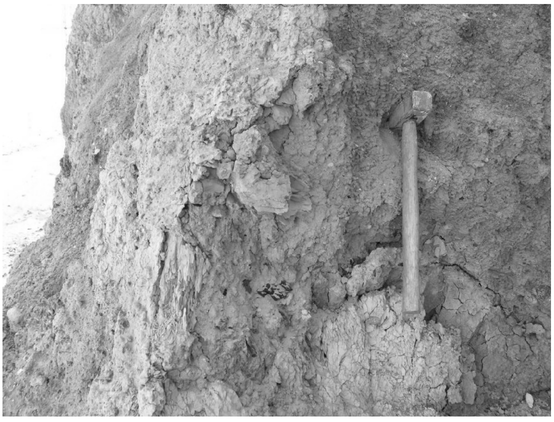

Figure 2. The material 15 years after filling

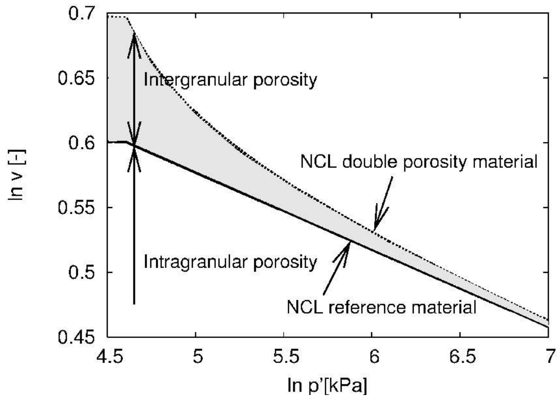

Figure 3. Isotropic NCL of a reference and double-porosity materials

2 Numerical modelling of double porosity materials

Due to the complexity of the internal structure of double-porosity clayfill materials, it is not possible to use standard engineering approaches to predict their behaviour. Realistic predictions require advanced numerical procedures and constitutive models. This paper considers saturated soils only. In reality however there is an unsaturated zone of variable depth in the clayfills and soil suction may need to be taken into consideration.

Doležalová and Kořán (2002) attempted to predict the behaviour of landfills numerically using a distinct element code PFC^{2D}. The claystone fragments were approximated by variably shaped clumps of rigid circular particles, the inter-granular discontinuities were modelled by similar material with reduced contact bonds and stiffness. The model parameters were determined by a trial-and-error procedure. The authors claimed a good match between the modelled and observed behaviour of a dump material. Although the micromechanics approach provided promising results, its practical application is limited due to the still insufficient computer power and difficulties in calibration of model parameters.

To overcome these problems, a new continuum-mechanics based approach to the constitutive modelling of double-porosity materials has been developed by Mašín et al. (2005). In their concept, the behaviour of claystone lumps and a composite double-porosity material is treated independently using a single constitutive model enhanced by the effect of the internal double-porosity structure. The approach is schematically demonstrated in Figure 3, which shows results of an ideal isotropic compression test on a “reference” material of clay lumps and a double-porosity material.

The total porosity of a composite material is made up of the intragranular porosity of the material of clay lumps and intergranular porosity between them (shaded area in Fig. 3). The behaviour of the “reference” material is described using a standard constitutive model for fine-grained soils, the effects of the internal structure are included by an enlargement of the state boundary surface of the composite material, in the similar way to that proposed by Cotecchia and Chandler (2000) for natural clays.

For simplicity, Modified Cam clay model (Roscoe and Burland, 1968) was chosen as a reference constitutive model, which was enhanced by an additional state variable that determines the current degree of structure and a function which specifies its degradation. An application of a more elaborate hypoplastic constitutive model for clays (Mašín, 2005) is expected in the later stages of the research project. This paper summarises laboratory experiments performed to date, which are required for the calibration of the reference constitutive model. The calibration itself is presented in Mašín et al. (2005).

3 Site and sampling

The present research aims at developing constitutive model and parameters for several typical sites of the clayfills of the North-Bohemian open-cast mines. Data from one site only, however, are presented in this paper. The former "5. květen" open-cast mine is located close to the town of Ústí nad Labem (Figure 4). The mine finished its activities in 1965, when the non-engineered restoration of the excavated area with the fragmented clayey overburden was completed. The landfill has been reclaimed by the end of the 20th century and a motorway connecting Prague (CZ) and Dresden (D) is now being built at the site.

The major infrastructure project led to an intensive site investigation directed mainly to CPT and to laboratory compressibility testing. Unfortunately no detailed study into the fundamental properties of the clayfill nor into its numerical modelling has been brought about by the motorway project.

Undisturbed samples were taken from three boreholes using a standard (thick-walled) sampler similar to the U100 of the British practice. However comparison of laboratory oedometer results showed no difference between specimens from the present sampler and from DIN thin-walled sampler (Boháč and Škopek, 2004). The quality of the samples therefore is deemed sufficient for the present laboratory research. The soils of the present site proved to be saturated from very shallow depths (1 m below surface).

The tested soil is kaolinitic-illitic clay of high plasticity CH, I_P=29-34%, w_L=58-62%. There was an admixture of about 25% of sand-sized particles in the soil.

4 Laboratory tests

Samples from shallow depths (1 to 5 metres) were tested in the oedometer and in the triaxial apparatus. Standard oedometers with specimens 100 mm in diameter were used. The triaxial specimens of the diameter of 38 mm were fitted with local submersible LVDT gauges for measuring axial strains at the small range. Standard platens were used without any measures to reduce end friction. All triaxial specimens were isotropically consolidated prior the strain controlled undrained compression with pore pressure measurements at the bottom platen.

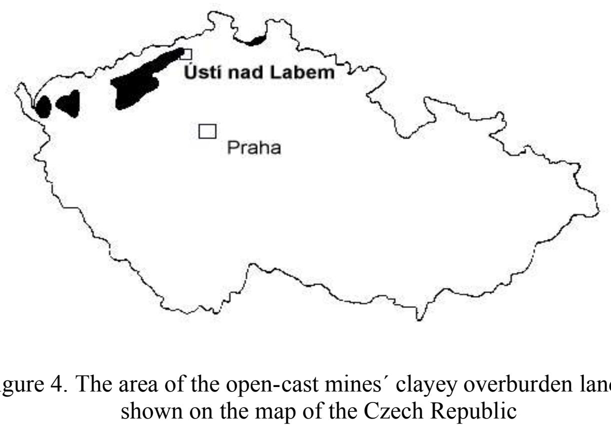

Figure 4. The area of the open-cast mines´ clayey overburden landfills shown on the map of the Czech Republic

4.1 Oedometer and isotropic NCL parameters

1D normal compression lines obtained from oedometer specimens and the isotropic normal compression lines from triaxial tests are used to estimate the compressibility of the clayfill (Figure 5). Both isotropic and oedometer tests give consistent slopes of the compressibility curves. Table 1 gives the slopes of 1D-NCL ranging between λ = 0.08-0.12 in the υ-ln p' graph and compressibility index C_c = 0.20-0.29 when plotting e-log σ_v'. The isotropic NCL from triaxial tests gives λ = 0.1 and C_c = 0.23-0.24. The possible cause of the scatter of the initial porosities of the oedometer specimens is the initial heterogenity of the clayfill material or the disturbance of the samples.

Table 1. Slopes of 1D-NCL and isotropic NCL

Test	λ(-)	Cc (-)
1D-NCL J801 1.3-1.5 m	0.08	0.20
1D-NCL J801 1.7-2.0 m	0.10	0.23
1D-NCL J802 0.5-0.8 m	0.11	0.24
1D-NCL J802 1.2-1.5 m	0.11	0.24
1D-NCL J802 2.1-2.3 m	0.09	0.22
1D-NCL J803 1.8-2.1 m	0.12	0.29
ISO-NCL test 3	0.10	0.23
ISO-NCL test 5	0.10	0.24
ISO-NCL test 6	0.10	0.23

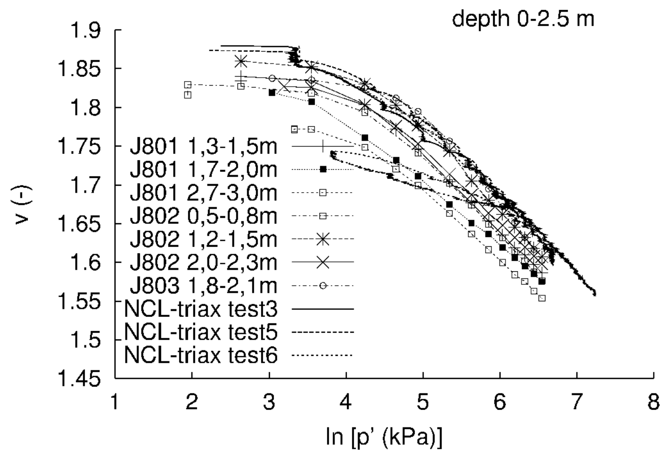

Figure 5. ISO-NCL and 1D-NCL

Table 2. M-C envelope parameters

State	M-C envelope
Peak	c ′= 0 kPa and Φ′= 27°
Post - peak	c ′= 0 kPa and Φ′= 22.4°

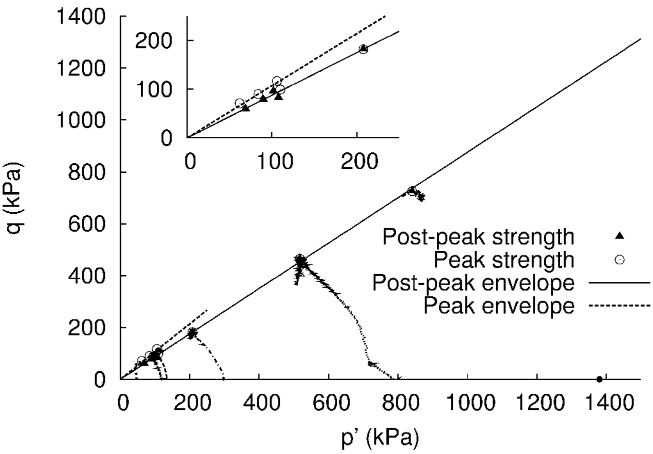

Figure 6. Stress paths and M-C envelopes

4.2 Strength

The test results of the undrained triaxial tests give different Mohr-Coulomb envelopes for the peak and for the post-peak (end of test) state. Burland (1990) and Georgiannou & Burland (2001) suggested that post rupture strength of overconsolidated (brittle) clays lies close to the CSL (intrinsic strength), which was also confirmed for clays similar to the present soil by Boháč et al (1995) and Boháč (1999). The post-peak state was evaluated in order to give an estimation of the critical states, which have not been achieved due to the development of shear planes at all tests. Testing of reconstituted specimens is under way.

It can be seen from Figure 6 that the specimens sheared at lower initial effective stresses are overconsolidated (dry of critical) and give the peak shear strength greater than the post rupture evaluation. The specimens of higher initial effective stresses are consolidated wet of critical and their peak strength is believed an adequate approximation of the critical state.

Figure 7 shows normalized stress strain curves. The specimens compressed to lower initial stresses show marked peaks at the q/p curve, which is not the case of specimens sheared at the higher initial stresses. The drop of the strength at the initial effective stress 790 kPa was probably caused by the shear plane, which developed at the final stage of the experiment.

The critical state parameters derived from reconstituted samples are not available yet, the research is in progress.

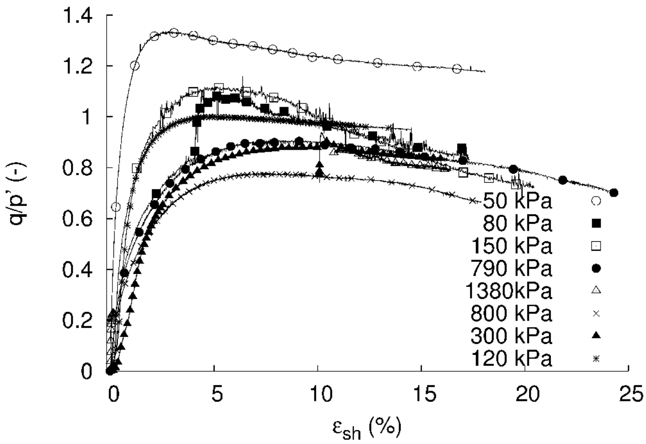

Figure 7. Stress-strain curves

4.3 Stiffness

Stiffness at small strains was measured using a pair of local LVDT gauges mounted vertically to the sides of the specimens. Averages of the readings from the two LVDT gauges were taken for stiffness calculation. In addition, sliding average between adjacent readings was used to smooth out the stiffness curves. The tangent shear modulus (G) was evaluated. The stiffness is strongly dependent on the mean stress and strain levels. The initial stiffnesses are approximately G_0 = 80 MPa and G_0 = 220 MPa for the specimens sheared at initial p' = 120 and 300 kPa, respectively (Figure 8).

The LVDT gauges give reasonable values of stiffness at the shear strains of $\varepsilon_{sh} \geq 0.001\%$. To estimate the stiffness at very small strains bender elements will be used in further research.

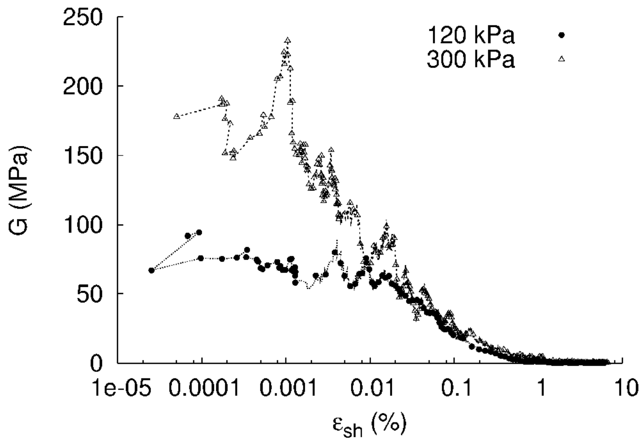

Figure 8. Stiffness

5 Conclusions

Laboratory experiments used in Mašín et al. (2005) for calibration of the Modified Cam Clay model for the landfill material have been described. Oedometric and isotropic compresion tests give consistent slopes of compressibility curves. Two different Mohr-Coulomb strength envelopes for the peak and for the post-peak state were estabilished by triaxial shear testing. At the shear strains of $\varepsilon_{sh} \geq 0.001\%$ the stiffness was measured using the local LVDT gauges mounted to the specimens.

Acknowledgements
The authors are grateful for the financial support by the research grants GAUK331/B-GEO/PřF and GAČR 205/03/1467.

References

Boháč, J. 1999. Strength and deformations of Brno Clay (in Czech). XI. Int. Scient. Conf., Techn. Univ. VUT, Brno, 33-36.

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

Boháč, J., Feda, J., Herle, I. and Klablena, P. 1995. Properties of fissured Brno clay. Proc. 11ECSMFE, Danish Geotechnical Society, Bulletin No. 11, Vol. 3, 19-24.

Boháč, J., Škopek, J. 2004. A spoil heap compressibility determined by trial embankments and by oedometer tests (in Czech). Proc. 32. Int Conf. "Zakládání Staveb", Brno, ČGtS, ISBN: 80-7204-356-0, 300-305.

Burland, J.B. 1990. On the compressibility and shear strength of natural clays. Géotechnique 40, No 3, 329-378.

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

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

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

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

Georgiannou, V. N. & Burland, J. B. 2001. A laboratory study of post-rupture strength. Géotechnique 51, No. 8, 665-675

Herštus, J. 1999. A landfill site investigation for placing a waste disposal (in Czech). Report for Research Grant GACR 103/99/0753, June 1999, Reat, Prague, 9 pp, 8 fgs.

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

Mašín, D. 2005. A hypoplastic constitutive model for clays. International Journal for Numerical and Analytical Methods in Geomechanics (accepted for publication)

Mašín, D., Herbstová, V., Boháč, J. 2005. Properties of double porosity clayfills and suitable constitutive models Proc. 16 ECSMGE, Osaka, Japan, in press.

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

Roscoe, K.H., 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.