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Numerical analysis of a test embankment on soft ground using an anisotropic model with destructuration

Marcin Cudny
Heiko P. Neher
Institute of Geotechnical Engineering, University of Stuttgart, Germany

ABSTRACT: The behaviour of a test road embankment constructed on soft soil deposits at Haarajoki, Finland is simulated with a multi-laminate constitutive model accounting for structural anisotropy and destructuration effects. Structural anisotropy is achieved by directional distribution of the state variables which are responsible for the bonding of natural soft soil material. The numerical calculations are completed with a finite element method program capable to perform coupled static/consolidation analysis of soils. Problems related to the initiation of *in situ* stress state, conditions of preconsolidation, as well as difficulties linked to estimation of the model parameters are discussed in this report. Despite simple assumptions concerning field conditions and non-viscous formulation of the constitutive model, the obtained final results are of a sufficient accuracy for geotechnical practice.

1 INTRODUCTION

In 1997 the Finnish National Road Administration has organised an international competition to calculate and predict the behaviour of a road embankment at Haarajoki. The embankment is founded on soft soil deposits which are typical for the region, FinnRA (1997). These deposits are characterised by a high degree of anisotropy and natural inter-particle bonding. The embankment was constructed in 1997 and the competition is already closed. Nevertheless, available data concerning the general *in situ* behaviour of the embankment and results of the associated laboratory tests are very useful for the validation of different modelling methods. Results of some finite element analyses have been already published in the literature, e.g. Aalto (1998) and Näätänen (1998). However, the scope of the presented analysis is not to compete with other modelling approaches but to validate and investigate the possibilities of a multi-laminate constitutive model for natural soft soils, presented in the affiliated workshop paper by Cudny (2003).

A key feature of the applied multi-laminate model is the introduction of a spatially distributed anisotropic overconsolidation. In the model, overconsolidation is directly related to the degree of bonding of soil fabric. For this reason, the characteristic mechanical process of destructuration of natural soft soils is combined with changes of strength anisotropy.

Instead of refining ground layers by different values of material parameters, the main soft soil deposit is simulated with the same *intrinsic* constants. Only the initial values of state variables representing bonding and overconsolidation are varying with depth. Intrinsic parameters are estimated on the basis of available laboratory tests results and parameter sets previously used in other calculation studies. Throughout the paper all stress quantities are effective.

2 EMBANKMENT AND GROUND CONDITIONS

The longitudinal and cross sections of the Haarajoki test embankment are shown schematically in Figure 1. The embankment is 3 m high and 100 m long. The width at the top is 8 m. In the longitudinal direction of the embankment half of the soft ground deposits were improved by the installation of vertical drains. A geotextile reinforcement is applied at the bottom of the embankment. In the current study only the part of the embankment without ground improvement (cross section 35840) will be analysed.

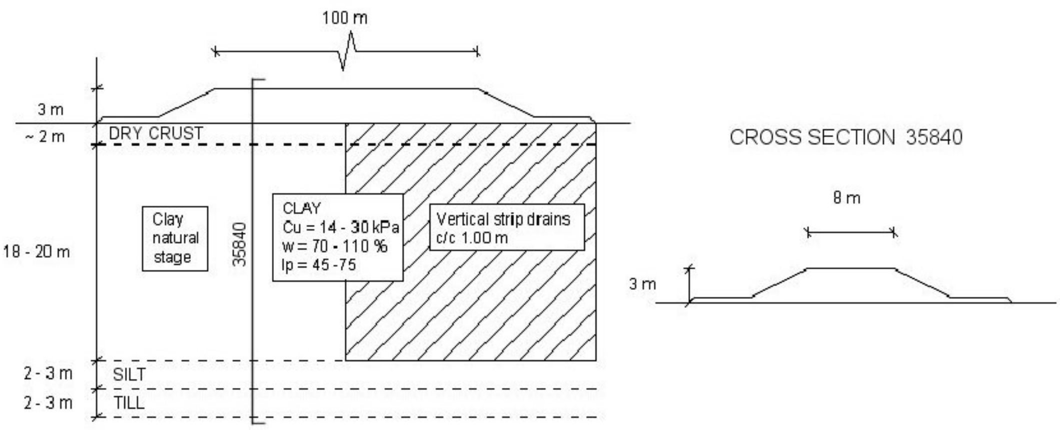

Figure 1. Longitudinal and cross sections of the Haarajoki test embankment. Cross section 35840, where no vertical drains were applied, is analysed.

The construction of the embankment was done in 0.5 m high stages using a gravel fill with a density of 21 kN/m^3. The construction was completed within 3 weeks.

The ground water table is located at surface level and excess pore pressures of –3 to 10 kPa have been measured in the deposit before the construction started. The soft ground consists of a shallow 2 m thick layer of highly overconsolidated crust laying over a 20 m thick soft soil deposit. The most compressible soil layers are located at a depth between 2 and 10 m.

3 INITIAL CONDITIONS AND MATERIAL PARAMETERS

The soft soil deposit at Haarajoki can be classified as a *fat* clay with a content of clay size particles higher than 50 %. The organic content is between 1.4 and 2.2 %. The water content varies between 67 and 112 % and is often higher than the liquid limit. This observations, together with the measured high sensitivity of the clay (up to 50), demonstrate an important mechanical evidence of a fragile inter-particle bonding.

Indeed, results of constant rate of strain (CRS) oedometer compression tests clearly display a destructuration process characteristic for Scandinavian soft clays. Some oedometer compression curves given in the records for the Haarajoki competition materials are shown in Figure 2. The degree of bonding, directly linked with the intensity of destructuration, decreases slowly with depth. However, the degree of bonding appears to be quite high and constant for the shallow layers up to a depth of 5 m. It is important to note that an intrinsic compression index λ^*, defined as the final inclination of compression curves in semi-logarithmic plots, is approximately constant for all layers.

Tab 1. Values of the material parameters

Layer	Depth	γ	φ	c	ψ	ν	κ^*	λ^*	β	Ω_v	a_r	k_x	k_y
	m	kN/m^3	deg	kPa	deg	-	-	-	-	.	-	m/d	m/d
embank.*	-	21	35	3.0	0	0.15	-	-	-	-	-	3.0	1.0
crust	0-2	17	29	7.7	0	0.2	0.004	0.070	0.84	0.6	1.0	6.5e-3	6.5e-3
soft soil	2-22	15	25	5.7	0	0.2	0.012	0.111	0.95	0.6	1.0	7.0e-4	4.5e-4

*) Mohr-Coulomb model is used for embankment material; additional parameter: E=50000 kPa

In the applied multi-laminate model the parameter of initial bonding is imposed separately on every sampling plane and for the cross-anisotropic sediments this distribution is defined as

$$b_0^k = b_0(1+\Omega_{ij}n_i n_j) = b_0\left[1-\frac{\Omega_v}{2}(1-3(n_2^k)^2)\right] \tag{1}$$

where b_0 is an average or isotropic bonding parameter; Ω_v is the vertical component of the deviatoric microstructure tensor Ω_{ij} and $\mathbf{n}^k$ is an unit vector normal to the k-th sampling plane.

The preconsolidation pressure on the k-th sampling plane is defined as

$$\sigma_{np}^k = \sigma_{neq}^{0k}(1+b_0^k) \tag{2}$$

This implies the following maximal and minimal values of preconsolidation pressures on the sampling planes with the vertical and horizontal normal directions respectively

$$\max(\sigma_{np}^k) = \sigma_{neq}^{0k}\left[1+b_0(1+\Omega_v)\right] = \sigma_{np}^v, \quad \min(\sigma_{np}^k) = \sigma_{neq}^{0k}\left[1+b_0\left(1-\frac{\Omega_v}{2}\right)\right] = \sigma_{np}^h \tag{3}$$

Estimation of the parameter b_0 can be carried out by back-calculations of oedometer tests or directly from isotropic compression tests like it is shown in Cudny&Vermeer (2003). The influence of the parameter b_0 on the shape of compression curves is depicted in Figure 2.

Another important problem, when imposing initial conditions, is the proper initiation of the directionally dependent preconsolidation pressure. It has to be noted that the preconsolidation pressure on the horizontal sampling planes can not be equated to the traditional preconsolidation pressure from oedometer tests as yielding observed on the macro level is an integrated plastic behaviour from the sampling planes and occurs progressively. Having estimated the value of b_0 and the initial *in situ* stress state $\boldsymbol{\sigma}^0$, i.e. $\sigma_{yy}{}^0$ and K_0, it can be found that the direct use of the distribution from Equation 2 will result in much higher preconsolidation than this measured or assumed. This happens especially in heavily structured soils. It follows that for the initiation of directional overconsolidation a reduced stress level $\boldsymbol{\sigma}^{0*}$ should be applied keeping the same value of K_0.

In the analysis, first the depth distributions of the mean preconsolidation stress p_p, bonding b_0 and K_0 were chosen as it is shown in Figure 3. Then, in every calculation point the reduced mean stress p_0^* and related stress components, needed for directional distribution, were calculated as

$$p_0^* = \frac{p_p}{1+b_0} \quad \Rightarrow \quad \sigma_{yy}^{0*} = \sigma_n^{0v*} = \sigma_{neq}^{0v} = \frac{3p_0^*}{1+2K_0}, \quad \sigma_{xx}^{0*} = \sigma_n^{0h*} = \sigma_{neq}^{0h} = K_0\sigma_{yy}^{0*} \tag{4}$$

In Equation 4 superscripts v and h are linked with vertical (y) and horizontal (x) directions normal to the sampling planes respectively. Results of the applied procedure are shown in Figure 4.

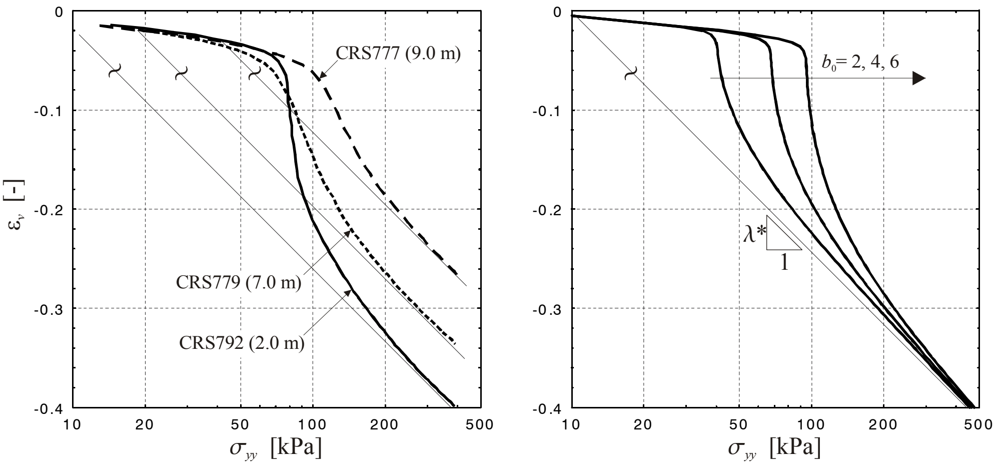

Figure 2. Left: Results of CRS oedometer compression tests on the soft clay from Haarajoki; Right: Influence of the parameter b_0 on the simulated behaviour during oedometer compression.

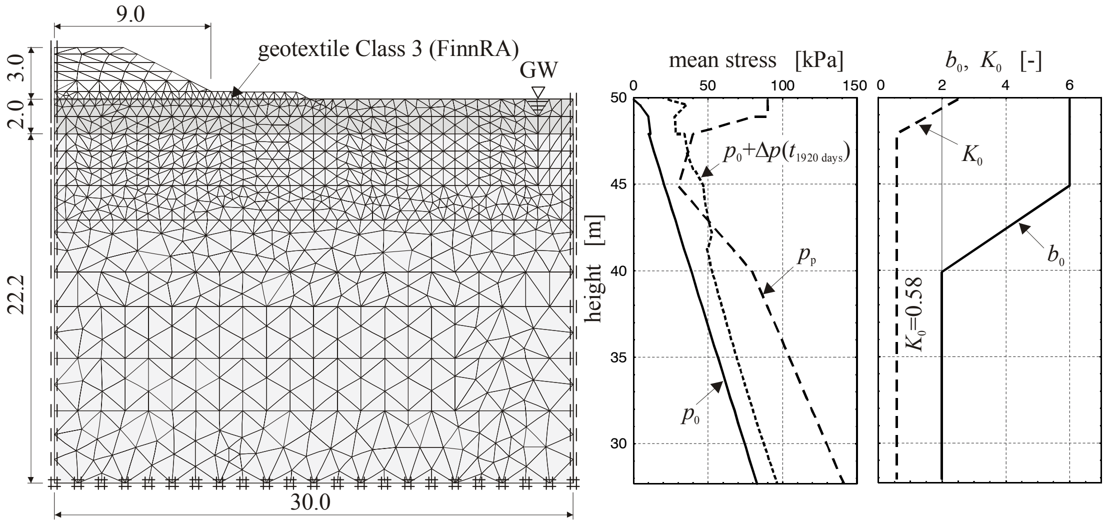

Figure 3. Finite element discretisation of the boundary problem and initial distributions of p_p, K_0 and b_0.

The parameter Ω_v controls the degree of the strength cross-anisotropy in the model. This parameter has an influence on the shape of macro yield surface which results from the multi-laminate model. Its influence is also visible when simulating oedometer compression tests on the vertically and horizontally trimmed samples, see Cudny&Vermeer (2003). Nevertheless, it seems to be impossible to estimate it on the basis of a single standard laboratory test. It was found that choosing values close to the K_0–value brings satisfactory results for soft natural soils. Values of remaining material parameters used in the analysis are listed in Table 1.

4 ANALYSIS AND RESULTS

The implementation of the model and the simulations of the Haarajoki test embankment behaviour have been completed with the 2D finite element code Plaxis, Brinkgreve (2002).

All calculation phases including 6-stages construction of the embankment and consolidation for the period of 1920 days (~5.3 years) have been computed as fully coupled static/consolidation analysis. So-called user defined soil model facility has been used both for the implementation of the constitutive law as well as for imposing the initial stress and the preconsolidation conditions.

The finite element discretisation of the boundary problem is shown in Figure 3. 1847 of 6-noded plane strain elements have been used. Measured and calculated vertical displacements are compared at the centre line and at the points located 4 and 9 m from the centre line. Horizontal displacements are compared along the vertical profile located 9 m from the centre line.

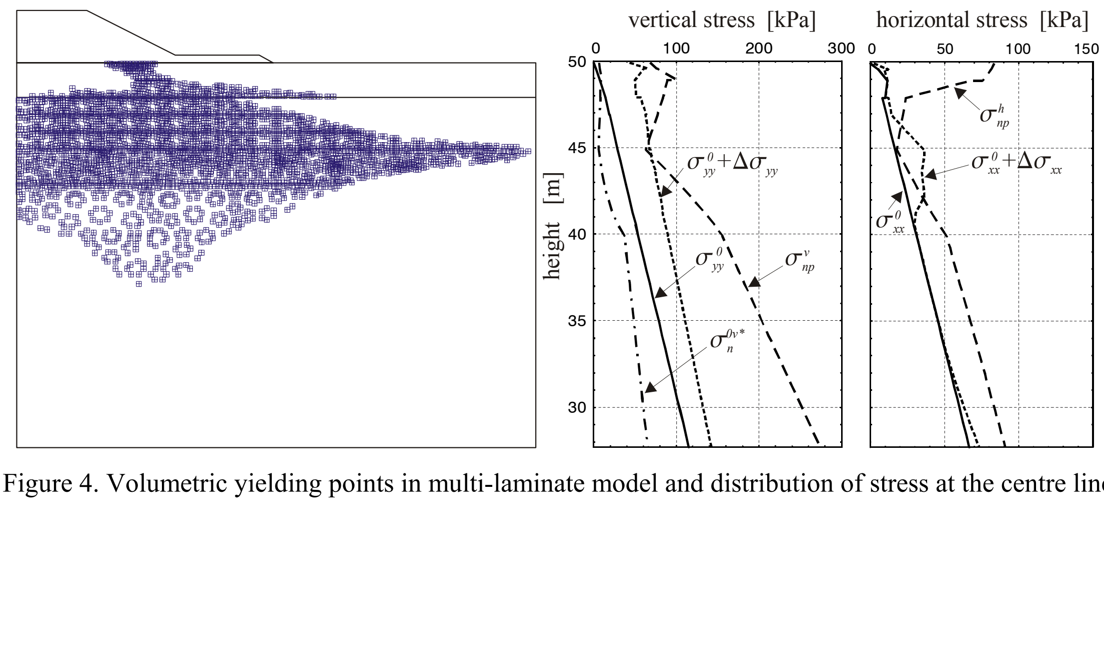

Figure 4. Volumetric yielding points in multi-laminate model and distribution of stress at the centre line

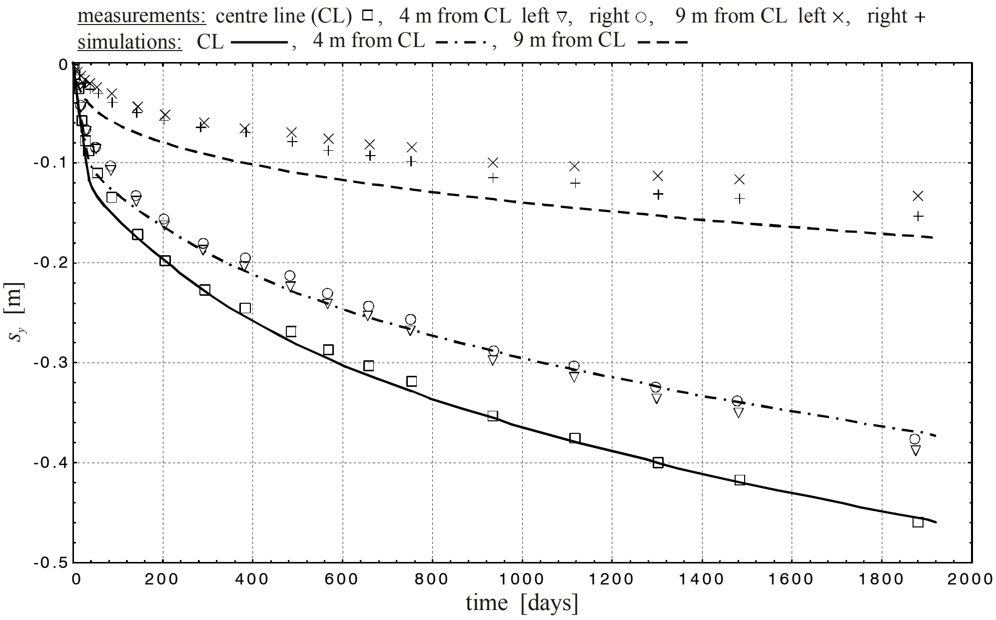

Figure 5. Measured and simulated time-settlement curves for points located at the embankment base.

These comparisons between field and numerical data are presented in Figures 5 and 6. The results obtained for the first set of material parameters and initial conditions were only satisfactory for the vertical displacements. The horizontal deformation was generally overestimated. After small corrections to the depth distribution of the preconsolidation pressure (increasing), very good results have been obtained for the vertical displacements, but the horizontal displacements are still overestimated.

Differences between the measured and simulated horizontal displacements have already occurred in the early construction phase of the emebankment. However, their absolute values have not changed much in the course of consolidation. This would suggest that sources of inaccuracy are linked with the applied isotropic elasticity in the model. The unsound influence of elastic isotropy may be important in the shallow layer of dry crust where the largest stress changes take place.

From the other hand, taking into account assumed simplicity of the initial field conditions and use of average intrinsic parameters for a quite deep soft soils deposit, the final accuracy of the simulations is satisfactory.

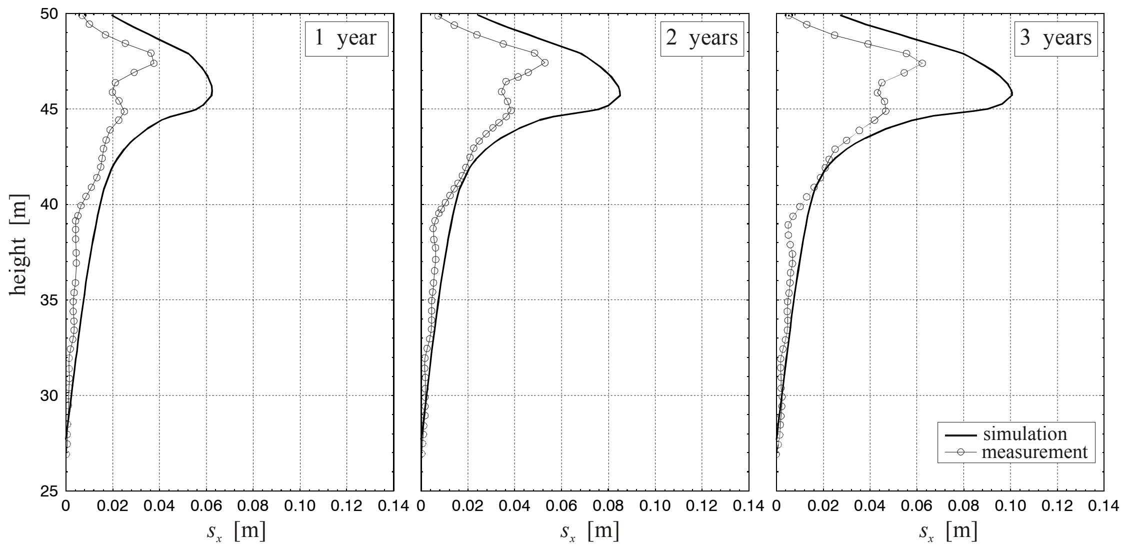

Figure 6. Comparison between measured and simulated horizontal displacements (s_x) and their development during the consolidation for the vertical profile located at the embankment toe.

5 CONCLUSIONS

The behaviour of a trial road embankment founded on soft soil deposits at Haarajoki, Finland has been simulated with a use of finite element analysis. The new multi-laminate constitutive model for soft natural soils, presented in the affiliated workshop paper by Cudny (2003), was applied and validated for this numerical task. The model incorporates structural anisotropy and is able to reproduce the destructuration effects occurring during the primary compression.

Problems related to the initiation of stress states and state variables which control the degree of bonding and preconsolidation are addressed in the paper.

6 ACKNOWLEDGEMENT

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