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PREDICTED AND MEASURED TIME-DEPENDENT BEHAVIOUR OF HIGHWAY EMBANKMENT ON COHESIVE SOIL STRATUM

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Abstract

Highway embankments are important structural elements in modern road infrastructure. If such a construction is built on cohesive low-permeability soils, it is necessary to perform a prediction of long-term settlements and excess pore pressures. The paper presents a numerical analysis of an instrumented embankment constructed in the Czech Republic using the finite element method. Two alternative constitutive models were employed throughout the analysis: standardly used linear elastic perfectly plastic model and elastoplastic model with volumetric and shear hardening with stress-dependent stiffness. A construction sequence was modelled in detail including durations of partial construction stages. Both the settlements of subsoil (in short-term and long-term conditions) and excess pore pressures measured in multiple depths were evaluated and compared with predictions. Results employing a more complex constitutive model show a reasonably good agreement with measurement both in terms of settlements and pore pressures. The application of a perfectly plastic constitutive model leads to an overestimation of settlements.

Keywords: embankment, settlement, excess pore pressures, finite element method, elastoplasticity

1 Introduction

Highway embankments are an integral part of modern road infrastructure. Due to space, ecological and economic constraints, they are often constructed in difficult geological conditions such as on highly compressible and low permeable fine-grained soils. In such cases, it is necessary to predict the development of settlement in time during construction and service life. Furthermore, the embankment construction rate must be controlled to avoid reaching soil shear strength due to the generation of excess pore pressure in the subgrade. Several such cases were reported in the literature ([1], [2]).

The finite element method is a standard computational tool in present geotechnical engineering. However, to obtain an acceptable response of a computational model, the finite element method must be combined with an appropriate constitutive model. Neglect the dependence of soil stiffness on stress and loading regime can lead to the overestimation of resulting displacements. On the other hand, if a pore pressure increase and consequently an effective stress decrease due to the contractant behaviour of fine-grained soil during undrained shearing is not involved in a particular constitutive model, the undrained shear strength might be overestimated.

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The proposed paper presents a back-analysis of highway embankment constructed in the Czech Republic. Apart from the monitoring of subgrade settlement, pore pressures in two boreholes were recorded via piezoelectric pressure sensors. Two alternative constitutive models were employed throughout the analysis: commonly used linear elastic – perfectly plastic model and more advanced elastoplastic constitutive model with shear and volumetric hardening.

2 Description of analysed embankment

The embankment is a part of the highway D47 Hrušov – Bohumín and situated on the west coast of Antošovice Lake. The maximum height and width of the embankment are 14,1 and 94 meters, respectively. The schematic cross-section of the embankment is shown in Fig. 1. The construction period was divided into 5 stages. Geotechnical monitoring consisted of hydrostatic nivelation between the shafts S1 and S2 in 31 points and pore pressure transducers situated in two boreholes (BH1 BH2), .
The top part of the subsoil to a depth of 1,8 m consists of anthropogenic materials. The natural Quaternary cover consists of sandy-silty clay (1,8 – 3,1 m BGL) and fluvial sandy gravel (3,1 – 6,9 m BGL).Stress-deformation behaviour of the embankment is most influenced by Tertiary silty clay with firm to stiff consistency locally with lens of sand. The Quaternary sediments were not involved in the FE model as they were partially excavated and their thickness is small compared to the underlying Tertiary clays. The embankment itself was built from a mixture of tailing material from local mine and a blast furnace slag with the following properties: r_d=2020 – 2100 kg/m^3, k=10^{-2} m/s, j´=37°, c´=3 kPa.

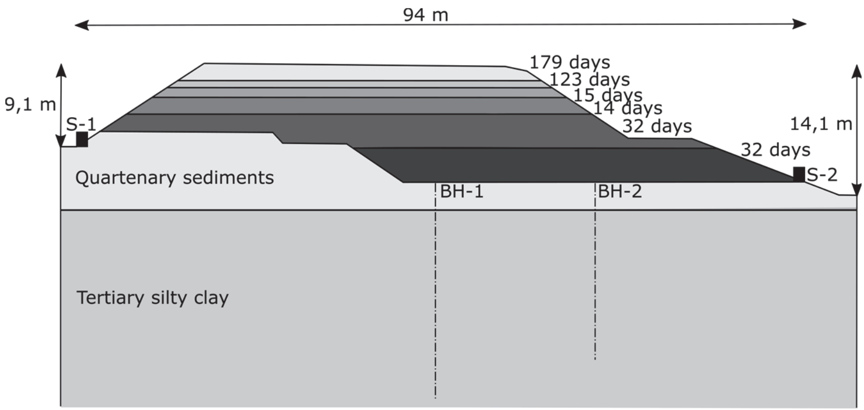

Figure 1 Schematic cross-section of the analysed embankment

3 Applied constitutive models

Two constitutive models were utilized throughout the analysis:
- The linear elastic – perfectly plastic Mohr-Coulomb (MC) model,
- The Hardening Soil model (HS) - elastoplastic model with shear and volumetric hardening [3].

The basic features of both constitutive models are graphically compared in Fig. 2 and Fig. 3 for compression and shear loading, respectively, where NCL is the normally consolidated line and URL is the unloading-reloading line.

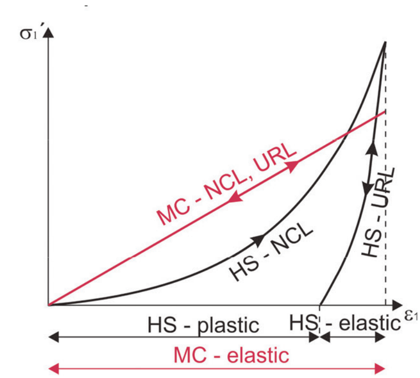

Figure 2 Compression loading – comparison of models

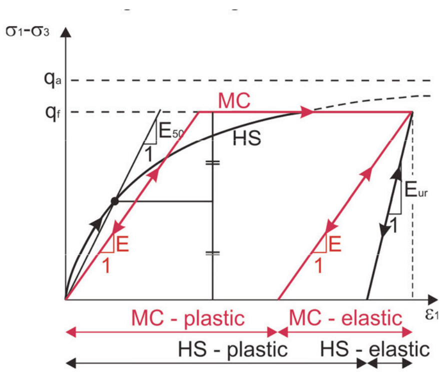

Figure 3 Shear loading – comparison of models

The HS model was based on the non-linear (hyperbolic) relationship between the axial strain and the deviatoric strain ([4], [5]). These non-linear elastic constitutive models were complemented by shear and compression hardening yield surfaces. An important feature of the HS model, especially in displacement analysis, is the stress dependency of soil stiffness. The stress-dependent oedometer stiffness modulus E_{oed} is given by Eq. (1). E_{oed}^{ref} is the reference value of the modulus valid for the reference vertical stress p^{ref}, j and c are the shear strength parameters, m is the power for the stress-level dependency of stiffness and K_0^{nc} is the K_0 value for normal consolidation (K_0^{nc}=1-sinj).

$$E_{oed} = E_{oed}^{ref}\left(\frac{c\cos\varphi - \sigma'_1\sin\varphi}{c\cos\varphi + p^{ref}\sin\varphi}\right)^m \tag{1}$$

4 Simulation of embankment construction

4.1 Computational model and values of input parameters

2D plain strain model was prepared using the Plaxis finite element code [6]. Due to the asymmetrical cross-section, the whole embankment was modelled. The construction sequence, as summarised in Tab. 1, was determined based on geotechnical monitoring report of the analysed cross-section [7]. The embankment height was held constatnt during the stages n. 5, 7, 9 in the particular section as construction activites took place on other sections of the highway. The total time-period analysed in this paper was 1800 days.

Construction of each layer is considered as the plastic stage (without the possibility of excess pore pressure dissipation) followed by consolidation analysis with a duration corresponding to the construction time of a particular sublayer.

Table 1 Construction sequence relevant to the analysed cross-section

ID	Name	Duration [day]
1	Initial (K_o) conditions	-
2	Construction of the 1st sublayer	32
3	Construction of the 2nd sublayer	32
4	Construction of the 3rd sublayer	14
5	Delay	27
6	Construction of the 4th sublayer	15
7	Delay	120
8	Construction of the 5th sublayer	123
9	Delay	151
10	Construction of the final 6th sublayer	179
11	Monitoring after construction	1107

Values of input parameters for the governing geological layer of Tertiary silty clay are summarised in Tab. 2. Poisson´s ratio during primary loading (n) and unloading-reloading (n_{ur}) is used in MC and HS model, respectively. Linear elastic behaviour before failure (MC model) is controlled by the Young´s modulus E_{ref}. Non-linear hyperbiloc stess – strain response during deviatoric loading in case of the HS model is controlled by the secant stiffness E_{50}^{ref} at 50 % of the maximum deviatoric stress.

4.2 Results

Measured and predicted settlement profiles for two various times since the beginning of construction are shown in Fig. 4. Settlements are significantly over-predicted in case of the calculation with the Mohr-Coulomb model in which stress-stiffness dependence is not involved. The prediction with the HS model provides more accurate predictions in terms of the final settlements in both times. However, the prediction underestimates the measured settlements on the left side of the embankment. This might be due to a greater thickness of Quaternary sediments in this particular area.

Table 2 Values of input parameters of Tertiary silty clay

Parameter		MC	HS
$\mathbf{g_{unsat}}$	**[kN/m³]**	**16,21**	**16,21**
g_{sat}	[kN/m³]	19,86	19,86
$k_x=k_y$	[m/day]	5e-4	5e-4
E_{ref}	[MPa]	9,84	
n / n_{ur}^*	[-]	0,35	0,2
E_{50}^{ref}	[MPa]	-	16,08
E_{oed}^{ref}	[MPa]	-	16,08
E_{ur}^{ref}	[MPa]	-	44,66
m	[-]	-	0,55
j´	[°]	24,3	24,3
c´	[kPa]	40,33	40,33

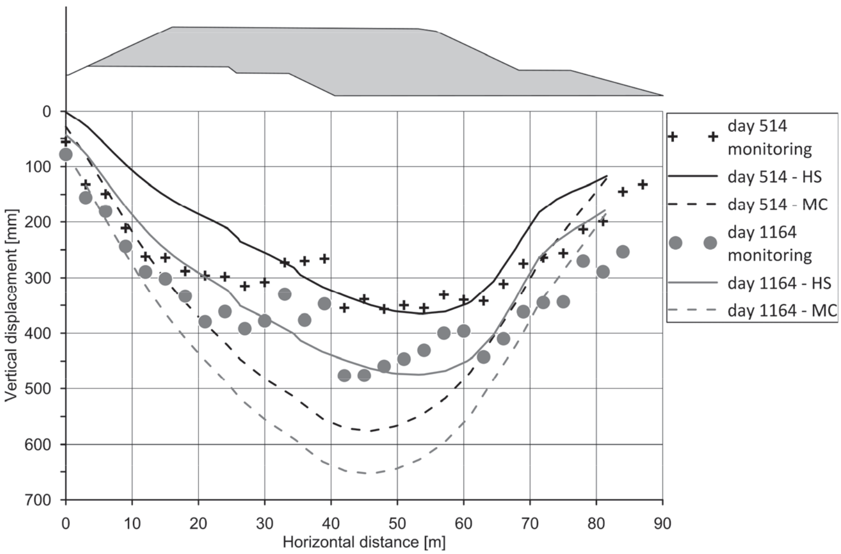

Figure 4 Measured and predicted settlement profiles

Measured and predicted time developments of excess pore pressures are shown in Fig. 5 (V1 – depth 5 m) and Fig. 6 (V2 – depth 10 m). Sharp increases in computed pore pressures arise due to the activation of corresponding sublayers. The times of these peaks approximately correspond to the times when the measured pore pressures increased. However, especially in the last stages which took much longer than the first stages, the activation of the sublayer with zero time interval (plastic analysis) followed by the consolidation analysis resulted in greater differences between the measurements and the predictions. The maximum values of pore pressures computed using the MC model are lower compared to the HS model. The Mohr-Coulomb model behaves elastically until reaching the failure state. In undrained shear loading, when no volume changes are allowed, no excess pore pressures are being generated and effective main stress remains constant.

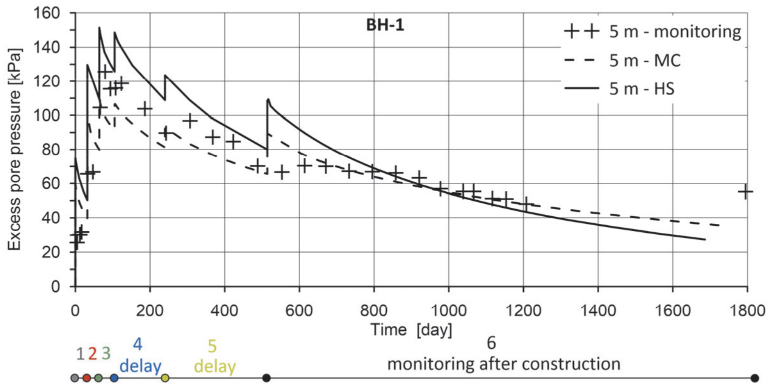

Figure 5 Development of excess pore pressures, borehole V1, depth 5 m

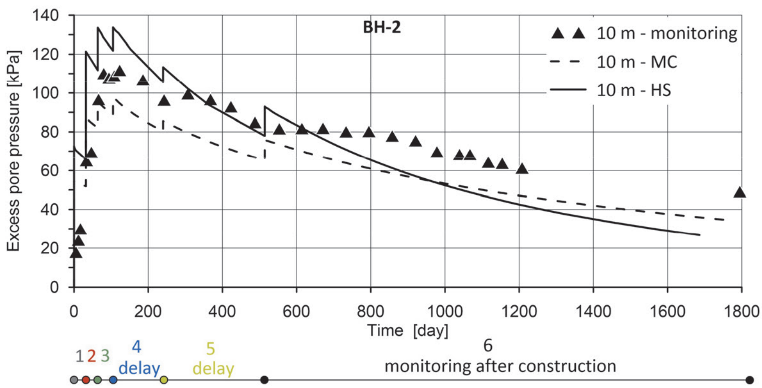

Figure 6 Development of excess pore pressures, borehole V2, depth 10 m

5 Conclusion

The performed analysis demonstrated that the combination of the finite element method and the appropriate material model provide sufficiently accurate embankment settlement prediction even in long-term conditions several years after the construction finished. Sufficient match is reached in case of the ultimate settlement and its position when using HS model. Predicted settlements are slightly underestimated on the left side of the embankment. This might be due to the higher thickenss of the Quaternary sediments in this area which were neglected in the presented analysis. Ignoring the stress dependency of the soil stiffness in case of the MC model leads to a substantial overestimation of predicted displacements. Measured excess pore pressures are predicited with a reasonable accuracy during the first 5 construction periods. However, the rate of the excess pore pressures dissipation during the subsequent monitoring is overestimated, indicating that the permeability of the Tertiary sediments is lower than anticipated.

Acknowledgment

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