NUMERICAL EVALUATION OF ROAD EMBANKMENT SETTLEMENTS

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Abstract: *Settlement of embankments represents a major serviceability concern in the design and operation of road infrastructure. Reliable settlement prediction is essential to estimate pavement deformations and reduce maintenance interventions during the service life of the road. This paper presents a numerical assessment of settlements of road embankments founded on soft overconsolidated soils. The analysis is performed using the finite element method and an advanced constitutive model formulated within the critical state soil mechanics framework. Two documented embankments with available field measurements are analysed to evaluate the applicability of the adopted numerical approach. The results indicate that reliable estimation of embankment settlements can be achieved using numerical analyses combined with constitutive models requiring a limited number of parameters obtained from standard laboratory testing*.

Keywords: *settlement, embankment, numerical analysis, constitutive model.*

NUMERIČKA PROCENA SLEGANJA SAOBRAĆAJNIH NASIPA

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Rezime: *Sleganje nasipa predstavlja jedno od najvažnijih pitanja upotrebljivosti u projektovanju i eksploataciji putne infrastrukture. Pouzdana procena sleganja od posebnog je značaja za predviđanje deformacija kolovozne konstrukcije i smanjenje potrebe za naknadnim intervencijama održavanja tokom eksploatacije saobraćajnice. U radu je prikazana numerička analiza sleganja saobraćajnih nasipa fundiranih na mekom prekonsolidovanom tlu. Analiza je urađena primenom metode konačnih elemenata i naprednog konstitutivnog modela formulisanog u okviru teorije kritičnog stanja tla. U cilju ocene primenljivosti usvojenog numeričkog pristupa razmatrana su dva dokumentovana primera nasipa za koje su dostupna terenska merenja. Rezultati pokazuju da se pouzdana procena sleganja nasipa može postići primenom numeričke analize u kombinaciji sa konstitutivnim modelima uz relativno mali broj materijalnih parametara, koji se mogu odrediti na osnovu standardnih laboratorijskih ispitivanja*.

Ključne reči: *sleganje, nasip, numerička analiza, konstitutivni model.*

1. INTRODUCTION

Settlement prediction is a central issue in the design and performance assessment of road embankments, since excessive vertical deformations may lead to pavement damage, loss of serviceability, increased maintenance demand, and reduced traffic safety [1, 2, 3]. The problem is particularly significant for embankments built on soft clay, where settlement develops progressively under staged loading and is governed by pore-pressure dissipation, stress redistribution, and the compressibility of both the embankment material and the underlying soil [4, 5, 6]. It is important to assess not only the magnitude of settlement, but also its development with time, particularly the portion that may remain after embankment construction.

In engineering practice, settlement has traditionally been estimated by analytical procedures based on one-dimensional consolidation theory and simplified assumptions for stress distribution. They provide a simple and rapid framework for preliminary settlement assessment, but they rely on idealised representations of the problem. Their applicability becomes more limited when the analysis must account for non-uniform geometry, staged construction, and hydro-mechanical response. For such cases, numerical methods offer a more realistic representation of pore-pressure development and dissipation, and time-dependent deformation within the embankment–foundation system [7, 8]. Finite element analysis has become a widely used tool in the study of embankments on soft grounds. Its main advantage is that the embankment, the foundation soil, and the

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construction sequence can be represented within the same computational framework, which makes it possible to follow the development of settlement during loading and subsequent consolidation. At the final stage of the analysis, the numerical procedure should be validated by comparison between the calculated results and field measurements of settlement and excess pore-water pressure.

The reliability of the predicted response depends strongly on the constitutive models adopted for the analysis. In practical applications, however, the choice of a constitutive model is not governed only by its theoretical capabilities, but also by the availability and reliability of the input material parameters. In many engineering problems, the range of laboratory and field data is limited, which restricts the use of more advanced models whose parameters cannot be identified with sufficient confidence. Within the framework of critical state soil mechanics, the Modified Cam Clay (MCC) model has long served as a reference constitutive model in geotechnical numerical analysis due to its clear mechanical basis and relatively small number of material parameters [9, 10]. The model has also provided the basis for a number of subsequent developments, particularly in the analysis of clay behaviour beyond the range of normally consolidated and lightly overconsolidated states. One such development is the Hardening State Parameter (HASP) model, introduced to improve the representation of overconsolidated clay behaviour while preserving the simplicity of the MCC framework [11-15].

This study presents a finite element analysis of settlement in road embankments founded on soft overconsolidated soils. Particular attention is given to the influence of the constitutive model on the predicted response. The analyses are based on two well-documented embankment case histories reported in the literature. The first part of the paper provides a brief outline of the governing equations and basic features of the HASP constitutive model. The second part presents the numerical results and compares the predicted evolution of settlement with the corresponding field measurements obtained over time.

2. HASP CONSTITUTIVE MODEL

The HASP model is an advanced constitutive model. The formulation combines the simplicity of Cam Clay-type models with a bounding-surface approach. A key feature of the model is a hardening rule defined in terms of state parameters [16]. The state parameter describes the position of the current soil state relative to the critical state line at the same mean effective stress. It is expressed as the difference between the current specific volume and the corresponding value on the critical state line, Figure 1a. In this way, both density state and stress level are included in the description of soil behaviour.

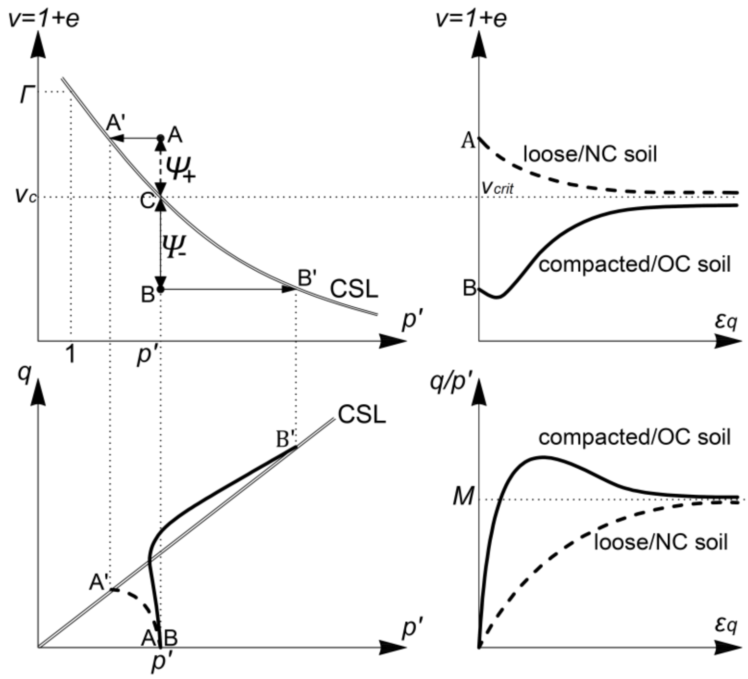

Figure 1. *State parameter concept*

Positive values correspond to loose or normally consolidated soil, whereas negative values correspond to compacted or overconsolidated soil, Figure 1a. These states are associated with different deformation trends during shearing. Loose or normally consolidated soil tends to contract as the critical state is approached. Compacted or overconsolidated soil may initially contract, Figure 1b, but with continued shearing it tends to dilate and may reach a peak strength followed by softening, Figure 1d. The state parameter therefore provides a convenient description of the initial soil state and its influence on the subsequent stress–strain response.

The yield and bounding surfaces (Figure 2) are defined by:

$$F(p', q, \theta) = \left(\frac{q}{M(\theta)}\right)^2 + p'(p' - p'_0) \tag{1}$$

$$\bar{F}(\bar{p}', \bar{q}, \theta) = \left(\frac{\bar{q}}{M(\theta)}\right)^2 + \bar{p}'(\bar{p}' - \bar{p}'_0) \tag{2}$$

where p'_0 is the maximum mean effective stress defining the size of the yield surface; $M(\theta)$ is the slope of the critical state line in the q-p' plane, expressed as a function of the Lode angle θ:

$$M(\theta) = X \cdot (1 + Y \cdot sin\, 3\theta)^Z \tag{3}$$

This form is introduced to account for the effect of the intermediate principal stress and to reproduce the variation of shear strength in the deviatoric plane, including the differences between triaxial compression, triaxial extension and plane strain conditions. The constants X, Y and Z control the shape of the yield and bounding surfaces in the deviatoric plane.

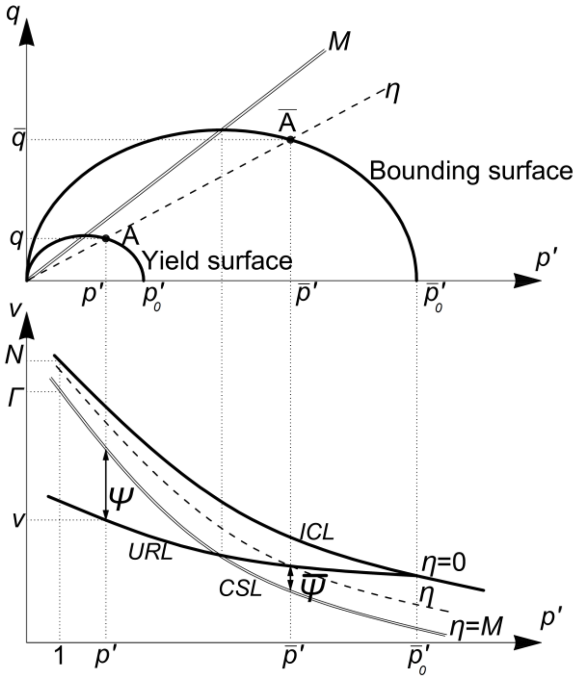

Figure 2. *HASP constitutive model*

The constitutive relations of the HASP model are:

$$\begin{Bmatrix} d\varepsilon_v \\ d\varepsilon_q \end{Bmatrix} = \begin{Bmatrix} \frac{1}{K} + \frac{\lambda-\kappa}{vp'\omega}\frac{M^2-\eta^2}{M^2+\eta^2} & \frac{\lambda-\kappa}{vp'\omega}\frac{2\eta}{M^2+\eta^2} \\ \frac{\lambda-\kappa}{vp'\omega}\frac{2\eta}{M^2+\eta^2} & \frac{1}{3G} + \frac{\lambda-\kappa}{vp'\omega}\frac{4\eta^2}{(M^2+\eta^2)(M^2-\eta^2)} \end{Bmatrix} \begin{Bmatrix} dp' \\ dq \end{Bmatrix} \tag{4}$$

where $\eta = q/p'$ is the stress ratio, v is specific volume, λ is the slope of the virgin compression line (VCL), κ is the slope of a swelling line (URL) in the $v - lnp'$ plane. The hardening coefficient ω is defined in terms of the state parameter at the current stress point Ψ, the state parameter at the conjugate point on the bounding surface $\bar{\Psi}$ (Figure 2), and the overconsolidation ratio R, as:

$$\omega = \left(1 + \frac{\bar{\Psi} - \Psi}{\bar{\Psi}}\right) R \tag{5}$$

$$\Psi = v + \lambda\, ln\, p' - \Gamma \tag{6}$$

$$\bar{\Psi} = (\lambda - \kappa)\, ln\left(\frac{2M^2}{M^2 + \eta^2}\right) \tag{7}$$

where Γ is specific volume at $p' = 1\,kPa$ on the critical state line. The overconsolidation ratio R, relating the current stress point to its conjugate point on the bounding surface, is defined as:

$$R = \frac{\bar{p}'}{p'} = \frac{\bar{q}}{q} = exp\left(\frac{\bar{\Psi} - \Psi}{\lambda - \kappa}\right) \tag{8}$$

The HASP model is formulated using a non-associated flow rule, since the plastic potential and the yield surface have different forms in the deviatoric plane. The model requires the following input parameters: the consolidation parameters λ and κ, the drained strength parameters M_C and M_E, Poisson's ratio μ, and the initial void ratio e_0. Here, M_C and M_E denote the slopes of the critical state line in the q–p' plane for triaxial compression and extension, respectively, whereas e_0 defines the initial soil state. The HASP model is implemented in the finite element software PLAXIS 2D through the UDSM formulation using an explicit modified Euler scheme with automatic substepping and error control as the stress integration procedure [13].

3. NUMERICAL ANALYSIS

Two embankments with well-documented case histories in the literature are selected for numerical analysis. Both embankments were heavily instrumented. The first embankment (Embankment A) is a highway embankment built on the soft deposits of Ariake clay near Saga City, in Japan [5]. The cross section of embankment with soil layers and positions of measuring instruments are given in Figure 3.

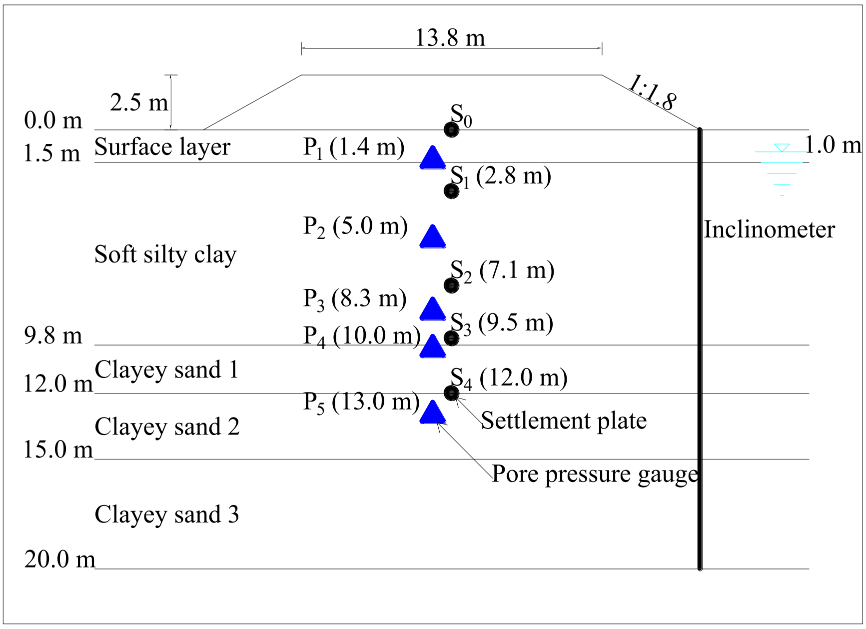

Figure 3. *Cross section of the Embankment A with soil layers and position of measuring instruments*

The second embankment (Embankment B) is located in Australia, near coastal town of Balina in New South Wales. Embankment was built as an upgrade of the Pacific Highway [6]. Foundation soil consists of an estuarine clay deposit occupying the floodplain near the mouth of Richmond River. Cross section of embankment with soil layers and position of measuring instruments are given in Figure 4.

Numerical predictions of settlements were conducted using plane strain fully coupled flow-deformation finite element analysis in PLAXIS 2D. In all numerical predictions, the horizontal displacements were fixed at the left, right and bottom boundaries. Vertical displacements were fixed at the bottom boundary. The drainage boundaries differed between embankments. The drainage boundaries must represent the actual drainage conditions at the site. For Embankment A, drainage boundaries were ground surface and the bottom boundary, while for Embankment B, they were ground surface and right boundary. All other boundaries were impermeable.

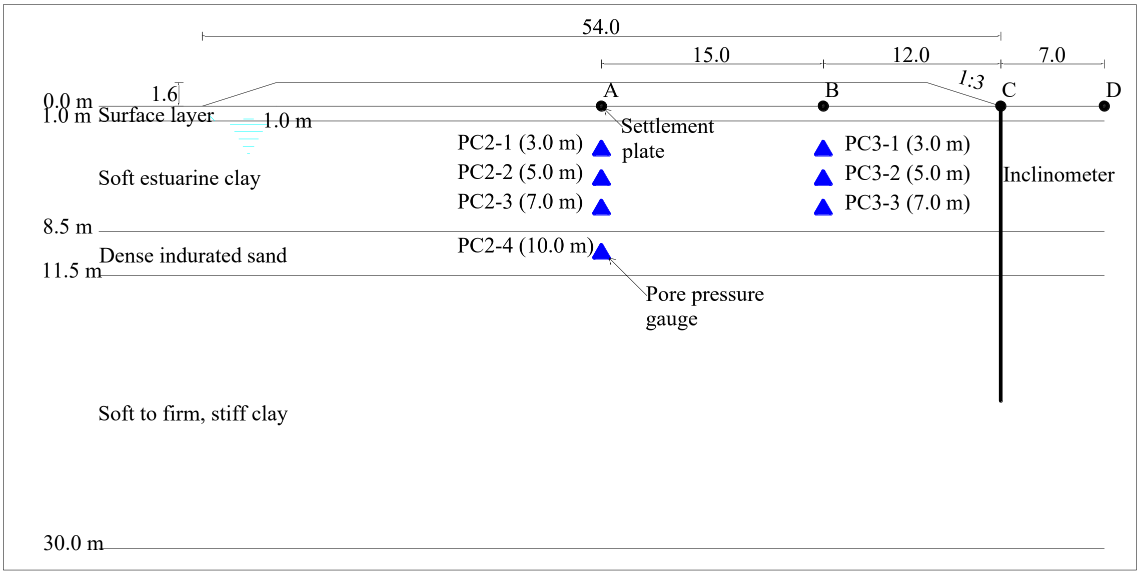

Figure 4. *Cross section of the Embankment B with soil layers and position of measuring instruments*

The construction of embankments was modelled as the first stage of the numerical calculation. The construction of Embankment A lasted 50 days, while the construction of Embankment B lasted 69 days. The second stage consisted of consolidation analysis continued up to four years after the end of embankment construction. Duration of second stage corresponds to the duration of measurements. All results are given in terms of days from the day zero – start of embankment construction.

In order to compare the predictions of the MCC and HASP constitutive models, the clay layers were represented by both models. Mohr-Coulomb (MC) model was used to represent the behaviour of embankments and sand layers. The numerical and soil model parameters for Embankment A and Embankment B are given in Table 1 and Table 2, respectively. The overconsolidation of clay layers is defined by overconsolidation ratio (OCR). Other numerical model parameters, like horizontal and vertical hydraulic conductivities and the coefficients of earth pressure at-rest for normally consolidated and overconsolidated soils can be found in [4].

Table 1. *Numerical and soil model parameters for Embankment A*

Depth $[m]$	Soil strata	MCC Parameters			HASP Parameters		μ	e_0	OCR
		κ $[-]$	λ $[-]$	M $[-]$	M_C $[-]$	M_E $[-]$	$[-]$	$[-]$	$[-]$
0.0-1.5	Surface clay layer	0.025	0.25	1.6	1.6	1.04	0.15	1.50	1.8-5.0
1.5-4.0	Soft silty clay	0.107	1.07	1.6	1.6	1.04	0.15	3.71	1.5-1.8
4.0-6.0		0.119	1.19					2.88	1.3-1.5
6.0-8.0		0.084	0.84					2.72	1.2
8.0-9.8		0.066	0.66					1.91	1.3
9.8-12.0	Clayey sand	MC parameters: $E' = 7500\ kPa$ $c' = 20\ kPa$ $\phi' = 35°$ $\psi = 5°$					0.15	0.80	1.0
12.0-15.0		MC parameters: $E' = 20000\ kPa$ $c' = 20\ kPa$ $\phi' = 35°$ $\psi = 5°$						0.80	1.0
15.0-20.0		MC parameters: $E' = 37500\ kPa$ $c' = 20\ kPa$ $\phi' = 35°$ $\psi = 5°$						0.70	1.0
Embankment		MC parameters: $E' = 5000\ kPa$ $c' = 20\ kPa$ $\phi' = 35°$ $\psi = 0°$					0.30	0.80	1.0

Table 2. *Numerical and soil model parameters for Embankment B*

Depth $[m]$	Soil strata	MCC Parameters			HASP Parameters		μ	e_0	OCR
		κ $[-]$	λ $[-]$	M $[-]$	M_C $[-]$	M_E $[-]$	$[-]$	$[-]$	$[-]$
0.0-1.0	Surface clay layer	0.011	0.078	1.33	1.33	1.07	0.3	0.80	6.0-11.0
1.0-2.5	Soft clay	0.02	0.587	1.07	1.07	0.86	0.33	2.29	4.5-6.0
2.5-3.5									2.2-4.5
3.5-8.5									1.5-2.2
8.5-11.5	Dense sand	MC parameters: $E' = 26672\ kPa$ $c' = 5\ kPa$ $\phi' = 30°$ $\psi = 0°$					0.28	0.65	1.0
11.5-15.0	Firm clay	0.009	0.134	1.11	1.11	0.89	0.3	1.69	1.0
15.0-20.0									1.0
20.0-30.0									1.0
Embankment		MC parameters: $E' = 5000\ kPa$ $c' = 10\ kPa$ $\phi' = 28°$ $\psi = 0°$					0.30	0.80	1.0

The settlement-time curves for settlement plates S_0 and S_1 are given in Figure 5 and Figure 6, respectively. Both soil models predict the end of primary consolidation and flattening of settlement-time curves four years after the beginning of construction. Both soil models predict similar values of settlement during embankment construction. After the end of construction, MCC soil model considerably underpredicts the settlements, while HASP soil model predicts settlement quite well, with little deviation from measured values.

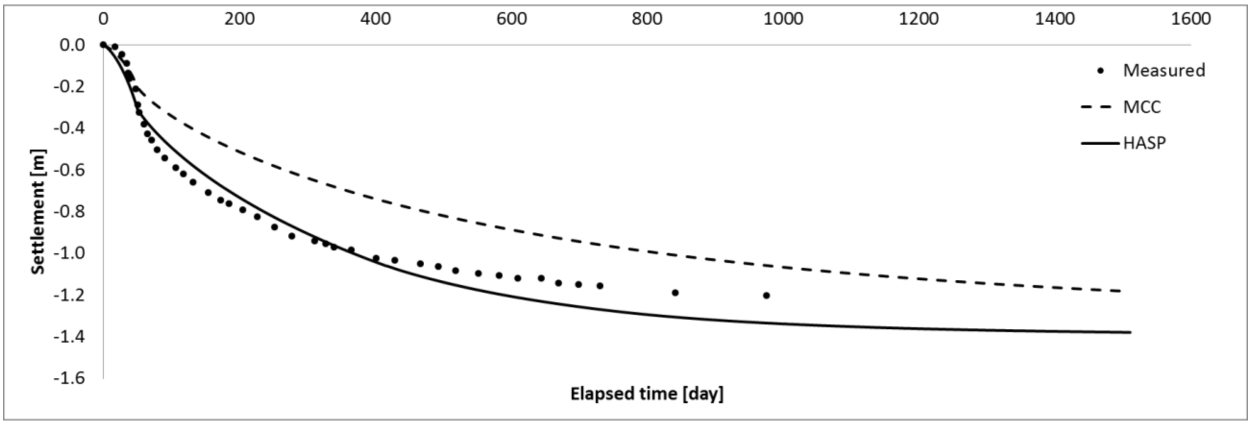

Figure 5. *Comparison of settlement-time curves for Embankment A – Point* S_0

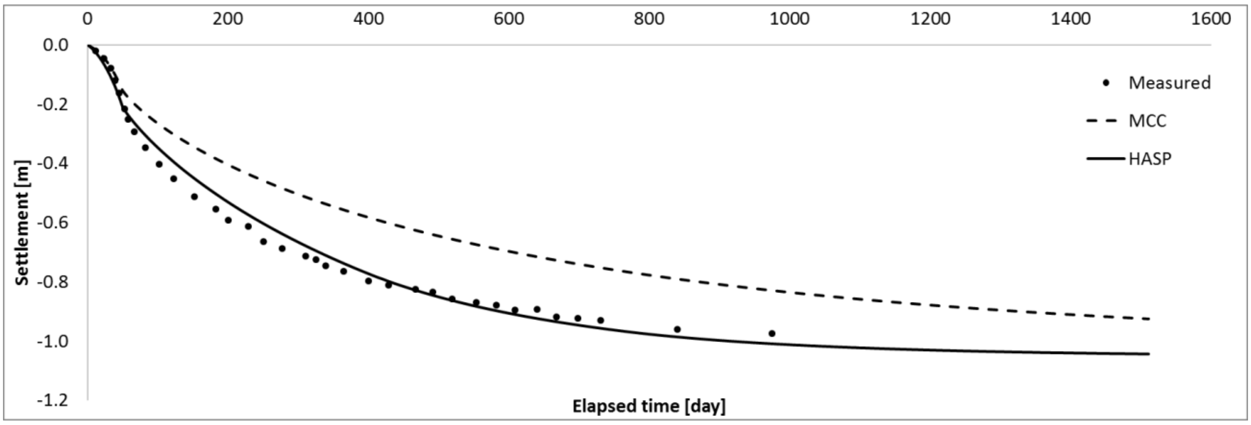

Figure 6. *Comparison of settlement-time curves for Embankment A – Point* S_1

The settlement-time curves for settlement plates located on the surface of terrain – point at the centre of the Embankment B and point 15 m from the centre of the Embankment B are given in Figure 7 and Figure 8, respectively. The settlement behaviour of Embankment B is similar to the behaviour of Embankment A. The MCC model significantly underpredicts the settlements when compared to measured values. The HASP model gives satisfactory prediction for point at the centre of embankment, while it moderately overpredicts the settlement at the point located 15 m from the centre.

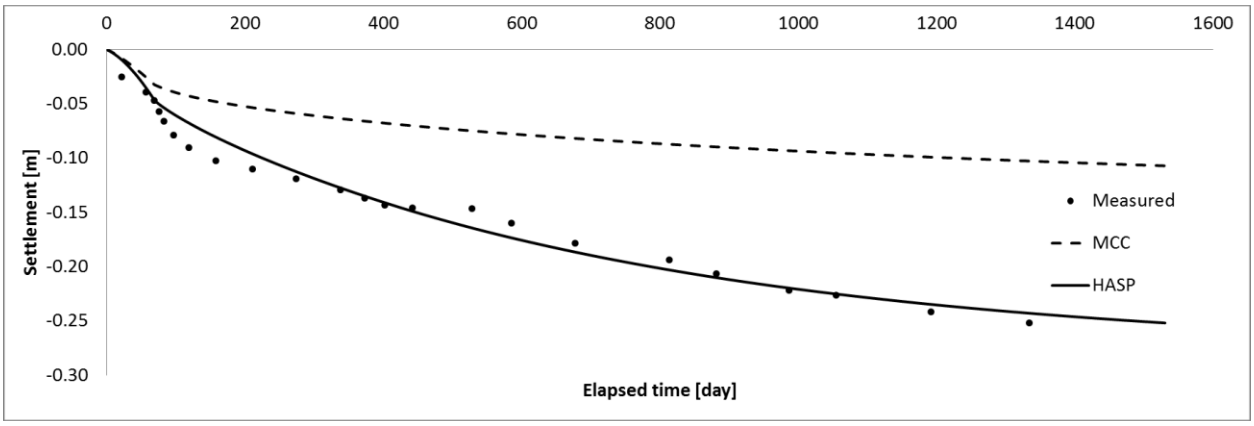

Figure 7. *Comparison of settlement-time curves for Embankment B – Point at the surface of the terrain at the centre of the embankment*

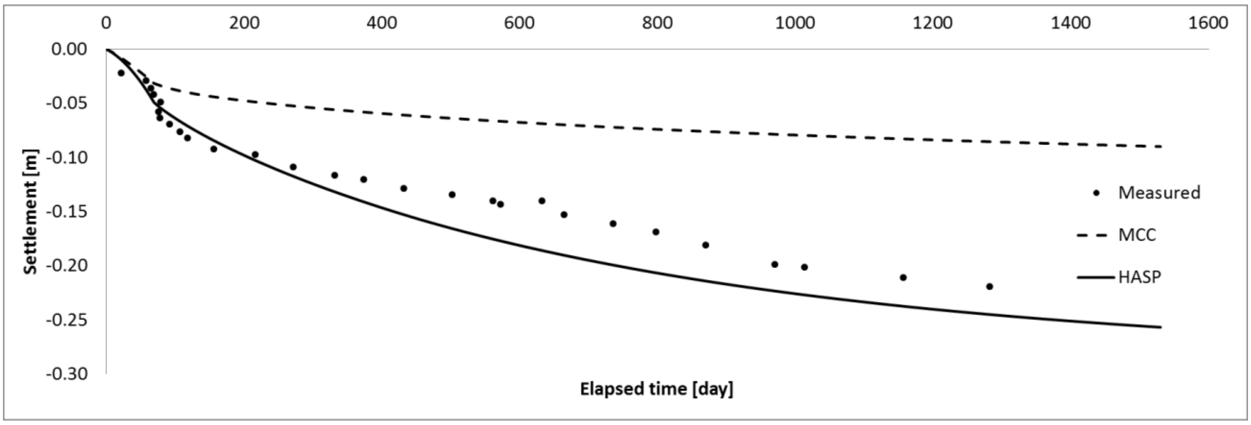

Figure 8. *Comparison of settlement-time curves for Embankment B – Point at the surface of the terrain at point 15 m from the centre of the embankment*

The prediction of the HASP soil model in general gives higher values of settlements compared to the MCC soil model. Those predictions are also more consistent with the field data. That is anticipated, as HASP soil model predicts elastoplastic behaviour from the very beginning of the deformation process, while MCC soil model predicts only elastic deformation as long as the clay is overconsolidated. If the size of the bounding surface is much larger than the size of the yield surface, that is, if OCR ratio has higher values, the differences in predicted settlement between two soil models is more pronounced.

4. CONCLUSIONS

This paper examines settlement of two road embankments constructed on soft overconsolidated soils. The analysis is based on documented case histories and field measurements, and is performed within a finite element framework. Particular attention was given to the influence of the constitutive model on the predicted response. The numerical results show that settlement prediction is strongly affected by the constitutive model adopted for the soft overconsolidated clay layers. This highlights a broader issue in numerical geotechnics: improved predictive capability usually comes with increased model complexity and a larger number of input parameters. In practice, however, the use of highly advanced models is often limited by the amount and quality of available laboratory data. From that standpoint, the HASP model is of particular interest because it improves the description of overconsolidated clay behaviour while retaining a relatively simple parameter set. Material parameters can be obtained from standard laboratory testing, which makes the model suitable for routine geotechnical analyses rather than only for research applications.

Acknowledgement

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