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Numerical modelling of Haarajoki test embankment on soft clays with and without PVDs

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Abstract. This paper investigates the time dependent behaviour of Haarajoki test embankment on soft structured clay deposit. Half of the embankment is constructed on an area improved with prefabricated vertical drains, while the other half is constructed on the natural deposit without any ground improvement. To analyse the PVD-improved subsoil, axisymmetric vertical drains were converted into equivalent plane strain conditions using three different approaches. The construction and consolidation of the embankment are analysed with the finite element method using a recently developed anisotropic model for time-dependent behaviour of soft clays. The constitutive model, namely ACM-S accounts for combined effects of plastic anisotropy, interparticle bonding and degradation of bonds and creep. For comparison, the problem is also analysed with isotropic Soft Soil Creep and Modified Cam Clay models. The results of the numerical analyses are compared with the field measurements. The results show that neglecting effects of anisotropy, destructuration and creep may lead to inaccurate predictions of soft clay response. Additionally, the numerical results show that the matching methods accurately predict the consolidation behaviour of the embankment on PVD improved soft clays and provide a useful tool for engineering practice.

Keywords: embankment; soft clay; creep; anisotropy; destructuration; prefabricated vertical drains

1. Introduction

Design and construction of embankments on soft clay deposits is a challenge for civil engineering. The main problems are the high compressibility and low permeability of the underlying deposit together with very low undrained shear strength. In order to improve the stiffness and strength of the soft clays, prefabricated vertical drains (PVD) are commonly installed into soft clays. It can be expected that the majority of the settlements will occur after construction as the excess pore pressures dissipate.

The stress-strain behaviour of soft sensitive clays is highly nonlinear and very complex. Fundamental features of these soils are high compressibility, time-dependent behaviour, fabric anisotropy (particle arrangement) and inter-particle bonding. Mitchell and Soga (2005) reported that the term creep is used to express time-dependent strains of soils under constant load. The development of time-dependent strains proceeds at a rate controlled by the viscouslike resistance

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of soil structure. Natural soft clays exhibit a significant degree of anisotropy developed during their deposition, sedimentation, consolidation history and any subsequent straining (Leroueil *et al.* 1985). Anisotropy influences the stress-strain behaviour of clays regarding of both elastic and plastic strains (Wheeler *et al.* 2003). Several researchers (Burland 1990, Leroueil and Vaughan 1990, Kavvadas and Amorosi 2000) stated that most natural clays exhibit some inter-particle bonding which effects mechanical behaviour of the clays. The term "*destructuration*" is used to define progressively destroyed inter-particle bonding during plastic straining. These features are important in geotechnical problems wherein long term behaviour is of interest such as embankment. To be able to perform more realistic simulations of time-dependent behaviour of an embankment over soft sensitive clays, it is necessary to use a constitutive model that accounts for combined effects of creep, anisotropy, bonding and destructuration. In recent years, there have been considerable developments in constitutive modelling of soft clays. A number of visco-plastic constitutive models incorporating effects of these features have been published in the literature (Wheeler *et al.* 2003, Koskinen *et al.* 2002, Leoni *et al.* 2008, Yin *et al.* 2010, Grimstad and Degago 2010, Yin *et al.* 2011). These models have been used successfully on the modelling long term behaviour of embankments on soft soils (e.g., Karstunen *et al.* 2005, Yildiz *et al.* 2009, Karstunen and Yin 2010). However, the previous studies have been generally focused on the long term behaviour of embankments constructed on soft clays without any ground improvement. Prefabricated vertical drains are often used to accelerate the consolidation of soft subsoil and the effect of creep might become significant in the improved area. Karstunen and Yin (2010) reported that field data shows clear evidence of the effect of viscosity, which influences the long-term behaviour of the embankment. Yin and Zhu (1999) stated that viscous behaviour, which is often ignored in routine analysis, should be considered to investigate long term behaviour of the embankments on soft clays. Until now, however, no studies have been made on the behaviour of embankments on PVD improved area with consideration of the combined effects of creep, anisotropy and destructuration.

Haarajoki test embankment, constructed on soft structured clay deposit with and without vertical drains, has been successfully employed to validate different constitutive models due to its well documented site investigation and field monitoring measurements (e.g., FinnRa 1997, Näätänen *et al.* 1998, Neher *et al.* 2001, Zhou and Yin 2004, Yildiz *et al.* 2009). Yildiz *et al.* (2009) simulated Haarajoki test embankment using the time-independent S-CLAY1 models which account for anisotropy and inter-particle bonding. It was found that ignoring these features lead to notable underprediction of long term settlements under the embankment. However, Yildiz *et al.* (2009) emphasized that the effect of creep might be important in the vertically drained area since PVDs speed up significantly the rate of consolidation and further investigations should consider the creep effect, using the time dependent extension of the S-CLAY1 models. So, this paper aims to simulate the time-dependent behaviour of Haarajoki test embankment with and without PVD. In the numerical analysis, a recently developed elasto-viscoplastic model, namely anisotropic creep model with destructuration (ACM-S), is used to represent soft clay behaviour. The model accounts for combined effects of creep, anisotropy and destructuration. For comparison, the problem is also analysed with two isotropic Modified Cam Clay (MCC) and Soft Soil Creep (SSC) models. The results of numerical studies are compared with field observations.

In this paper, firstly the constitutive model is briefly described and then some short information about the Haarajoki test embankment is given. Following that, large strain finite-element analyses are carried out for both unimproved and the PVD improved cases. Finally, the results of model predicitons are compared with the observed embankment behaviour.

2. Constitutive model ACM-S

Leoni *et al*. (2008) developed a new anisotropic creep model, namely ACM, to incorporate the anisotropy into the time-dependent behaviour of soft clays. Later, its extension called ACM-S which accounts, additionally, for the effect of bonding and degradation of bonds with plastic straining was proposed by Kamrat-Pietraszewska (2011). The model based on SSC (Vermeer *et al*. 2000) and S-CLAY1 (Wheeler *et al*. 2003, Karstunen *et al*. 2005) models. The isotropic SSC model is a visco-plastic version of the well-known Modified Cam Clay model (Roscoe and Burland 1968). From the Leoni *et al*. (2008) viewpoint, the assumption of an isotropic yield surface which adopted in SSC model does not represent the observed behaviour of natural soft clays and a more realistic model must account for the initial anisotropy and changes in the fabric anisotropy due to plastic straining. The ACM and its extension adopt a rotated yield surface for current stress surface (CSS) purposed for S-CLAY1 model (Wheeler *et al*. 2003), see Fig. 1. The other yield surface is normal consolidation surface (NCS) which assumed to have same inclination with CSS (Fig. 1). In triaxial stress space the size of the CSS is defined as:

There is an equivalent mean stress, p'_{eq} which determines the size of yield surface for CSS and it is defined as

$$p'_{eq} = p' + \frac{(q - \alpha \cdot p')^2}{(M^2 - \alpha^2) \cdot p'} \tag{1}$$

where p'_{eq} is an equivalent mean stress, M is the stress ratio at critical state and α describes the orientation of yield surface. By setting α to zero in Eq. (1) the model would reduce to isotropic SSC model.

The NCS is defined by $p'_{eq} = p'_q$, where p'_q is the pre-consolidation pressure. The pre-consolidation pressure p'_q evolves the volumetric creep strain ε_v^c according to the hardening law and it is defined as

$$p'_p = p'_{p0} \cdot \exp\left(-\frac{\varepsilon_{vol}^c}{\lambda_i^* - \kappa^*} \right) \tag{2}$$

where λ_i and κ^* are the intrinsic modified compression index and modified swelling index,

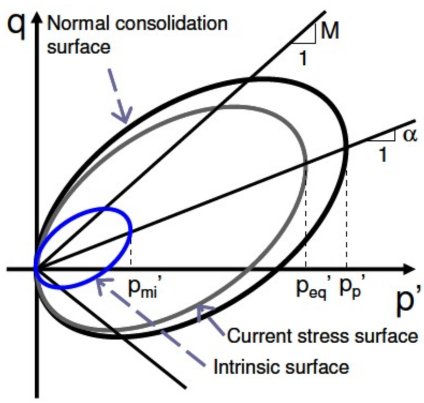


Fig. 1 Yield Surface for ACM-S (Kamrat-Pietraszewska 2011)

respectively defined as $\lambda_i^* = \lambda_i / (1 + e_o)$ and $\kappa^* = \kappa / (1 + e_o)$, where λ_i and κ are the slopes of the intrinsic compression and swelling line in the $e_0 -$ ln p' plane and e_0 is the initial void ratio. The distance between the CSS and NCS is treated as a generalised overconsolidation ratio $OCR^* = p'_p / p'_{eq}$. Creep volumetric strains are defined as

$$\dot{\varepsilon}^c_{col} = \frac{\mu_i^*}{\tau}\left(\frac{1}{OCR^*}\right)^{\beta} \tag{3}$$

where μ_i^* is the intrinsic modified creep index, β is the creep exponent and τ is the reference time. The creep exponent is defined as

$$\beta = \frac{\lambda_i^* - \kappa}{\mu_i^*} \tag{4}$$

The evolution of α is governed by creep strains according to the rotational hardening law and is given by

$$\dot{\alpha} = \omega\left[\left(\frac{3q}{4p'} - \alpha\right)\dot{\varepsilon}^c_{col} + \omega_d\left(\frac{q}{4p'} - \alpha\right)\dot{\gamma}^c\right] \tag{5}$$

where $\dot{\gamma}^c$ is the deviatoric creep strain rate. The soil constants ω and ω_d that control the rate of rotation are related to basic soil parameters. For further details, the interested reader is referred to Leoni *et al*. (2008) and Kamrat-Pietraszewska (2011).

In the model, a notional intrinsic yield surface is used to describe bonding effects between clay particles and degradation of the bonds (Fig. 1). The shape and orientation of the intrinsic yield surface are same as for the yield surface of the natural soil but smaller in size (Fig. 1). The size of intrinsic yield surface is specified by a parameter p'_{mi} and it is related to the size of yield surface (p'_{eq}) for the natural soil defining the current degree of bonding

$$p'_{eq} = (1 + \chi)p'_{mi} \tag{6}$$

where χ is the amount of bonding. The initial value of χ can be estimated based on sensitivity ($\chi \approx S_t - 1$). The degradation of bonding is a function of volumetric and deviatoric creep strains (destructurational law) and it is defined as

$$d\chi_0 = -\xi \cdot \chi_0 \cdot \left(| d\varepsilon_v^c | + \xi_d \cdot d\varepsilon_d^c\right) \tag{7}$$

where χ_0 is the initial amount of bonding and the parameters ξ and ξ_d control the absolute and relative rate of destructuration of bonds. A detailed description of the model can be found in Kamrat-Pietraszewska (2011).

3. Determination of model parameters

The ACM-S involves a number of soil constants and state variables which can be classified in four groups:

- *Standard soil constants and state parameters:* These parameters are soils constants ν' (Poisson's ratio), ϕ'_{cv} (friction angle at critical state), λ^* (modified compression index) and κ^* (modified swelling/recompression index). Furthermore, the initial values for two state variables, namely e_0 (initial void ratio) and p_{m0} (initial size of the yield surface) are required. The initial value of p_{m0} is calculated based on the *OCR* (vertical overconsolidation ratio), normally consolidated K_0^{NC} value (lateral earth pressure at rest, estimated by Jaky's formula) and the initial vertical effective stress.
- *Anisotropy parameters* (α_0, ω *and* ω_d)*:* Wheeler *et al*. (2003) reported that it is observed that there is a close correlation between the initial inclination α_0 and K_0^{NC} based on a large amount of one-dimensional consolidation data. According to Jaky's formula, K_0^{NC} is defined as

$$K_0^{NC} \approx 1 - \sin \phi'_{cv} \tag{8}$$

The M values are calculated for all soil layers as below

$$M = \frac{6 \sin \phi'_{cv}}{\left(3 - \sin \phi'_{cv}\right)} \tag{9}$$

Assuming an associated flow rule and one-dimensional loading conditions, the ratio between deviatoric and volumetric plastic strain rates is approximately 2/3 (Wheeler *et al*. 2003) and it follows that

$$\alpha_0 = \frac{\eta_0^2 + 3\eta_0 - M^2}{3} \tag{10}$$

where

$$\eta_0 = \frac{3(1 - K_0^{NC})}{1 + 2K_0^{NC}} \tag{11}$$

The shear rotation parameter ω_d is also found to correlate with the K_0^{NC}-value (Wheeler *et al*. 2003) according to

$$\omega_d = \frac{3}{8}\left[\frac{4M^2 - 4\eta_0^2 - 3\eta_0}{\eta_0^2 - M^2 + 2\eta_0}\right] \tag{12}$$

Another new material parameter, ω is simply defined as a function of the compression index, λ^*. Leoni *et al*. (2008) assumed that anisotropy is practically erased when α decreases to 1/10th of its initial value, which means $\alpha_0 / \alpha = 10$, ω can be defined as below

$$\omega = \frac{1}{\lambda^*} \ln \frac{10M^2 - 2\alpha_0\omega_0}{M^2 + 2\alpha_0\omega_0} \tag{13}$$

- *Bonding and destructuration parameters* (χ_0, ξ *and* ξ_d)*:* A conservative method based on measured sensitivity is used to determine the value of initial amount of bonding χ_0 for a natural soil. Determination of the destructuration parameters ξ and ξ_d requires an optimization procedure by comparing the experimental results with model simulations, as explained by Koskinen *et al*. (2002). Noting that any data on isotropic consolidation test on Haarajoki clay is not available. Hence, in the numerical analyses, typical values of ξ and ξ_d for Finnish clays purposed by Yildiz *et al*. (2009) are used.

- *Viscosity parameter* (μ_i^*): μ_i^* is the intirinsic modified creep index and calculated by using $C_{\alpha e}$ (creep index) since the viscosity parameters are independent of destructuration process (Yin *et al*. 2011).

4. Haarajoki test embankment

In 1997, Finnish National Road Administration has constructed a test embankment in Haarajoki, about 40 km northeast of Helsinki in Finland (FinnRa 1997). The embankment was constructed as part of a noise barrier. The longitudinal cross section of the embankment is shown schematically in Fig. 2. The embankment is 2.9 m high and 100 m long. The width at the top is 8 m and the gradient of the slopes is 1/2. A half of the embankment was constructed on natural deposits without any additional ground improvement (cross section 35840) and the other half was constructed on area improved with prefabricated vertical drains (cross section 35880). 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 embankment is founded on a 2 m thick dry crust layer underlain by a 20 m thick layer of soft clays/clayey silts. The layers underneath the soft soil layer consist of silt and till which can be considered as permeable soils. The groundwater table is at the ground surface. The subsoil is divided into seven sublayers with different compressibility parameters and overconsolidation ratios. The water content of the soft clay layer varies between 67 and 112% depending on the depth. The plasticity index values change between 45 and 75% and the undrained shear strength lies between 14 and 30 kPa (FinnRA 1997). The bulk density varies from 14 to 17 kN/m^3 and specific gravity varies from 2.73 to 2.79. The Haarajoki deposits can be characterized as a sensitive anisotropic soft soil with sensitivity values (determined with fall cones

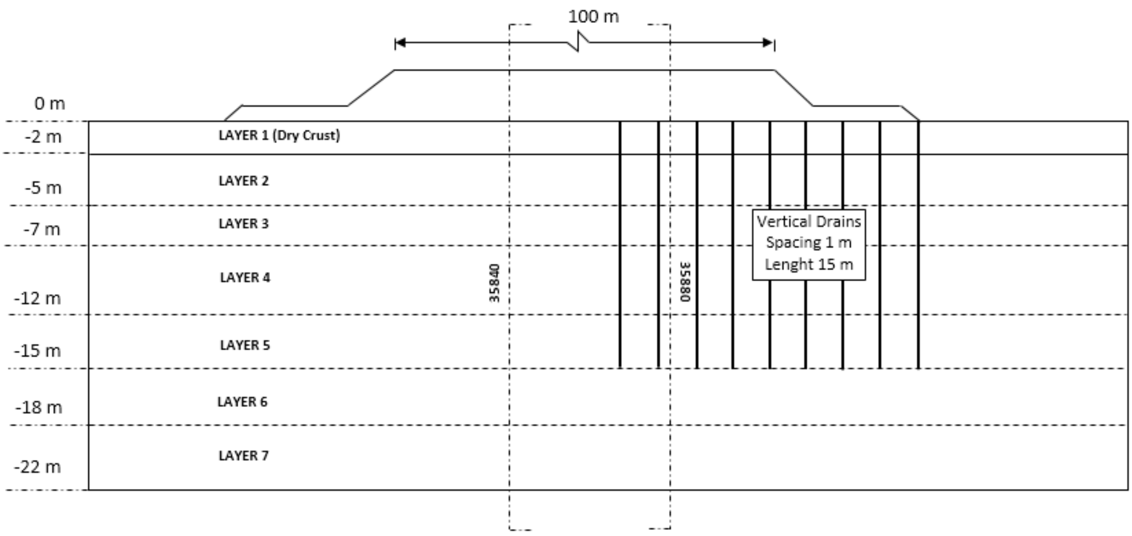


Fig. 2 Haarajoki test embankment (longitudinal section)

tests) between 20 and 55. The organic content is between 1.4 and 2.2% at a depth of 3-13 m. Further details on the site and typical characteristics of the deposits can be found in Yildiz *et al.* (2009). The soft clay deposit was modeled as a lightly overconsolidated soft clay with vertical preconsolidation pressures varying with the depth.

The values of soil parameters were estimated for each layer based on laboratory results provided by FinnRA (1997). The values of permeability used for calculations were reported by Näätänen *et al.* (1998) based on vertical and horizontal constant rate of strain (CRS) oedometer tests. As there is natural variation between clay samples further magnified by potential sample disturbance, for each clay layer a number of tests of the same type were used to estimate the average values of parameters. A number of publications are available which report the required

Table 1 MCC soil parameters

Depth (m)	γ (kN/m³)	v'	M	e_0	κ	λ	k_v (m/day)	k_h (m/day)
0-2	17.5	0.2	1.50	1.40	0.009	0.18	1.73×10^{-4}	3.46×10^{-4}
2-5	14.3	0.2	1.15	2.90	0.033	1.33	5.18×10^{-5}	1.04×10^{-4}
5-7	14.3	0.2	1.10	2.60	0.037	0.96	4.32×10^{-5}	8.64×10^{-5}
7-12	15.1	0.2	1.07	2.35	0.028	0.90	4.32×10^{-5}	8.64×10^{-5}
12-15	15.1	0.2	1.15	2.20	0.033	1.06	4.32×10^{-5}	8.64×10^{-5}
15-18	15.7	0.2	1.50	2.00	0.026	0.45	4.32×10^{-5}	8.64×10^{-5}
18-22	17.5	0.2	1.50	1.40	0.009	0.10	1.73×10^{-4}	3.46×10^{-4}

Table 2 SSC soil parameters

Depth (m)	ϕ_{cv}'	κ^*	λ^*	μ^*
0-2	36.9	0.004	0.080	9.65×10^{-4}
2-5	28.8	0.008	0.340	1.34×10^{-3}
5-7	27.7	0.010	0.267	1.21×10^{-3}
7-12	27.0	0.009	0.270	8.30×10^{-4}
12-15	28.8	0.010	0.331	6.79×10^{-4}
15-18	36.9	0.009	0.150	4.63×10^{-4}
18-22	36.9	0.004	0.044	6.18×10^{-4}

Table 3 ACM-S soil parameters

Depth (m)	α_0	λ_i^*	μ_i^*	ω	ω_d	χ_0	ξ	ξ_d
0-2	0.63	0.080	$9.65\text{x}10^{-4}$	37	1.02	4	0	0
2-5	0.44	0.087	$1.34\text{x}10^{-3}$	33	0.70	19	8	0.2
5-7	0.55	0.061	$1.21\text{x}10^{-3}$	49	0.97	19	8	0.2
7-12	0.44	0.066	$8.30\text{x}10^{-4}$	44	0.70	25	8	0.2
12-15	0.46	0.084	$6.79\text{x}10^{-4}$	35	0.76	25	8	0.2
15-18	0.61	0.083	$4.63\text{x}10^{-4}$	36	1.01	8	8	0.2
18-22	0.61	0.080	$6.18\text{x}10^{-4}$	37	1.01	45	0	0

parameter values (e.g., Näätänen *et al*. 1998, Yildiz *et al*. 2009). The values of the input parameters and state variables for the models are listed in Tables 1-3. In the finite-element analyses, the decrease in the permeability as the void ratio decreases was taken into account according to Taylor (1948). Numerous measuring devices (settlement plates, piezometers, inclinometers, pressure cells) were installed under the test embankment for monitoring the vertical and lateral displacements and the pore pressures. The depths and locations of the instruments under Haarajoki test embankment were given in Yildiz *et al*. (2009).

5. Finite element analysis

The construction and the subsequent consolidation of Haarajoki embankment on structured clays with and without PVDs were simulated with three constitutive models (MCC, SSC and ACM-S) using PLAXIS 2D Version 8.6 (Brinkgreve 2002). The ACM-S has been implemented in the finite-element program as user-defined models (Kamrat-Pietraszewska 2011). The results of the numerical analyses were compared with the field measurements. Cross Section 35840 built on natural clay without PVDs was firstly simulated and then Cross Section 35880 on the vertically drained area was simulated. In this paper, the main focus is on long term settlements for both the unimproved and the PVD improved area.

5.1 Unimproved case (*Cross Section 35840*)

The Cross Section 35840 is built on the natural deposits without any ground improvement. The embankment was assumed symmetric and only half of the embankment is considered in the finite element analyses. The plane strain condition and 6-noded triangular elements were used and updated mesh analysis taking into account the effects of large deformations was used in the numerical analyses. The boundaries of the geometry used in the finite element analyses have an extent of 60 m in the horizontal direction from the embankment centerline and 22 m in the vertical direction. A finite element mesh with 982 elements is used to model the subsoil and the embankment. Fig. 3 shows the finite element mesh for the embankment on natural soil. The groundwater table is located at the ground surface. The lateral boundaries are restrained horizontally and the bottom boundary is restrained in both directions. Drainage boundaries are assumed to be at the ground surface and at the bottom of the mesh, whereas the lateral boundaries are closed. The embankment construction consists of two phases: first, the embankment loading is applied under undrained conditions, assuming the embankment to be drained material and next, a consolidation phase is simulated via fully coupled consolidation analysis. The real construction schedule has been simulated in the calculation. The construction of embankment was completed in 35 days. Mesh sensitivity studies were done to confirm that the mesh was dense enough to give accurate results for all of the constitutive models concerned.

The embankment which is assumed to be made of granular fill was modelled with Mohr Coulomb model assuming the following material parameters: $E' = 40{,}000$ kN/m^2, $\nu' = 0.3$, $\phi' = 38°$, $\psi' = 0°$, $c' = 1$ kN/m^2 and $\gamma = 21$ kN/m^3, where E' is the Young's modulus, ν' is the Poisson's ratio, ψ' is the dilatancy angle and γ is the unit weight of the embankment material.

The observed and predicted vertical settlements versus time at the centre, 4 m (crest) and 9 m (toe) away from the centreline at station 35840 are presented in Fig. 4. The observed and predicted surface settlements under the embankment are also shown in Fig. 5. The results show that all

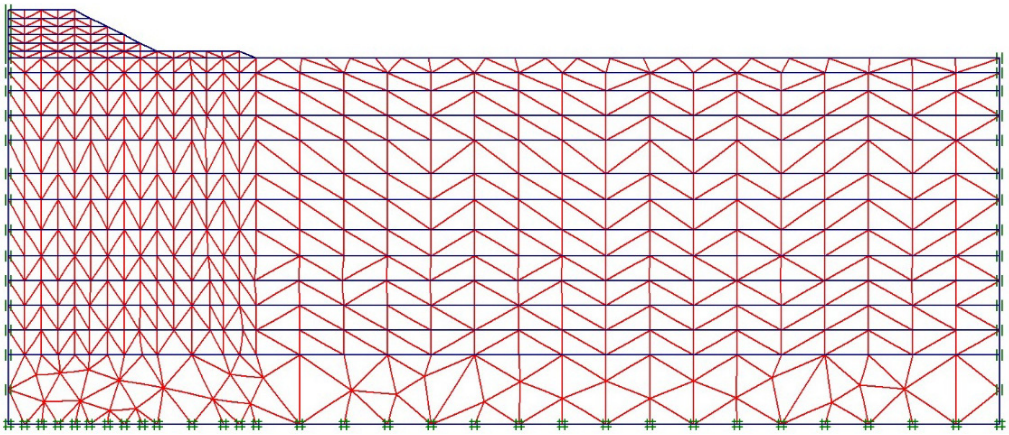
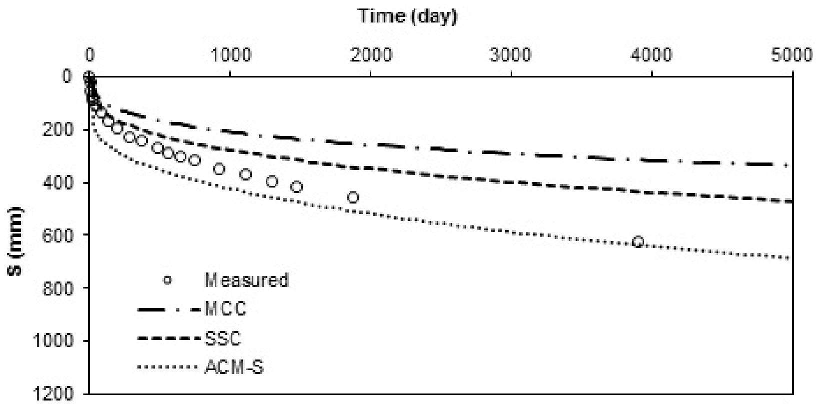
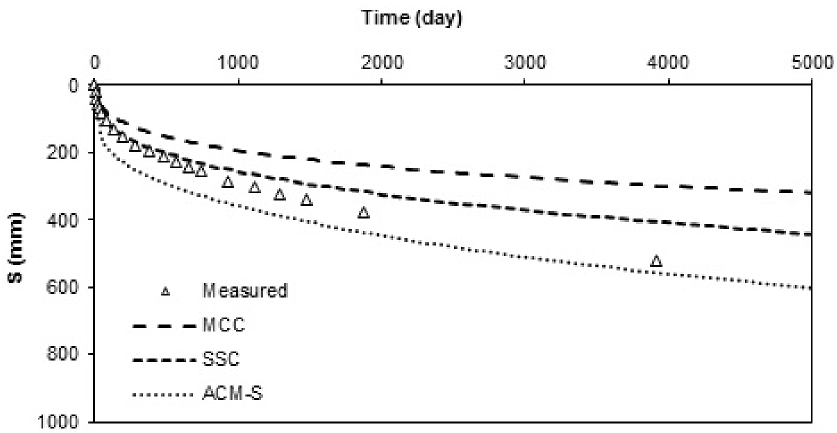


Fig. 3 Finite element mesh for unimproved section

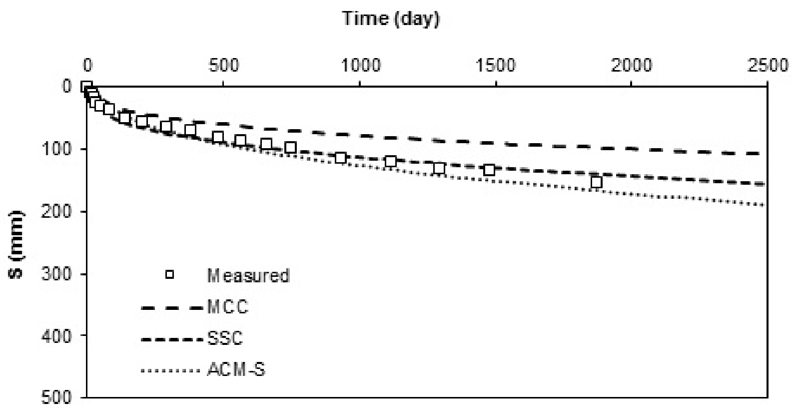


(a) Embankment center



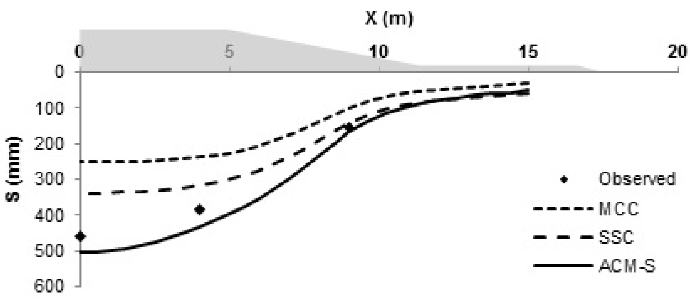
(b) Embankment crest

Fig. 4 Comparison between measurements and predictions for consolidation settlements

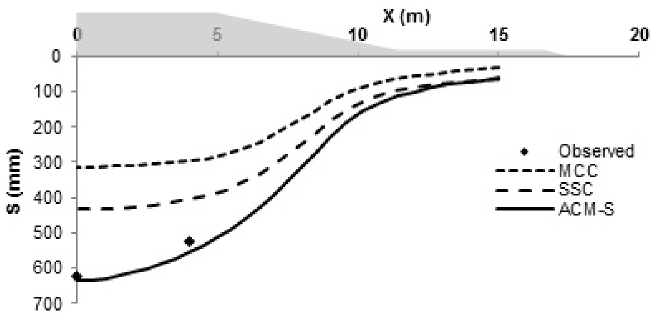


(c) Embankment toe

Fig. 4 Continued



(a) Settlements after 5 years



(b) Settlements after 10.7 years

Fig. 5 Comparison between measurements and predictions for surface settlements

models predict a maximum settlement at the center of the embankment. The maximum settlement measured at the center of the embankment is about 0.46 m after 5 years of consolidation. The MCC and SSC models predict a maximum settlement of about 0.25 m and 0.34 m, respectively, after 5 years of consolidation, while the ACM-S predicts a maximum settlement of about 0.50 m. The observed settlement at the center of the embankment is 0.63 m after about 10.7 years of consolidation. The MCC and SSC models predict a maximum settlement of about 0.33 m and 0.43 m after 10.7 years of consolidation at the center of the embankment, respectively, while the ACM-S predicts a maximum settlement of about 0.64 m. As seen that the ACM-S is in a good agreement with the observed settlements. Both isotropic models (MCC and SSC) predict a lower rate of settlement during the entire simulation and thus the prediction is not as good as the anisotropic creep model (ACM-S). The results show that anisotropy, destructuration and creep play important role on the settlement behaviour of the embankment and neglecting these effects lead to notable underprediction of settlements for this particular boundary value problem.

5.2 Improved case (*Cross Section 35880*)

Cross section 35880 is situated in the middle of the part of Haarajoki test embankment constructed on soft clay improved with PVDs. The length of the vertical drains is 15 m and they were installed in a square grid with 1 m spacing underneath the embankment. The prefabricated drains have a typical rectangular cross section and were converted to be equivalent to a circular drain having a diameter d_w of 67 mm based on "*perimeter equivalence*" proposed by Hansbo (1979).

$$d_w = \frac{2(w+t)}{\pi} \tag{14}$$

where d_w = equivalent diameter of a drain; and w and t = width and thickness of the drain, respectively. In the field, vertical drains are installed using a steel mandrel which causes significant remoulding of the subsoil, especially in the immediate vicinity of the mandrel. Chu *et al*. (2004) reported that the smear zone around each vertical drain will have reduced lateral permeability, which adversely affects consolidation process. Yildiz *et al*. (2009) stated that Haarajoki deposits can be characterized as a very sensitive anisotropic soft clay and the water content is often higher than the liquid limit. Hence, considerable disturbance is expected in the subsoil during the installation of vertical drains. However, many uncertainty factors are available and there is no available test data for this particular soil in this study. Yildiz *et al*. (2009) and Yildiz (2009a) suggested that a good quality of match can be achieved by assuming the values of $d_s / d_m = 5$ (d_s and d_m are the diameters of smear zone and mandrel, respectively) and $k_h / k_s = 20$ (k_h and k_s are the permeability values of natural and disturbed soils, respectively) when there is no test data for evaluating the smear zone.

Finite element analyses of embankments are commonly carried out based on 2-D plane strain assumption. However the flow around vertical drains is axisymmetric. Therefore multi-drain system must be converted into an equivalent plane strain model to employ a realistic 2-D finite element analysis for vertical drains. There are different approaches for modelling the PVD improved subsoil in the literature (e.g., Zeng and Xie 1989, Hird *et al*. 1992, Kim and Lee 1997, Indraratna and Redana 2000, Chai and Miura 2001). These matching techniques generally base on unit cell concept (Fig. 6) and assume that the consolidation behaviour takes place in an independent single drainage system which has linear compressibility characteristics without any lateral movements. Such restrictive conditions do not represent real soft soil behaviour and actual field conditions. As mentioned before, the stress-strain behaviour of natural soft clays is highly nonlinear and very complex. Fundamental features of soil, such as anisotropy, creep and destructuration are also quite different. In the field, the subsoil usually has many layers incorporating a large number of vertical drains. Therefore, it is necessary to verify these matching methods for full-scale simulation of a real embankment on multi-layer subsoil incorporating a large number of vertical drains (Yildiz 2009b). So, three exiting approaches for modelling the PVD improved subsoil were adopted to analyse the behaviour of PVD-improved soft subsoil under embankment loading in this study. Firstly, these methods are briefly summarized below

Method I

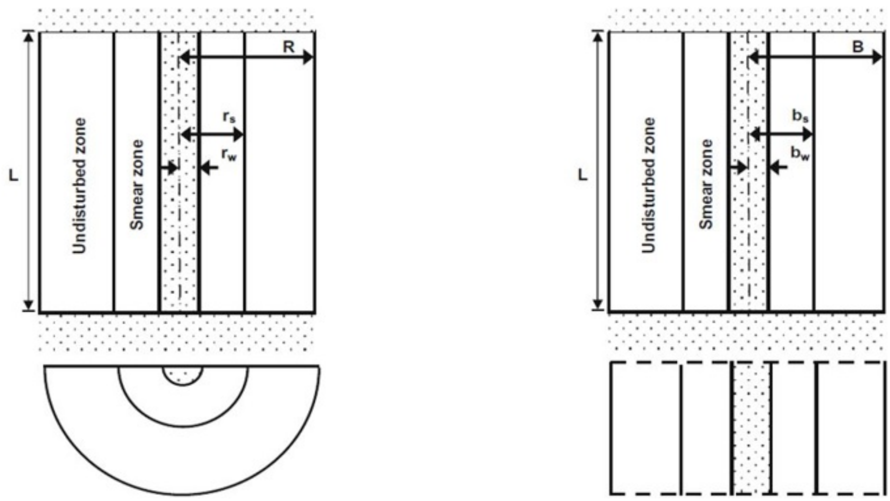
The first method was developed by Hird *et al*. (1992). The matching can be achieved by adjusting drain spacing and/or soil permeability. In this approach, a value of B (half-width of unit cell) is preselected and the equivalent plane strain permeability k_{pl} is calculated by Eq. (15)

$$\frac{k_{pl}}{k_{ax}} = \frac{2B^2}{3R^2\left[\ln\left(\frac{R}{r_s}\right)+\left(\frac{k_{ax}}{k_s}\right)\ln\left(\frac{r_s}{r_w}\right)-\left(\frac{3}{4}\right)\right]} \tag{15}$$

where k_{ax} and k_s are the horizontal permeabilities of the undisturbed and disturbed soil, respectively. In the equivalent plane strain analyses, B = 1 m was preselected and the value of k_{pl} is calculated as $0.052k_{ax}$ by using Eq. (15). The finite element mesh used for the equivalent plane strain analyses is illustrated in Fig. 7(a). A special drain element is used to model a vertical drain in the analyses where excess pore pressures are set to zero. This method has an important advantage that no smear zone needs to be represented in the numerical model.

Method II

The second method was developed by Indraratna and Redana (1997). In this approach, the vertical drain system is converted into an equivalent parallel drain well by adjusting the coefficient of soil permeability. They assumed that the half width of the drain (b_w) and half width of the smear



(a) Axisymmetric unit cell (b) Plane strain unit cell

Fig. 6 Unit cell model

zone (b_s) may be kept the same as their axisymmetric radii r_w and r_s, respectively, which suggests $b_w = r_w$ and $b_s = r_s$ (Fig. 6). The influence of smear effect can be modelled by the ratio of the smear zone permeability to the undisturbed permeability. By assuming the magnitudes of R and B to be the same, Indraratna and Redana (1997) presented a relationship between k_{hp} and k'_{hp}, as follows

$$\frac{k'_{hp}}{k_{hp}} = \frac{\beta}{\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k'_h}\right)\ln(s) - 0.75 - \alpha\right]} \tag{16}$$

where k_h is the horizontal permeability of the undisturbed soil and k'_h is the horizontal permeability of disturbed soil, where the subscript p represents the plane strain condition. The associated geometric parameters α and β are given by

$$\alpha = \frac{2}{3}\frac{(n-s)^3}{(n-1)n^2} \tag{17}$$

$$\beta = \frac{2}{3}\frac{(s-1)}{(n-1)n^2}\left[3n(n-s-1) + (s^2+s+1)\right] \tag{18}$$

where $n = R/r_w$ and $s = r_s/r_w$. A special drain element is used to model each vertical drain in the numerical model. The smear zone of each vertical drain also needs to be discretely represented in the numerical model. The finite element mesh used for the equivalent plane strain analyses is illustrated in Fig. 7(b). In particular, number of elements and input parameters for soil layers become very large because the numerical model incorporates multi-layer subsoil (seven layers) and large number of vertical drains under the embankment. So, the finite element mesh consists of 2400 6-noded triangular elements. Since the half-width (B) of the unit cell is equal to 0.5 m in this study the values of k_{hp} and k'_{hp} are calculated as $0.284k_h$ and $0.012k_h$ in the undisturbed and disturbed zones, respectively.

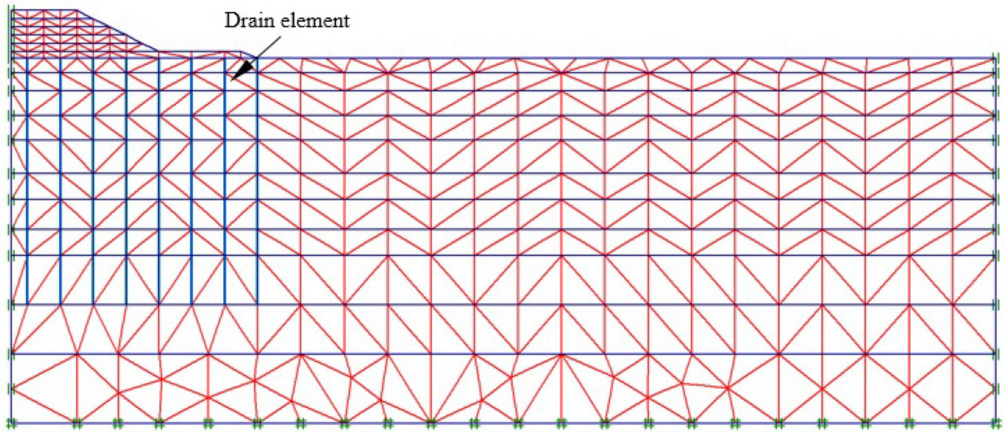
Method III

The third method is a simple approximate approach developed by Chai and Miura (2001). An equivalent value of vertical hydraulic conductivity (k_{ve}) of PVD-improved subsoil is estimated in this method. The value of k_{ve} approximately represents both the effect of vertical drainage of natural subsoil and the effect of radial drainage due to existence of PVD. Using this approach, the behaviour of PVD-improved subsoil can be analysed in a similar manner to that of unimproved natural subsoil. The equivalent vertical hydraulic conductivity k_{ve} can be expressed as

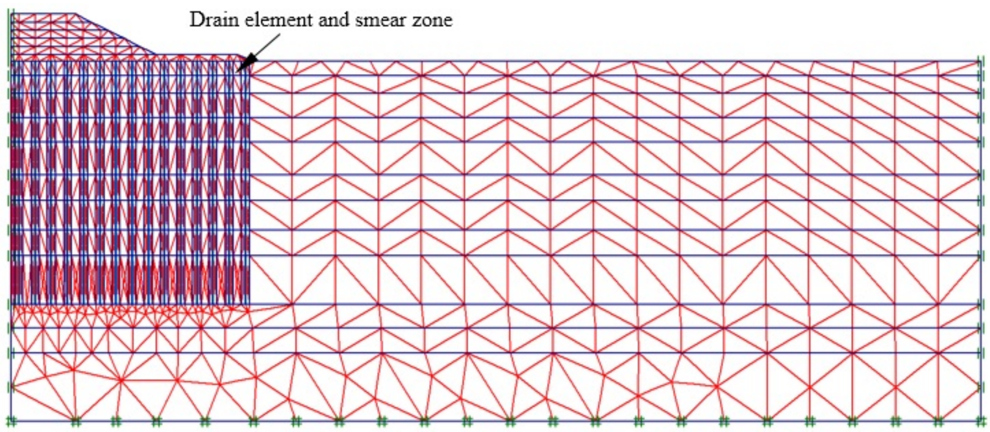
$$k_{ve} = \left(1 + \frac{2.5l^2}{\mu D_e^2}\frac{k_h}{k_v}\right)k_v \tag{19}$$

where k_v = hydraulic conductivity in the vertical direction, l = drainage length, D_e = diameter of unit cell. The value of μ can be expressed as

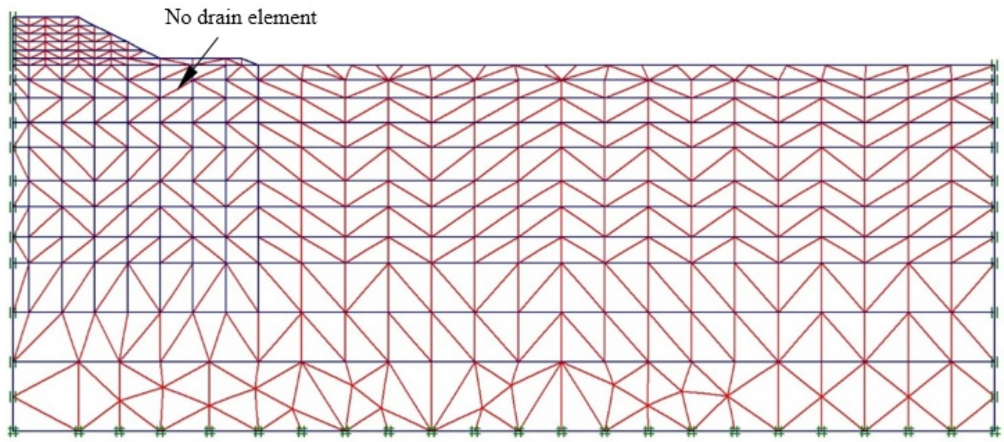
$$\mu = \ln\frac{n}{s} + \frac{k_h}{k_s}\ln(s) - \frac{3}{4} + \pi\frac{2l^2k_h}{3q_w} \tag{20}$$



(a) Method I



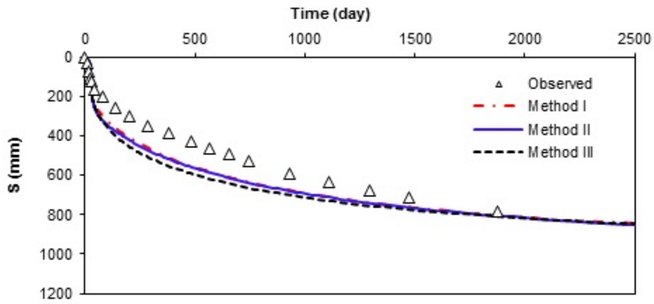
(b) Method II



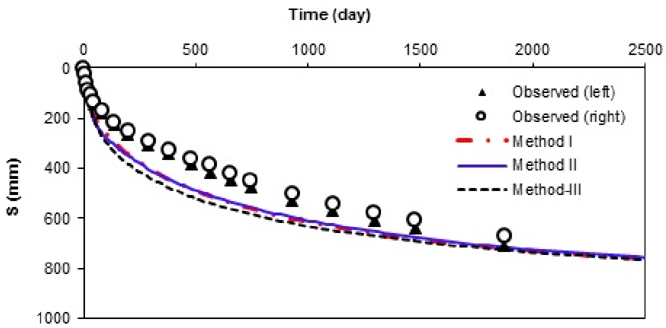
(c) Method III

Fig. 7 Finite element mesh for three matching methods where $n = D_e / d_w$ (d_w = diameter of smear zone); $s = d_s / d_w$ (d_s = diameter of smear zone), k_h and k_s = horizontal permeabilities of the natural soil and smear zone, respectively. This method is the simplest one among the three matching methods since vertical drains and their smear zones don't need to be represented in the equivalent plane strain model. The finite element mesh used for the equivalent plane strain analyses is illustrated in Fig. 7(c). The equivalent vertical hydraulic conductivity k_{ve} is calculated as $22.97k_v$ by using Eq. (19).

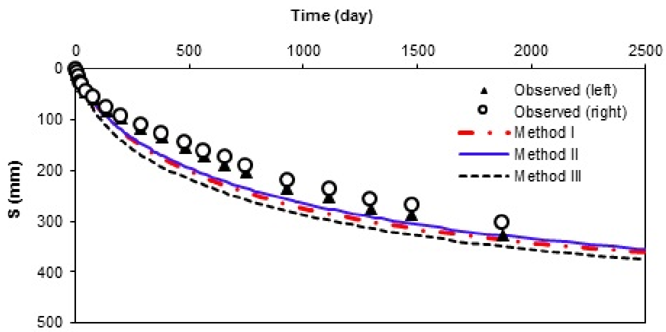
The first set of finite element analyses is performed to see efficiency of three different methods in the numerical analysis of Haarajoki embankment. The results have been plotted only for the ACM-S. The predicted vertical settlements at the centre, 4 m (crest) and 9 m (toe) away from the



(a) Embankment center



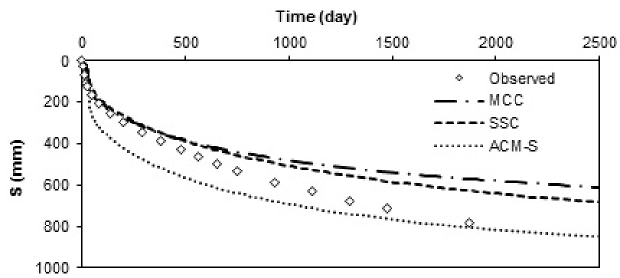
(b) Embankment crest



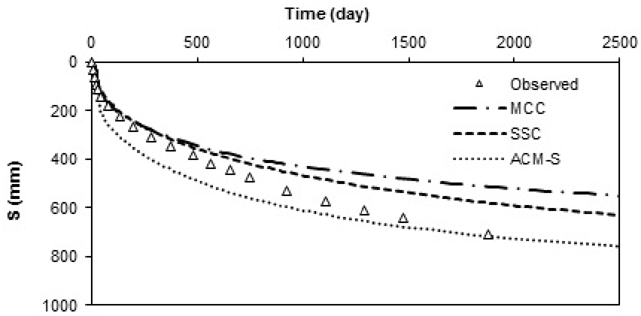
(c) Embankment toe

Fig. 8 The predicted vertical settlements and field measurements centreline at station 35880 are compared with the field measurements in Fig. 8. As seen that these matching methods give acceptable results when real boundary conditions and complex visco-plastic model are used in the finite element analyses. The rate of consolidation in the predictions is slightly faster than that in measured values. This may be caused from certain simplifying assumptions made in the matching methods as explained before. Such restrictive conditions are not likely to be achieved in normally or lightly overconsolidated soils in the field conditions. However, the match is found to be satisfactory for three methods. Method II produced the best agreement with the observed results. These techniques provide useful tools for engineering practice and an easy way to analyse the behaviour of PVD-improved subsoil. Otherwise, we have to carry out a 3D full-scale simulation to analyse the behaviour of PVD-improved subsoil. However, 3D analyses are very sophisticated and require large computational effort when applied to a real embankment project with a large number of vertical drains.

Based on Method II, the settlements predicted by three constitutive models (MCC, SSC and ACM-S) for 5 years of consolidation time are compared to the field measurements in Fig. 9. The predicted settlements by three models versus time at the ground surface under the embankment are compared to the field measurements in Fig. 10. The maximum settlement measured at the center of the embankment is about 0.79 m after 5 years of consolidation. The MCC and SSC models predict a maximum settlement of about 0.57 m and 0.63 m, respectively, while the ACM-S predicts a maximum settlement of about 0.79 m after 5 years of consolidation. As seen that the ACM-S well predicted the surface settlement after 5 years of consolidation time whereas both isotropic models generally underestimated the surface settlements. Comparing the settlements predicted by

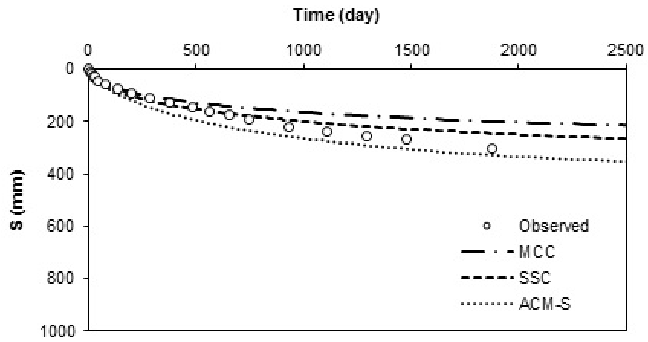


(a) Embankment center



(b) Embankment crest

Fig. 9 The predicted vertical settlements and field measurements



(c) Embankment toe

Fig. 9 Continued

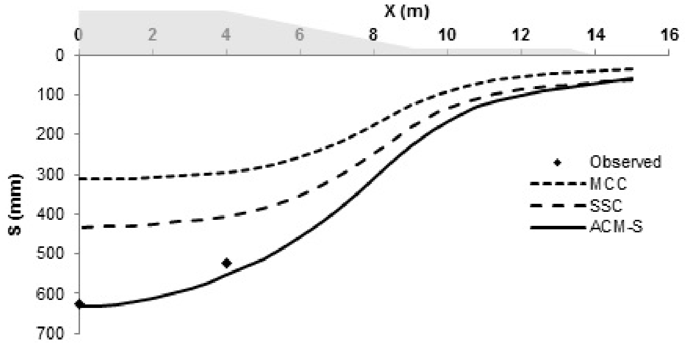


Fig. 10 Comparison between measurements and predictions for surface settlements

SSC and MCC, it is clear that the consideration of creep effect in SSC model increases settlements. However accounting, additionally, for anisotropy, initial bonding and destructuration (ACM-S) leads to predict much larger settlements in the analyses. Therefore, both isotropic models predicted a lower rate of settlement during the entire simulation and the predictions are not as good as the ACM-S predictions. However, the rate of consolidation in the calculations by ACM-S developed slightly faster than the rate of consolidation in measured values. This is due to the matching effect which is not perfect. Similar results were also repoted by Yildiz (2009a and b).

6. Conclusions

This paper investigates long term behaviour of Haarajoki test embankment on with and without PVD improved soft clay. A recently developed visco-plastic ACM-S is used to represent the soft soil. The model accounts for creep effect, initial and plastic strain induced anisotropy and inter-particle bonding and degradation of bonds. For comparison, the embankment is also analysed with isotropic MCC and SSC models. Both isotropic models generally underestimate time dependent settlements underneath the embankment. The comparisons between calculations and measurements demonstrate that the settlements predicted by ACM-S generally agrees with the field measurements. The model can describe time dependent behaviour of the embankment on structured clay reasonably well. However, ACM-S significantly overpredicts the lateral deformations underneath the crest of the embankment. The numerical simulations demonstrate that the creep, anisotropy and micro-structure have a significiant influence on the time-dependent behavior of subsoil with and without PVD under embankment loading. Neglecting these effects leads to notable underprediction of settlements for this particular boundary value problem.

The actual 3D behaviour of vertical drains can be converted into equivalent plane strain conditions with three different matching approaches. The numerical simulations demonstrate that the settlements predicted by three different approaches are found to be satisfactory when real boundary conditions and complex soil models are used in the finite element analyses. These techniques are useful tools for engineering practice and provide an easy way to analyse the behaviour of PVD-improved subsoil. Method II produces the best agreement with the field measurements. However, this approach requires largest computational effort since each drain and its smear zone need to be discretely represented in the simulations. Method III is the simplest approach and can be directly applied in any situation since the behaviour of PVD-improved subsoil can be analysed in a similar manner to that of unimproved natural subsoil.

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