Numerical analysis of an embankment on soft clay deposit with and without PVD-Improvement

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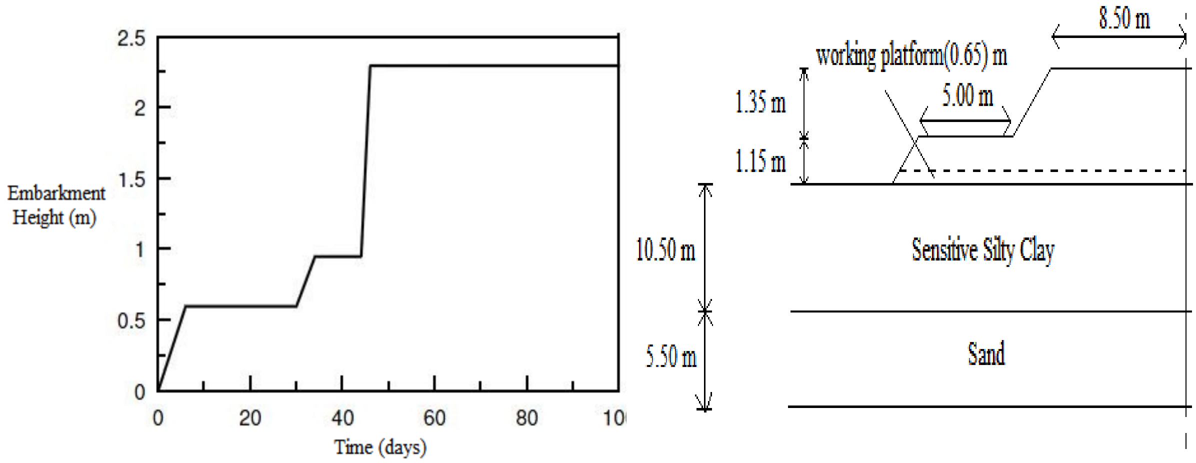
ABSTRACT**:** The case study analysed within this paper deals with a fully instrumented trial embankment constructed on soft clay foundations in Sunshine Motorway in Queensland, Australia. Trial embankment was constructed with three different ground improvement schemes (Section A: PVDs with 1m spacing, Section B: No PVDs and Section C: PVDs with 2m spacing). Subsoil layer at this site is composed of very soft, highly compressible, saturated organic marine clays of high sensitivity. The construction and consolidation of an embankment with and without prefabricated vertical drains is analysed with the finite element method using recently proposed constitutive model is namely S-CLAY1. The model accounts for initial and plastic strains induced anisotropy. The results of the numerical analyses are compared with the field measurements. The good performance of the finite element model in predicting time dependent behaviour of Sunshine Motorway embankment is presented.

1 INTRODUCTION

Prefabricated vertical drains (PVDs) method is still one of the classical and popular methods in practice. The application of preloading with prefabricated vertical drains has been used to accelerate the consolidation and to reduce future settlements by shortening the drainage path. Design of an embankment involving a large number of discrete vertical drains and their own independent influence zone should be conducted with a fully three dimensional analysis. 3D finite element modelling of vertical drain system is very sophisticated and requires large computational effort when applied to a real embankment project with a large number of PVDs. 2D finite element analyses (FEA) of embankments have commonly been conducted under plane strain conditions. However, the actual field conditions around vertical drains are truly 3D and therefore, it is necessary to convert the vertical drain system into equivalent plane strain condition. The predicted ground disturbance (smear effects) is considered with an idealized one zone. The influence of the smear zone will have reduced lateral permeability, which adversely affects soil consolidation. The behaviour of soft soils improved by vertical drains is analysed using a plane strain finite element method incorporating the constitutive model named S-CLAY1.

2 SITE CONDITIONS

In 1992, Queensland Department of Main Roads was commissioned to monitor and interpret the findings of a fully instrumented this trial embankment. Subsoil layer is composed of very soft, highly compressible, saturated organic marine clays of high sensitivity at this site. The trial embankment was constructed with three different ground improvement schemes (i.e. Section A: PVDs with 1m spacing, Section B: No PVDs and Section C: PVDs with 2m spacing). The embankment constructed approximately 90m in length and 40m in width and constructed in stages using a loosely compacted granular material ($\gamma_t \approx 19$ kN/m^3) up to a height of 2.3m (see Fig. 1). Berms were constructed to the design width of 5m on the instrumented side and 8m on the opposite side. A and B sections were the two primary sections of the trial embankment and each measured 35m in length and Section C, an intermediate case, was approximately 20m in length. A working platform 0.65m thick 0.5m thick drainage layer composed of 7mm size gravel. In this study, only Sections B and C were analysed. In the analysis, the soil profile was divided into 3 sublayers. The subsoil consists of a silty clay layer (2.5m depth) overlying very soft to soft silty clay extending from 2.5m to 5.5m depth. A 5.5m thick, medium silty clay layer underlies the soft silty clay layer. The groundwater level is at the ground surface. The vertical drains in both Sections A and C were installed in a triangular grid pattern. In Section C, PVDs were installed to a depth of 11m whereas a conventional surcharge without PVDs was constructed in Section B.



(a) Loading for embankment (b) Embankment cross section

Fig. 1. Sunshine Motorway Embankment

3 SCLAY-1 MODEL

The S-CLAY1 model was proposed by Wheeler et al. (2003). The model is an extension of conventional critical state models, with anisotropy of plastic behaviour represented through an inclined yield surface and a rotational component of hardening to model the development or erasure of fabric anisotropy during plastic straining. In the triaxial stress space for a cross anisotropic sample, the yield surface of the S-CLAY1 model can be expressed in terms of mean stress *p* and deviator stress *q*.

$$f = (q - \alpha p')^2 - (M^2 - \alpha^2)(p'_m - p')p' = 0 \quad (1)$$

where M is the value of the stress ratio $\eta = q / p'$ at critical states, p'_m defines the size of the yield curve and α defines the orientation of the yield curve. α is a measure of the degree of plastic anisotropy of the soil and α=0 the soil behaviour is isotropic. The model parameters can be obtained from the results of standard laboratory tests. The model assumes isotropic elastic behaviour and an associated flow rule. The S-CLAY1 model incorporates two hardening laws. One concerns changes in the size of the yield surface and the other concerns changes in the orientation of the yield surface. The former is the same as used in the MCC model, and the latter can be expressed as

$$d\alpha = \mu\left[\left(\frac{3\eta}{4} - \alpha\right)\langle d\varepsilon^p{}_v\rangle + \beta\left(\frac{\eta}{3} - \alpha\right)\left|d\varepsilon^p{}_d\right|\right] \quad (2)$$

4 SOIL PARAMETERS

The numerical analysis was based on constitutive S-CLAY1 model. Soil model incorporated in the finite element code, PLAXIS V. 8.6 (Brinkgreve and Vermeer, 1998). The mesh discretization with 15-node triangular elements is shown in Fig. 2. Half width of embankment was modelled. The adopted parameters of 3 subsoil layers obtained from standard laboratory tests are listed in Table 1.

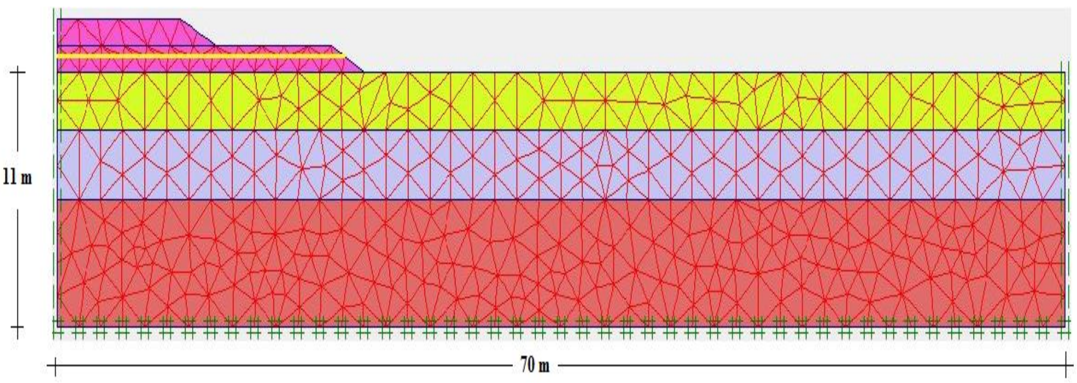


Fig. 2. Finite element mesh for plane strain analysis.

Table 1. SCLAY-1 Soil parameters for subsoil layers

Depth (m)	γ (kN/m³)	ν	ϕ`	e_o	κ	λ	M	η	α	β	μ
0.0-2.5	16.4	0.3	30	1.6	0.049	0.494	1.20	0.75	0.46	0.76	24
2.5-5.5	13.7	0.3	30	2.2	0.202	2.016	1.20	0.75	0.46	0.76	24
5.5-11.0	15.9	0.3	29.5	1.8	0.053	0.532	1.18	0.73	0.45	0.73	24

The prefabricated vertical drains were modelled with zero thickness drain elements (the excess pore pressure along this element is assumed to be zero). In this study, matching techniques proposed by Hird et al. (1992) is used. The matching technique represents the typical arrangement of vertical drains in plane strain finite element analyses. Geometric matching: the drain spacing is matched while maintaining the same permeability coefficient. The geometric matching was done according to the following equation in the absence of well resistance

$$\frac{B}{R}=\left\{\left(\frac{3}{2}\right)\left[\ln\left(\frac{R}{r_s}\right)+\left(\frac{k_h}{k_s}\right)\ln\left(\frac{r_s}{r_w}\right)-\left(\frac{3}{4}\right)\right]\right\}^{\frac{1}{2}} \tag{3}$$

B is the half width of the plane strain unit cell; R, r_w and r_s are radius of the axisymetric unit cell, the drain and the smear zone respectively; k_h and k_s are horizontal permeability of the undisturbed and smeared soil, respectively (Fig. 3). The equivalent permeability k_{pi} is calculated via the following equation (Hird et al. 1992).

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

Indraratna and Redana (1997) converted the vertical drain system into an equivalent parallel drain wall by adjusting the coefficient of soil permeability. They assumed that the half-widths of unit cell B, of drains b_w, and of smear zone b_s are the same as their axisymmetric radii R, r_w and r_s, respectively. Ignoring the well resistance, the equivalent permeability of the model is then determined by

$$\frac{k'_{hp}}{k_{hp}}=\frac{\beta}{\left[\ln\left(\frac{n}{s}\right)+\left(\frac{k_h}{k'_h}\right)\ln(s)-0.75-\alpha\right]} \tag{5}$$

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

$$\alpha=\frac{2}{3}\frac{(n-s)^3}{(n-1)n^2} \tag{6}$$

$$\beta=\frac{2}{3}\frac{(s-1)}{(n-1)n^2}\left[3n(n-s-1)+\left(s^2+s+1\right)\right] \tag{7}$$

where $n=R/r_w$ and $s=r_s/r_w$. In this study, the ratio between the horizontal and vertical permeability within the smear zone was set to 1. The permeability ratio between the undisturbed and the disturbed smear zone (k_h/k_s) is 2 and extent of smear zone (r_s) is 5 times the radius of the vertical drain (r_w) (Indraratna et al. 2007). Equivalent plane strain permeabilities of Section B and Section C are listed in Table 2.

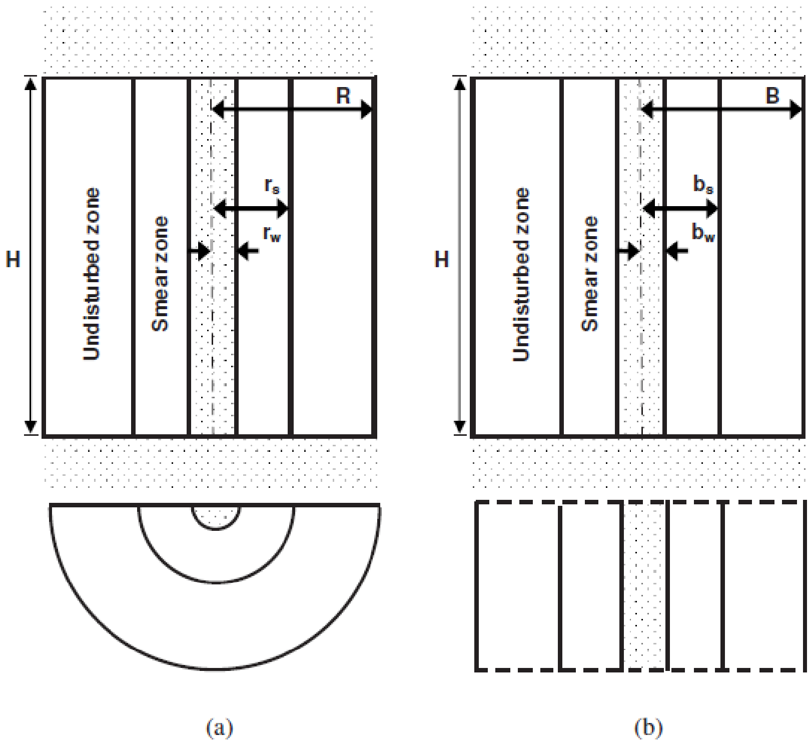


Fig. 3. Definition of symbols for unit cell analysis: (a) axisymmetric unit cell; (b) plane strain unit cell (Yildiz et al. 2009).

Table 2. Equivalent plane strain permeabilities of Section B and Section C

Depth (m)	Section B $k_h = k_v$ (m/day)	Section C k_h (m/day)
0.0-2.5	8.398e-4	1.19e-4
2.5-5.5	2.938e-5	4.17e-6
5.5-11.0	3.629e-5	5.15e-4

5 RESULTS of NUMERICAL ANALYSIS

The behaviours of the embankment on soft clay with PVDs and without PVDs were simulated using constitutive model SCLAY1. Section B was simulated without vertical drains and Section C was simulated with 2m spacing vertical drains on soft soil. The results of the numerical analyses were compared with the field measurements. The predicted and measured surface settlements are illustrated in Figs. 4 and 5. The predictions of the vertical displacements by the anisotropic model S-CLAY1 is good agreement with field observations for Sections B and C (Fig.4 and Fig.5). This result shows that the installation of vertical drains significantly decreases the settlement time. Predicted and measured excess pore pressure variation with time at the centre line of Section B at a depth of 4.0 m was presented in Fig. 6. It should be noted that for Section B the finite element analysis indicate very low dissipation of pore pressure. However the measured values in the field show substantially higher pore pressure dissipation settlement in Section B as well as greater pore pressure dissipation (Oh, E., 2006). The predicted lateral displacements versus depth underneath A point on the the embankment are compared with the 2D FEA results in Fig. 7. after 62 days. The maximum horizontal displacement predicted by FEA analysis is about 0.27 m. That would indicate, while the lateral deformation continue to take place during immediate settlement and the consolidation settlement seems more or less one dimensional in nature.

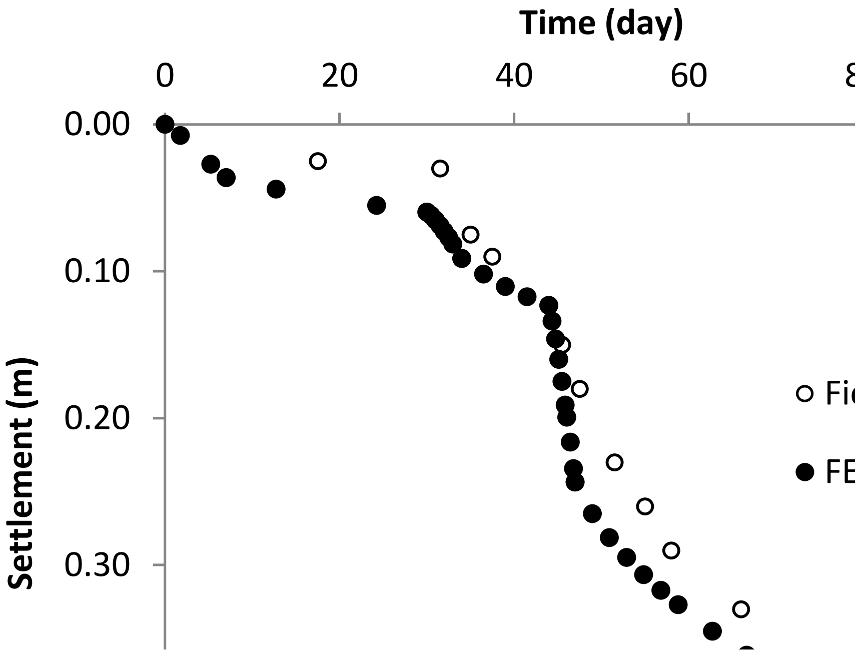


Fig. 4. Surface settlements at the embankment centreline for Section B (No drains)

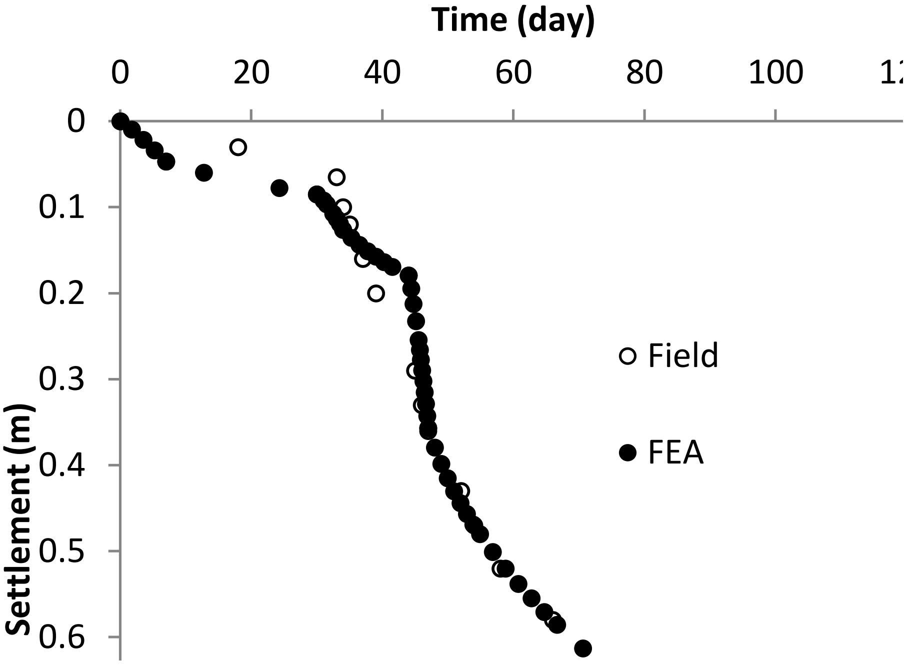


Fig.5. Surface settlements at the embankment centreline for Section C (2m drain spacing)

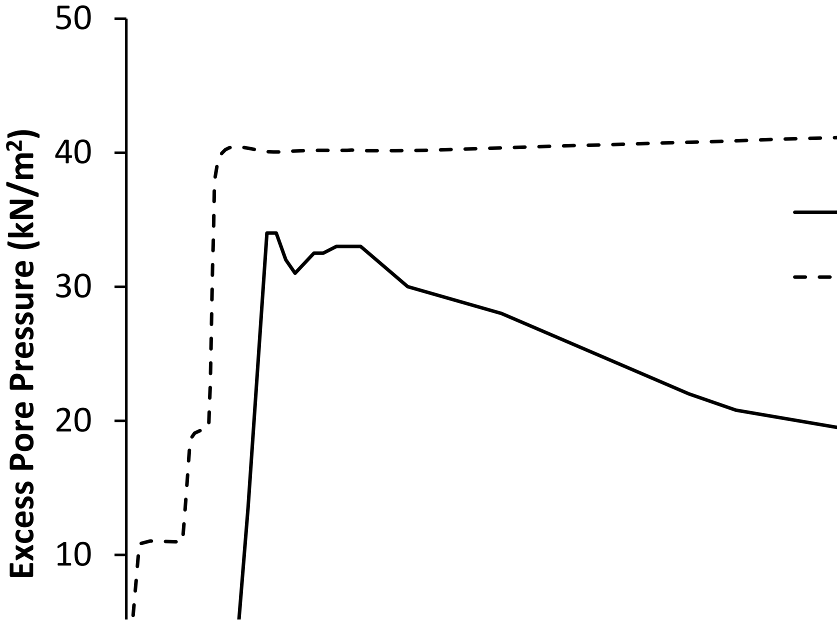


Fig. 6. Predicted and measured excess pore pressure variation with time the centre line of Section B at a depth of 4.0 m

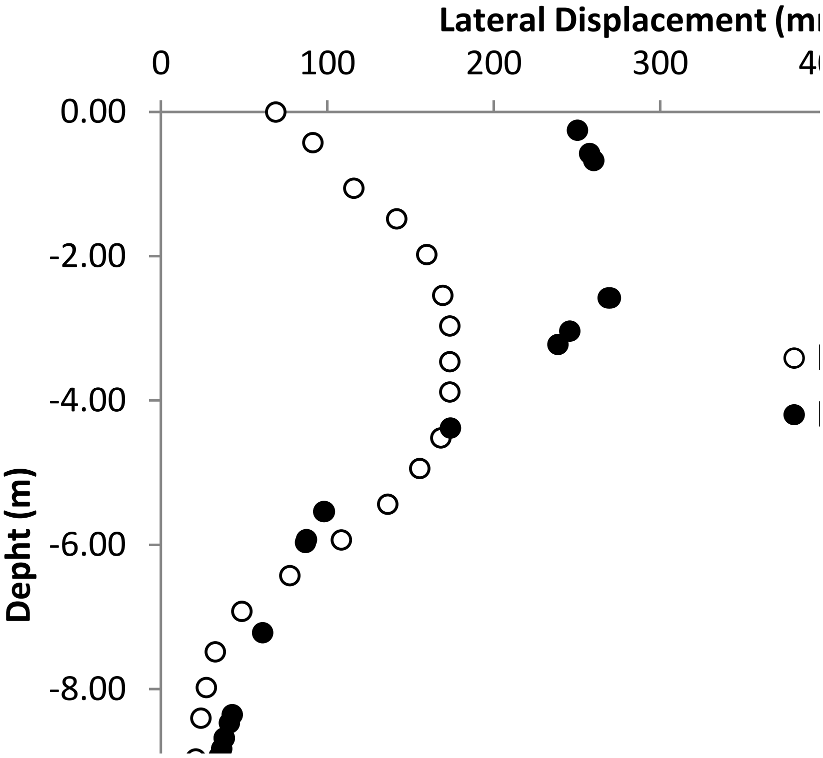


Fig. 7. Lateral displacements in Section B at A point

6 CONCLUSIONS

This paper presents 2D finite element analyses of an embankment with PVD and without PVD on soft clay. A recently developed elasto-plastic S-CLAY1 which accounts for plastic anisotropy and its extension is used to represent the soft soil. The results of the numerical analyses were compared with the field measurements. The numerical simulations demonstrate that the agreement between the finite-element predictions using the anisotropic constitutive model S-CLAY1 and the field observations is generally very good. 2D behaviour of vertical drains is converted into equivalent plane strain conditions with matching techniques proposed by Hird et al. (1992). The matching procedures proposed for the equivalent plane strain model were adopted in the study, based on the verification of the matching procedures with advanced model S-CLAY1. A multidrain analysis of the whole embankment on PVD-improved subsoil was performed using the combined matching procedure by Hird et al. (1992). The back analyses showed that the settlements calculated with the S-CLAY models agreed with the field measurements when r_s /r_w=5 (extent of smear zone over the radius of the vertical drain) and k_h /k_s=2 (The permeability ratio between the undisturbed and the disturbed smear zone). Further investigations should consider the creep effect, using the time dependent advanced soil models.

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