Influence of structure degradation on the behaviour of embankments on soft soil

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1. Introduction

As part of the upgrade of the Pacific Highway along the east coast of Australia, a substantial length of raised embankment is required to be constructed over areas of soft clay. Fully instrumented trial embankments have been constructed on estuarine clay deposits along the proposed alignment to acquire observational data on the settlement and stability of the highway embankments during and following the proposed construction. This provides a useful opportunity to verify finite element codes for elastoplastic consolidation analysis, which in turn can be used to analyse the performance of the highway embankments under varying design scenarios.
Compared with reconstituted clays in the laboratory, natural clays may generally exhibit extra strength and are able to exist at a higher void ratio than the equivalent reconstituted soils at a given stress ([1], [3]). Soft clays are an extreme example as the loading process may cause a breakdown of the initial structure and thus a destructuration process ([9]).
For the finite element analysis of the trial embankment on soft clay in this paper, [7] has previously adopted the Modified Cam-Clay model. However, due to the deficiency of Modified Cam-Clay in accounting for the structure in soft clay, some of the predictions do not agree very well with the field-monitored data. Thus a specific constitutive model that is capable of addressing the initial structure and destructuration process in soft clays is desirable for the analysis. Fortunately, several models have been proposed to rationalise the behaviour of natural soils (see, e.g., [2], [6], and [5]). In this paper, the Kinematic Hardening Structure Model proposed by [6] is employed to verify the settlement of the embankment again by the finite element method. The model parameters are determined from a set of field and laboratory data from soft soils in Eastern Australia. Coupled analysis of deformation and pore pressure evolution (see [8]) is adopted in the finite element implementation to predict the consolidation behaviour. The accuracy in settlement prediction using the consolidation theory of classical soil mechanics is also evaluated and compared with the finite element model predictions.

2. Kinematic hardening structure model (KHSM)

In this section, the Kinematic Hardening Structure Model (KHSM, [6]) is briefly described. This model has been developed to account for initial structures, small strain stiffness, stiffness degradation with strain history, and hysteretic response in cyclic loading. The response associated with the elastic part in the KHSM is expressed in terms of the bulk and shear moduli, K and G, which are assumed to depend linearly on the pressure p':

$$K = \frac{\partial p'}{\partial \varepsilon_v^e} = \frac{p'}{\kappa^*}, \qquad G = \frac{3(1-2\mu)}{2(1+\mu)}K \tag{1}$$

where ε_v^e denotes the elastic volumetric strain, κ^* is the slope of the swelling line in a volumetric strain-logarithmic mean stress compression plane rather than in a specific volume-logarithmic mean stress compression plane and μ is the Poisson's ratio.
The analytical equations of the reference surface, the bubble and the structure surface as well as the corresponding plastic potentials used in the KHSM are defined as follows:

$$
\left.\begin{array}{lll}
\text{Reference surface :} & f_r = g_r = \left(\dfrac{q}{M(\theta)p'_c}\right)^2 + \left(\dfrac{p'}{p'_c} - 1\right)^2 - 1 \\
\text{Bubble surface :} & f_b = g_b = \left(\dfrac{q - q_{\bar{\alpha}}}{M(\theta)p'_c}\right)^2 + \left(\dfrac{p'}{p'_c} - \dfrac{p'_{\bar{\alpha}}}{p'_c}\right)^2 - R_b^2 \\
\text{Structure surface :} & f_s = g_s = \left(\dfrac{q}{M(\theta)p'_c}\right)^2 + \left(\dfrac{p'}{p'_c} - R_s\right)^2 - R_s^2
\end{array}\right\}
$$

where the slope of the critical state line, M, is expressed as a function of the Lode angle θ, and determines the shape of the failure surface in the deviatoric plane. Accordingly, M can can be written as:

$$
M = M_{max}\left[\frac{2\alpha^4}{1 + \alpha^4 - (1 - \alpha^4)\sin 3\theta}\right]^{\frac{1}{4}} \tag{2}
$$

noindent By setting the parameter $\alpha = (3 - \sin\phi)/(3 + \sin\phi)$ the failure surface coincides with the Mohr-Coulomb hexagon at all vertices in the deviatoric plane (where ϕ is the friction angle of the soil at critical state), while setting $\alpha = 1$ recovers the Von Mises circle.

3. Finite element analysis

The finite element mesh used to represent the soils under the embankment is shown in Figure 1a. Due to symmetry, only half of the embankment is considered. The total area of the soil considered is 63 m wide and 31 m deep. The soil profile used in the analysis is shown in Figure 1a, with the bottom layer of firm clay extended to 31 m deep. The embankment is represented by vertical loads which increased over time to a constant value of 30 kPa from x=0 m to x=22 m and linearly decreasing to zero from x=22 m to x=27 m. Each grid square in Figure 1a consists of two triangular elements each of which has 6 displacement nodes and 3 pore pressure nodes. Nodes along the bottom boundary are restrained both vertically and horizontally, representing a rough boundary condition. The left and right boundaries are restrained in the horizontal directions, representing smooth contact vertically. The top boundary is set to be free drained to zero pore pressure, while the bottom boundary is set to be undrained. Four reference points on the ground surface, namely a(x=0 m), b(x=15 m), c(x=27 m) and d (x=34 m), are chosen to monitor the settlements of the trial embankment. Two other points, f (depth at 3 m or y=28 m) and h (depth at 5m or y=26 m), are chosen to monitor the excess pore pressures.
Appropriate optimization procedures have been used to obtain the model parameters for the finite element analysis

4. Results and discussions

Since construction of the embankment began, settlements and pore pressure data under the trial embankment have been measured over a 1300 day period. The predicted settlements at the ground surface and excess pore pressures at depths of 3 m and 5 m beneath the embankment centre are compared to the measured data in Figure 2 and Figure 3, respectively.
As can be seen from Figure 2, the finite element prediction of the time dependent settlements under the centre of the embankment (Reference Point a, x=0 m) agrees very well with the measured data. Only slightly smaller settlements are predicted by FEM than measured after 20 days and around 100 days. These differences are no larger than 10%. Generally, the predicted settlements by the finite element analysis using the Kinematic Hardening Structure model, with parameters derived from triaxial and oedometer tests, are relatively accurate.
Due to difficulties in matching the boundary condition at the ground surface, and the fluctuating ground water table in the FEM computations with the actual behaviours, the computed and measured absolute values of excess pore pressures cannot be directly compared. Thus in

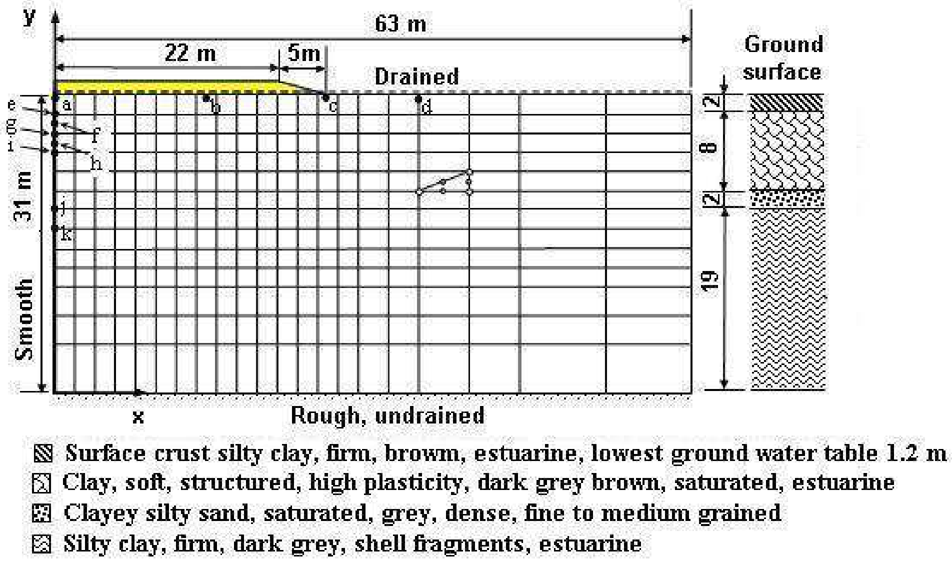

FIGURE 1. Teven Road trial embankment: Finite element mesh and typical soil profiles.

Figure 3 the predicted and measured excess pore pressures are normalised against their maximum values respectively, resulting in a plot of the percentages of generation and dissipation of the excess pore pressures. It is shown that the times at which the maximum excess pore pressures occur are well predicted. At about 1000 days since the construction of the embankment, around 60% of the excess pore pressure generated at a depth of 3 m has been dissipated according to the field data, while the finite element analysis predicts a dissipation of around 20%. The predicted dissipation of the excess pore pressure generated at a depth of 5 m is about 20% less than the field data at a time of around 1000 days. In general, compared to the settlements, the excess pore pressures predicted by the finite element method appear much less accurate, which is in accordance with those reported by Wroth and Simpson (1979).

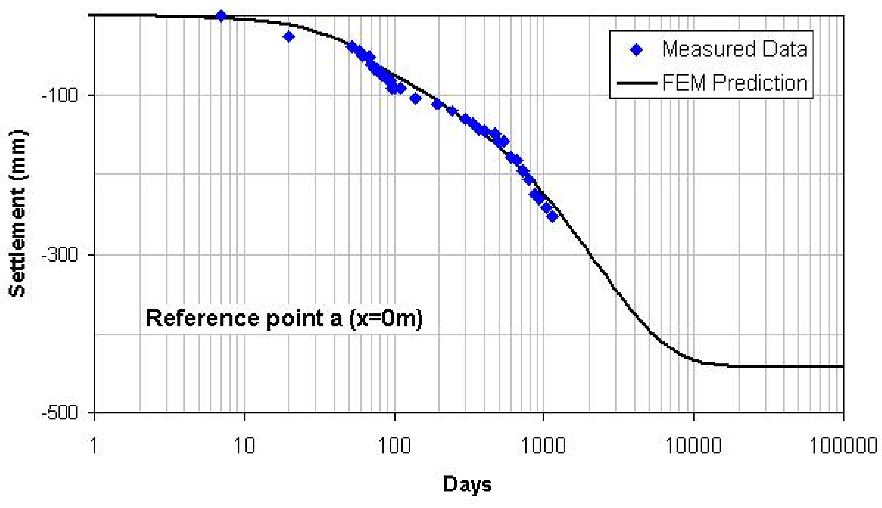

FIGURE 2. Teven Road trial embankment: Measured and predicted settlements for reference point a.

5. Conclusions

The performance of a fully instrumented trial highway embankment on thick estuarine clay deposits is studied using the finite element method and the Kinematic Hardening Structure Model (KHSM). The performance of the numerical model in predicting the load response behaviour of soft soils found on the East Coast of Australia is evaluated. The settlements predicted by the KHSM compare well with the data obtained from the embankment site.

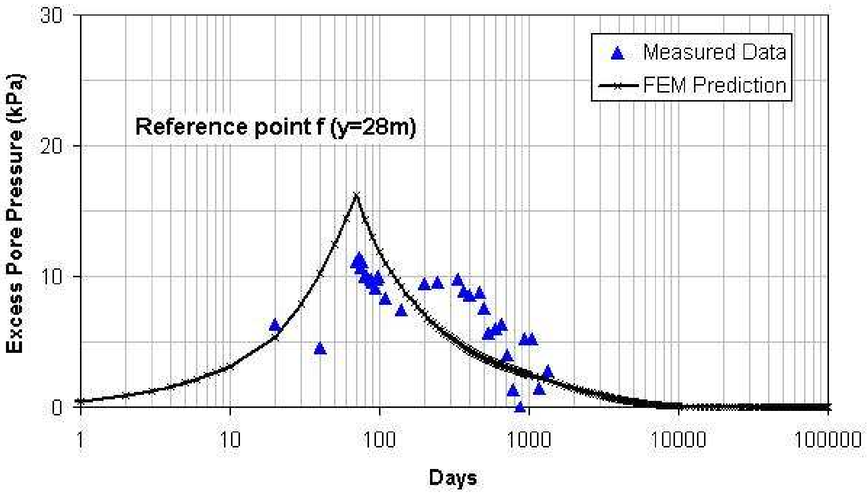

FIGURE 3. Teven Road trial embankment: Comparison between measured and predicted excess pore water pressure beneath the centre of the embankment.

However, the predicted evolution of excess pore pressures by the finite element model does not agree well with the field data. The numerical model tends to under-predict the dissipation of excess pore pressures, even though the predicted rate of the settlement is relatively accurate. This discrepancy is attributed partly to the highly variable boundary conditions in situ and partly to the inadequacy of the soil model. This paper also shows the settlement and the dissipation of excess pore pressure are closely related to the destructuration processes in soft clays. This proves the necessity of using constitutive models that take into account the effects of damage to structure to analyse geotechnical problems related to natural soft soils.

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