Settlement analysis of foundation soil over a long time and comparison with field performance

Creep analyse des sols de fondation sur de longues périodes et la comparaison avec le champ de performance

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ABSTRACT

This paper models the consolidation of the foundation soil of a wide geogrid reinforced embankment close to its centre-line. An elastic viscoplastic model has been used for the analysis. A creep function that takes into account of the non-linear nature of creep has also been incorporated in this model. The predicted results are compared with the field measurement data and with the analysis results obtained using two other models (i.e. Kutter and Sathialingham, 1992 and modified Cam-Clay).

RÉSUMÉ

Cette étude de modèles de la consolidation des sols de fondation d'un grand géogrille renforcé remblai près de la ligne. Un élastique viscoplastique modèle a été utilisé pour l'analyse. Une fonction de fluage, qui tient compte de la nature non linéaire du fluage a également été intégrée dans ce modèle. Les prévisions de résultats sont comparés avec les données de mesure et l'analyse des résultats obtenus à l'aide de deux autres modèles (c'est-à-dire et Sathialingham Kutter, 1992 et modifié Cam-Clay).

Keywords : Soft Clay, Creep, Embankment, Consolidation, time dependent behaviour

1 INTRODUCTION

The stress-stain behaviour of clayey soils is non-linear, irreversible and time dependent. The design of structures, directly and indirectly, on the clayey soils needs good understanding and modelling of the time-dependent stress-stain behaviour of the soils (Yin 2001).

In general there are two types of time dependent behaviour of soils. One is related to the dissipation of pore water pressure and is also known as consolidation and the other one is related to the creep or rate dependent nature of soil. Elasto-plastic models like Modified Cam Clay (MCC) can model the first type of time dependent behaviour when used in a coupled form but is deficient in modelling the second type.

Based on Bjerrum's (1967) concept of delayed compression and Perzyna's (1966; Perzyna 1963) formulation of viscoplasticity, Kutter and Sathialingam (1992) proposed a simple Elastic-Visco-Plastic (EVP) model to describe the time dependent behaviour (secondary compression or creep or stress relaxation or rate dependent behaviour) of soil. The creep coefficient in the model was treated as a constant and the shape of the Critical State Surface (CSS) and the Yield Surface (YS) in the octahedral plane (π-plane) was considered to be a circle. However, the failure of soil is known to better follow the Mohr-Coulomb failure criteria (Britto & Gunn 1987; Yin 2001). Karim and Gnanendran (2008) proposed a modified version of the Kutter and Sathialingam (1992) model in which they treated the shape of the yield surface in π-plane to be a distorted hexagon. This model will be used in the present analysis.

A number of studies have revealed that creep occurs at higher rate initially and becomes slower with time (Yin 2001; Kutter & Sathialingam 1992; Yin et al. 2002). According to many researchers (Berre & Iversen 1972; Leroueil et al. 1985) the creep coefficient may not be a constant. To estimate creep, a logarithmic function is generally used to fit the oedometer test data. The use of a constant creep coefficient sometimes might lead to misleading results (Yin 1999). Karim et al. (2009) proposed an exponential function (as in equation 1) to capture the non-linear nature of the creep coefficient. This exponential function allows the creep coefficient to vary with some state parameters,

$$C_\alpha^* = C_{\alpha\max} \times \exp\left(-N \times \left(\overline{P}_0 - P_L\right)\right) \tag{1}$$

where C_α^* is the tangential creep coefficient at any time (slope of the tangent at any point of the void ratio vs. log(time) plot), $C_{\alpha\max}$ and N are positive constants, $\overline{P}_0$ is the creep-inclusive pre-consolidation stress, as explained in Kutter and Sathialingham (1992) which can be calculated using the following equation,

$$\overline{P}_0 = \exp\left(\frac{e_N - e - \kappa \ln P}{\lambda - \kappa}\right) \tag{2}$$

and P_L is the creep-exclusive pre-consolidation pressure which can be calculated using the MCC equations,

$$P_L = P + \frac{q^2}{PM^2} \tag{3}$$

In the above equations, e is the current void ratio of the soil, e_N is the void ratio at the unit mean normal stress on the normal consolidation line, λ and κ are the MCC compression and recompression indices respectively and P and q are the mean normal effective stress and the deviatoric stress respectively.

In this paper, the above creep function will be used in association with the Karim and Gnanendran (2008) model to analyse the long term settlement and pore water pressure response of the soil near the centre-line of a wide geogrid reinforced embankment (Leneghan Embankment). We will start with a brief description of Leneghan embankment.

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2 LENEGHAN EMBANKMENT

Lo et al. (2008) presented a detailed description of Leneghan embankment in terms of soil properties, geogrid reinforcement used, construction sequence adopted and field instrumentations. A brief description is presented here for easy understanding of this paper.

The Minimi to Beresfield extension of the Sydney-Newcastle Freeway was constructed in the 1990's by the Road and Traffic Authority (RTA), New South Wales (NSW), Australia, and is located 150 km north of Sydney. This project involved the construction of embankments over three swamps near the Leneghan Drive. The middle one, referred to as the Leneghan embankment in this paper, posed the greatest geotechnical challenges. It was about 300 m long, 60 m wide at ground level and 32 m wide at the crest and was constructed to a Reduced Level (RL) of 5.5 m.

The sub soil consisted of mainly very soft to soft alluvial clay of about 16 m thickness. The top three meters of the clay layer was found to be firm and over-consolidated. Due to soft and compressible nature of the soil, a number of measures were taken to confirm the stability of the embankment. This included the use of wide stabilizing berms, light-weight fill materials, staged construction and surcharging followed by removal of the surcharge. In order to accelerate consolidation settlement, Prefabricated Vertical Drains (PVDs) were installed throughout the soft clay layers. To ensure stability, the embankment was constructed in three stages, allowing rest periods between them. For more detailed description of the embankment construction, instrumentation and material properties, readers are referred to Lo et al. (2008) and Karim et al. (2009).

3 DETERMINATION OF MODEL PARAMETERS

The Karim and Gnanendran (2008) model requires 7 parameters for a complete description of stress-strain behaviour of soil. Six of them are the conventional Cam-Clay parameters and the 7^{th} one is the creep coefficient (which can be taken as either a constant or as a variable). To conduct a coupled-consolidation analysis, the hydraulic permeability values are also required.

The Cam-Clay parameters adopted for the consolidation analysis are listed in Table- 1. These parameters were determined from a comprehensive laboratory testing program and are explained in Lo et al. (2008).

Table 1. The MCC Material parameters

RL of emb.	M	λ	κ	υ	e_N
+0.3 to-2.8 m		0.33	0.066		2.877
-2.8 to-10m	1.11	0.38	0.076	0.3	3.291
-10 to -16 m		0.33	0.066		2.877

3.1 *Creep Parameters*

The creep parameters for the EVP model discussed earlier were taken from Karim et al. (2009) and were determined by long term (up to 3 months) oedometer testing. The coefficients of Equation (1) were determined to be $C_{\alpha max} = 0.08$ and N = 0.027. Using the same test data, the constant creep coefficient was found out to be 0.031 (based on the first two weeks of test data).

3.2 *Hydraulic Permeability*

The horizontal hydraulic permeability in this analysis will be controlling the consolidation. As shown by Lo et al. (2008) and Karim et al. (2009), the horizontal hydraulic permeability of the soil can be taken as a constant multiple of the vertical hydraulic permeability. The vertical hydraulic permeability was found to vary with void ratio according to the equation proposed by Taylor (1948) as below (see Karim et al. 2009),

$$\log K = \log K_i - \frac{e_0 - e}{C_k} \tag{4}$$

where K_i is the reference hydraulic permeability at the reference void ratio of e_0 and C_k is the slope of the K – e graph (in a semi-log plot with K on vertical axes).

By analysing the oedometer test data, the coefficients of the equation (for vertical permeability) was found to be $C_k = 0.83$ and $K_{vi} = 1.5E-5$ (suffix v indicates the hydraulic permeability in the vertical direction) for a reference void ratio of $e_0 = 1.78$ (Karim et al. 2009).

The multiplier (K_h/K_v) was obtained by back-analysing the first 12 months of settlement data with the unit-cell analysis. In essence, the multiplier was varied systematically until a best match was obtained. The back calculated value of the multiplier was found to be 1.98 (Karim et al. 2009).

4 NUMERICAL IMPLEMENTATION

The EVP models discussed earlier was implemented in the UNSW@ADFA modified version of AFENA (originally developed by Carter and Balaam 1995). Axi symmetric unit cell analyses, adopting the three different models (MCC, Kutter and Sathialingam 1992 and Karim and Gnanendran 2008) for the foundation soil, were carried out for predicting the settlement and excess pore pressures in the centre-line region of the wide Leneghan embankment. To model the foundation soil in the unit cell analysis, 192 six nodded linear strain triangular elements, with a total of 429 nodes, were used along with three sets of input soil parameters to represent the three different layers of soil. The inner vertical boundary (representing the periphery of the drain) and the top layers were modelled as free draining and the outer vertical boundary as impermeable. The bottom boundary was modelled as impermeable based on the observed excess pore water pressure response at piezometer P2.13 (Figure 1 in Lo et al. 2008). Geogrid was omitted in this analysis and this might have given a small error on the conservative side.

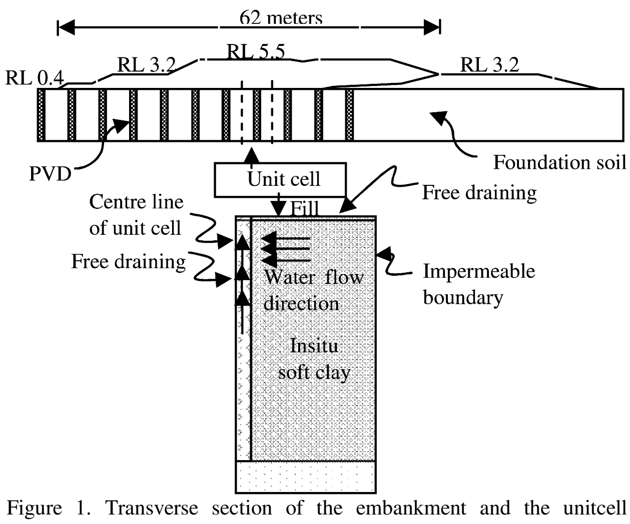

Figure 1. Transverse section of the embankment and the unitcell idealisation (not drawn according to scale).

Embankment loading was applied by uniformly distributed forces in incremental steps so as to simulate the construction history. Since the embankment was constructed to a specific RL, the extra embankment weight, due to settlement and construction to a specific RL, was also included in the boundary forces. The total embankment loading so calculated agreed with the earth pressure cell readings presented in Lo et al. (2008). At the end of 20 months (construction reached a RL of 5.5 m in 12 months), 1 m of extra fill was imposed for 8 months and then excavated back to the specified RL of 5.5 m. This preloading process was also simulated in the analysis which was carried out using approximately 20,000 time steps.

5 SYNTHESIS OF RESULTS

Following figures present the analysis results from three different analysis approaches namely MCC analysis (from Karim et al. 2009), EVP Analysis- 1 with a constant creep coefficient and Kutter and Sathialingam (1992) model (from Karim et al. 2009) and EVP Analysis- 2 with a varying creep coefficient (as proposed in Karim et al. 2009) and Karim and Gnanendran (2008) model.

Figure 2 presents the settlement plotted against time as predicted by three different analyses. As expected the MCC analysis under-predicted the final settlement by about 18%. The reason for it is obvious. The creep or rate dependent behaviour of soil was not taken into account. However the first 400 days of settlement was predicted with reasonable accuracy. This might be attributed to the fact that the hydraulic permeability was back calculated using first one year of settlement data. The predicted settlement started to under predict the field values after that.

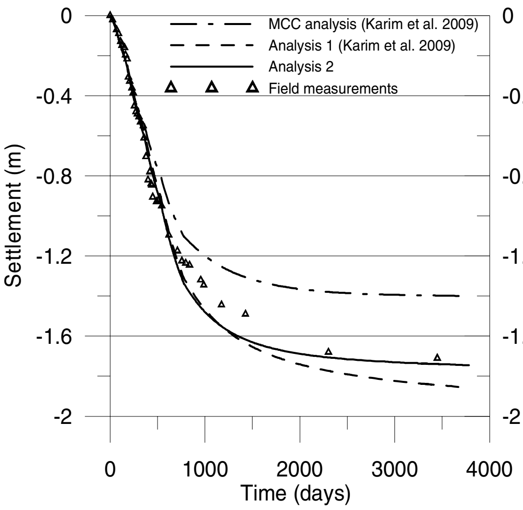

Figure 2. Time vs. settlement from three different analyses.

EVP Analysis- 1 predicts the field settlement with reasonable accuracy for up to about 700 days and after that it starts to deviate on the conservative side. The predicted settlement kept on deviating from the field recorded values and the final settlement was over predicted by about little less than 10%. The reason behind it may be the use of constant creep coefficient. In the field, the creep was occurring at a much slower rate during later stages of the consolidation whereas the use of a constant creep coefficient did not account for this phenomenon accurately.

The predicted settlement by EVP analysis- 2 closely followed the EVP Analysis- 1 prediction up until about 1200 days and after that it started to move towards the field measurements. It might appear perplexing that the two analyses approaches (EVP Analysis- 1 and EVP Analysis- 2) in spite of using different creep parameters predicted very similar settlement up till about 1200 days. This might be due to the fact that the settlement behaviour up till that period was dominated by hydrodynamic lag. The difference between the two analyses approaches became apparent only at a point in time when the process was no more dominated by excess pore water pressure dissipation.

Figures 3 and 4 presents the pore water pressure responses, as predicted by the three different analysis approaches and the corresponding field measurements, at two different depths (i.e. RL -4.5 m and RL -7.5 m respectively). The overall predictions from all the three analyses are with reasonable accuracy. The EVP Analyses- 1 and 2 predictions traced the upper boundary of the field excess pore water pressure measurements. Whereas, the MCC analysis predicted the average excess pore water pressure, the piezometers installed in the field have a tendency to move towards the drains and as a result, some times record lower excess pore water pressure. Keeping this in mind it can be said that the excess pore water pressure might have been under predicted by the MCC analysis.

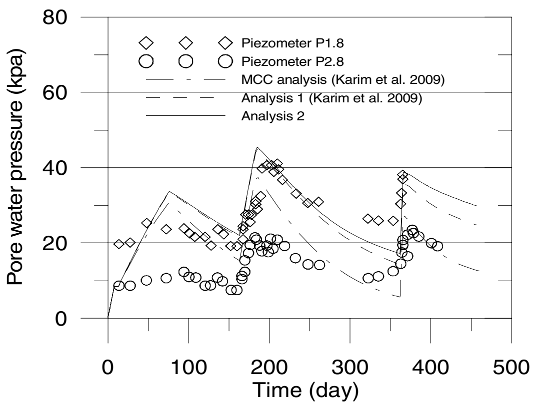

Figure 3. Excess pore water pressure response (field values and analysis results) at RL -4.5 m.

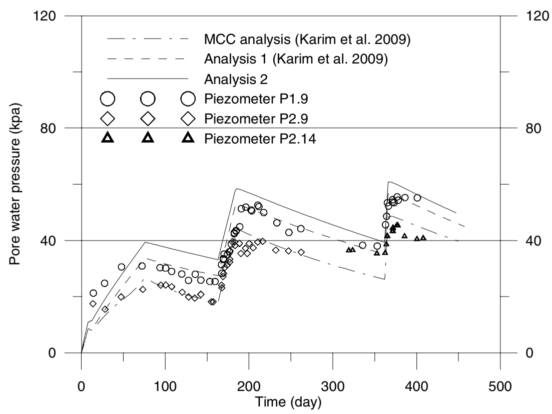

Figure 4. Excess pore water pressure response (field values and analysis results) at RL -7.5 m.

The EVP analysis- 2 consistently predicted higher excess pore water pressure than the other two analysis approaches and that might be due having lower hydraulic permeability than the other two analyses. It was closely followed by the Analysis- 1 results.

It might be interesting to note that the three different analyses used different values for (back calculated) hydraulic permeability. The EVP analysis- 1 used a K_h/K_v ratio of 2.04 whereas for the MCC analysis this ratio was 3.32 (Karim et al. 2009). It might be concluded form this that the value of hydraulic permeability is model dependent and that is because of different stress-strain relationships in different models. The effect of using different hydraulic permeability was also reflected in the excess pore water pressure response of the foundation soil as the model that used higher hydraulic permeability predicted less excess pore water pressure.

6 CONCLUSIONS

Though the idealisation of the axisymmetric analysis is inherently biased towards predicting the settlement on the conservative side, the MCC model under predicted the final settlement significantly. The use of an EVP model predicts the settlement with reasonable accuracy. The use of a constant creep coefficient led to significant over prediction of the settlement. The use of a non-linear function for the creep coefficient helped to predict the final settlement of Leneghan embankment near the centre-line with higher accuracy. The necessity of using an appropriate model that account for creep or time dependency is obvious form this observation.

All the three analyses predicted the excess pore water pressure with reasonable accuracy. However taking into account the tendency of the piezometers installed in the field to move towards the PVDs, the MCC analysis might have under predicted the excess pore water pressure response. Another interesting finding is that the value of hydraulic permeability is model dependent.

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