Long-term performance of preloaded road embankment

Comportement à long terme d'un remblai routier préchargé

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ABSTRACT: The results from an investigation into the long-term performance of the preloaded Nerang-Broadbeach Roadway (NBR) embankment near the Gold Coast in Queensland, Australia, are presented in this paper. The soil profile along this roadway consists of deep Cainozoic estuarine alluvial soft clay deposit. To predict the performance of the preloaded embankment, two fully coupled nonlinear Finite Element Analyses (FEA) were conducted adopting an elasto-viscoplastic (EVP) and an elasto-plastic Modified Cam Clay (MCC) model to represent the soft clay using the UNSW Canberra modified version of the nonlinear stress analysis program AFENA. It was found that the MCC model under-predicted the ultimate settlement while the creep-based EVP model captured it well but over-predicted the pore pressure response. Observational approaches using the Asaoka and Hyperbolic methods were also applied from which it was observed that, when the soft soil exhibited creep, after a certain cutoff time increment (Δt), the Asaoka plot became parallel to the 45° line and the settlement prediction was unrealistic compared with the field measurement. After a modification was introduced into the Asaoka method for creep-susceptible soil, the predicted settlement was found to be in good agreement with that obtained from Hyperbolic method and the details are presented in the paper.

RÉSUMÉ : Les résultats d'une enquête sur le comportement à long terme de la route Nerang-Broadbeach (NBR), en remblai ayant fait l'objet d'un préchargement près de la Gold Coast dans le Queensland (Australie) sont présentés dans ce papier. Le profil de sol le long de cette chaussée se compose de dépôts profonds d'argile molle marine de l'horizon alluvial « Cainozoic ». Afin de prédire la performance du remblai préchargé, deux analyses couplées aux élément finis non linéaires (FEA) ont été réalisées en adoptant un modèle élasto-viscoplastique (EVP) et un modèle élasto-plastique modifié du type Cam Clay (MCC) ; ces modèles ont été choisis pour représenter l'argile molle en utilisant la version modifiée de l'analyse en contrainte non linéaire du programme AFENA. Il a été constaté que le modèle MCC a sous estimé les tassements ultimes, tandis que le modèle EVP basé sur le fluage a bien évalué ces tassements mais a surestimé la réponse en terme de pression interstitielle. Les approches observationnelles utilisant la méthode d'Asaoka ou la méthode hyperbolique ont été aussi utilisées ; il a été constaté que lorsque le sol exhibe du fluage, après un certain seuil dans l'incrément de temps (Δt), la courbe d'Asaoka devient parallèle à la droite à 45° et la prédiction de tassement devient irréaliste par rapport aux mesures sur le terrain. Après qu'une modification ait été introduite dans cette méthode d'Asaoka pour les sols sujets au fluage, le tassement prédit a été trouvé en bon accord avec celui tiré de la méthode hyperbolique.

KEYWORDS: Preload, soft clay, Modified Cam Clay model, Elasto-viscoplastic model, creep, Asaoka method, Hyperbolic method.

1 INTRODUCTION

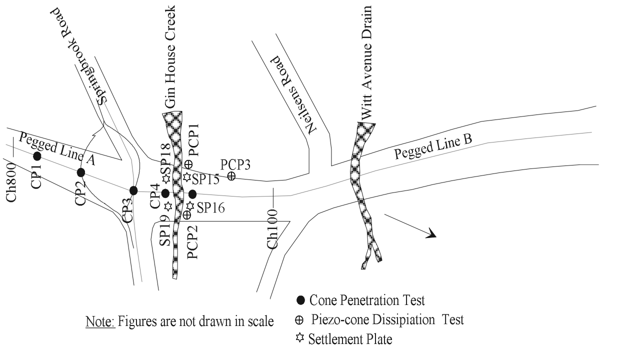
The Nerang-Broadbeach Roadway (NBR) was constructed by the Queensland Department of Transport and Main Roads (QDTMR) and completed in 2001. It is located closer to the Gold Coast Highway in the South part of Surfers Paradise, Gold Coast, Queensland, Australia. It was constructed to accommodate the region's transport network and enhance road safety. The roadway embankment was founded on deep Cainozoic estuarine alluvial, soft sensitive deposits of thicknesses from 5 to 21 m overlaying greywackes and argillite bedrock. This estuarine deposit is highly compressible, exhibits low bearing capacity and undergoes extensive time-dependent settlement when subjected to extrinsic loads. Although there are several techniques for accelerating the ongoing settlement of estuarine clay and to mitigate post-construction damage, preloading in conjunction with surcharging has been proven to be one of the most efficient ground improvement techniques for estuarine clay in the Queensland region (Islam et al. 2012, 2013). The NBR was divided into five distinct preloading embankment sections: *North of Main Drain; Main Drain to Meadow Drive; Meadow Drive to Witt Ave Drain; South of Witt Avenue Drain; and Gin House Creek (Fig. 1)*. Performance of the embankment section located in between *Gin House Creek* and *Witt Avenue Drain* and nearer to settlement plate SP18 that had a preloading height of 3 m is examined in this paper.

Field monitoring data (measured settlement and excess pore water pressure) of this embankment was compared with the corresponding predicted responses obtained from Finite Element Analysis (FEA). In particular, nonlinear fully coupled FEAs were carried out adopting a creep-based elastic-viscoplastic (EVP) model and Modified Cam Clay (MCC) elasto-plastic model for the foundation soil from which it was found that the creep-based EVP model captured the field settlement of the embankment better than the MCC model but over-estimated the excess pore water pressure. The ultimate settlement was estimated using Asaoka's and Hyperbolic observational methods in this study. Since the foundation soft soil exhibited creep, after a certain cutoff time increment (Δt), Asaoka plot became parallel to the 45° line and the predicted settlement was unrealistic compared to those obtained from FEA using the creep-based EVP model as well as Hyperbolic method. Therefore, some modification was necessary for the Asaoka method for capturing the ultimate settlement of creep-susceptible foundation soil and it is the focus of this paper.

2 SUBSURFACE CONDITIONS OF SITE

To delineate the subsurface conditions of the NBR, two subsoil investigations were carried out by the QDTMR, in 1991 (Main Roads 1991) and 1999 (Main Roads 1999), from which a poor subsoil strata was identified. This led to further investigations of QDTMR in 2000 (Main Roads 2000) and 2001 (Main Roads 2001) which included six borehole tests, twenty electric cone penetrometer tests (CPT) and four piezocone dissipation tests (CPT-u). The reasons behind the boreholes were to obtain undisturbed soil samples for laboratory testing and to conduct further in-situ field testing. The CPT and CPT-u tests were conducted to profile the soil layer and to determine the potential sand lenses as well as to correlate the parameters. Field responses were monitored using settlement plates and piezometers. In this paper, the data from a settlement plate (SP18 – See Fig. 1a) and a piezometer are used for assessing the predictability of the performance. The site plan, locations of field tests and instrumentation, along with the depths of the clay deposits, are shown in Fig. 1.

The subsoil consisted of moisture contents from 24 to 101%, liquid limits of 35 to 68% and plasticity index of 15 to 33%. The saturated unit weight of the estuarine soft clay varied from 14.43 to 20 kN/m^3. The sensitivity of the soil ranged between 3.75 to 7.00 and the undrained shear strengths of the clay deposits obtained from field vane shear tests from 30 to 92 kN/m^3. The soil properties of the NBR site are shown in Fig. 2.



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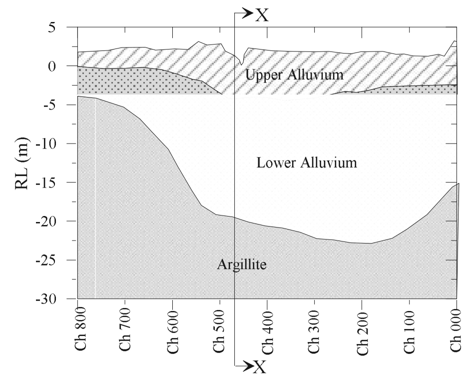


(b)

Figure 1. (a) Site plan and (b) Soil profile along NBR

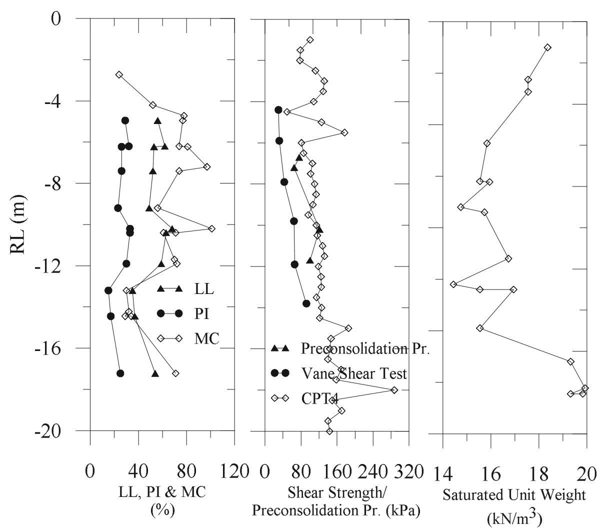


Figure 2. Foundation soil properties of NBR

The soil profile along the NBR area comprised alluvium overlying bedrock in two distinct strata: upper alluvium and lower alluvium. The upper alluvium consisted of a 2m thick topsoil or silty clay overlying 2 m of loose sand. Depending on the physical properties and compressibility characteristics; the lower alluvium was divided into three distinct layers: Clay-1 (7 m); Clay-2 (5 m); and Clay-3 (5 m). It was observed that, in the Clay-1 layers, the organic component was 8.4 %.

3 FINITE ELEMENT ANALYSIS

Fully coupled, elasto-plastic (Roscoe and Burland, 1968) and elasto-viscoplastic (Karim et al., 2010) nonlinear FEA of the NBR embankment were carried out considering plane strain analyses using a UNSW Canberra, modified version of the FEA program AFENA (Carter and Ballam 1995). Due to the symmetry of the embankment section to reduce computational time, only half part of the embankment was considered for analysis.

The soft soil was initially modelled as an elasto-plastic MCC material and the results were compared with those subsequently obtained adopting the creep-based EVP model. The sand layer in the foundation soil, argillite bed rock and embankment fill materials were modelled as elastic perfectly plastic materials using the Mohr-Coulomb failure criterion.

Consolidation parameters (λ and κ) were calculated from a 1-D consolidation test data and the strength parameter (φ or M) estimated from the correlation of the CPT and CPT-u tests. The flow parameter (co-efficient of permeability) was back-calculated from the CPT-u test data using the relationship proposed in Teh and Houlsby 1991 and Karim et al. 2010. For the CPT and CPT-u test data interpretations CPeT-IT 2012 were used. The void ratio (e_N) of the in situ soil at the unit mean-normal effective stress on the normal consolidation line, the preconsolidation pressure (p_{c0}) and conventional secondary consolidation co-efficient (C_α) were calculated from a 1-D consolidation of the test data. The model parameters used in MCC and EVP models are tabulated in Tab. 1.

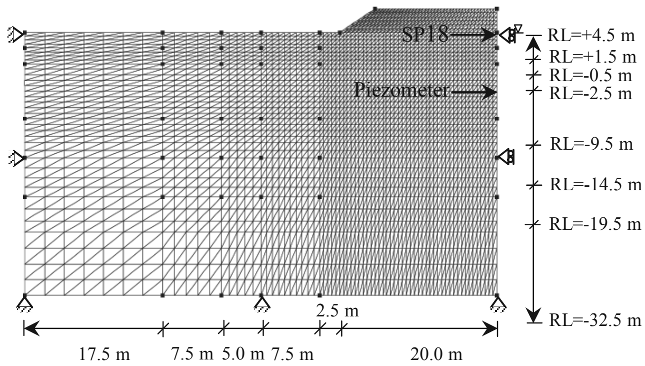


Figure 3. FE geometry used for 2D plane strain analysis (X-X section)

The length and width of the embankment section were 1.3 km and 40 m respectively, with the height of fill materials and depth of its foundation 3 m and 21 m respectively. The construction period of the embankment was 15 days. The settlement plate was placed at RL = + 1.5 m on the centre line of the embankment to monitor the ongoing field settlement and the piezometer at RL = - 4.6 m to monitor the field's excess pore water pressure. The side slope of the embankment is 1V: 2H.

The finite element mesh consisted of 11,267 nodes and 5,520 elements with six noded nonlinear triangular elements used for finite element discretisation. It was observed from the FEAs that the predicted settlements from the MCC and EVP models were 425.55 mm and 498.00 mm respectively for 360 days. On the other hand, the measured settlement for the same time duration was 478.00 mm. It is evident that the MCC model under-predicted the settlement which may have been due to ongoing creep settlement.

Table 1. Material parameters used in analyses

RL (m)	M	λ	κ	e_N	K_0	p_{c0} * (kPa)	Vertical permeability coefficients $^{\otimes}$		
							K_i (m/day)	e_0	C_k
Fill Materials	---	---	---	---	0.42	$E' = 3000$ kPa, $\varphi' = 35°$, $c' = 5.0$ kPa			
+1.5 to -0.5	1.51	0.43	0.043	4.10	0.40	50.00	2.50×10^{-5}	1.70	1.00
-0.5 to -2.5	---	---	---	---	0.50	$E' = 5000$ kPa, $\varphi' = 32°$, $c' = 4.0$ kPa			
-2.5 to -9.5	1.33	0.39	0.062	3.85	0.46	80.00	2.5×10^{-5}	1.70	1.00
-9.5 to -14.5	1.20	0.23	0.030	2.70	0.50	112.00	2.5×10^{-5}	1.70	1.00
-14.5 to -19.5	1.07	0.13	0.013	2.51	0.55	114.00‡	2.5×10^{-5}	1.70	1.00
-19.5 to -32.5	---	---	---	---	0.42	$E' = 15000$ kPa, $\varphi' = 35°$, $c' = 50.0$ kPa			

Notes: Poisson's ratio considered 0.3
* At top of soil layer
$^{\otimes}$ From 1D consolidation tests
‡ Gradient of p_{c0} after -14.5 m 5.5 kPa/m of depth

The excess pore water pressure was monitored for 217 days and observed to be better predicted by the MCC than EVP model. Up to 73 days, the MCC model captured the measured excess pore water pressure well but then started to over-predict it.

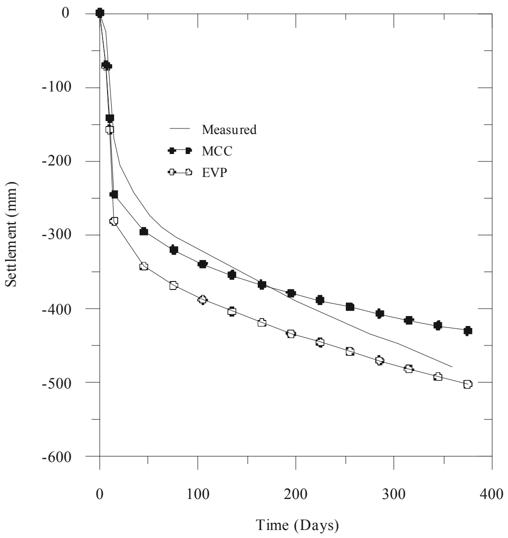


Figure 4. Comparison of measured and predicted settlements

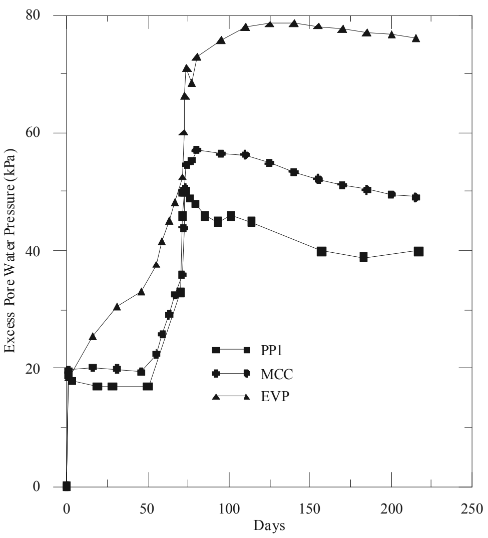


Figure 5. Comparison of measured and predicted excess pore water pressures

4 OBSERVATIONAL APPROACH

Observational approaches, such as the Asaoka (1978) and Hyperbolic (Tan 1995) methods, allow predictions of the ultimate settlement of estuarine clay. In Asaoka (1978) method, settlement (ρ_t) at any time (*t*) can be expressed as a linear plot defined by Eqn. 1 and the ultimate settlement by Eqn 2.

$$\rho_t = \beta_0 + \beta_1 \rho_{t-1} \tag{1}$$

$$\rho_{ult} = \frac{\beta_0}{1-\beta_1} \tag{2}$$

where β_0, β_1 are the co-efficients representing the intercept and slope of the fitted straight line proposed by Asaoka (1978) respectively, and the intercept point of the fitted line and 45° lines stands for the ultimate settlement. Applicability of Asaoka method for predicting the creep-included settlement of soft clays has been questioned previously (Islam et al. 2012, Lansivaara 2003). Moreover, effectiveness of the Asaoka method is biased by the selection of the time interval (Δt). For these reasons, in the present study, the prediction of the ultimate settlement obtained from the Asaoka method was compared with the ultimate settlement prediction from 'Hyperbolic' method.

For Asaoka Plot, the settlement data obtained for the settlement plate (SP18) was extracted for a particular constant time interval value Δt (e.g. 7 days) and the maximum monitored settlement over the field monitoring period was considered as the peak settlement value. By trial and error, consideration of the settlement-time data range after 60% consolidation was found to be appropriate for predicting the ultimate settlement of the NBR embankment using Asaoka method. Similar approaches have been reported by Tan (1996) which was supported by Bergado et al. (1991).

Different values of Δt (= 7, 14 and 21 days) were attempted for predicting ρ_{ult}. It was observed from the application of Asaoka method for this field case that, with increases in the time interval (Δt), the predicted ultimate settlement decreased but, after a certain cutoff time interval (Δt), their magnitudes became identical which is in agreement with the findings of Arulrajah (2005). The regression value for the corresponding Asaoka plot was found to be about 0.99.

For the NBR embankment, the ultimate settlement predicted using the Asaoka and Hyperbolic methods were almost identical (517.00 mm and 517.25 mm respectively). In both cases, data beyond 60% of the consolidation (Tan 1996) were considered, as supported by Bergado et al. (1991). It is therefore concluded that when the soft soil exhibits significant creep, the ultimate settlement prediction by the Asaoka method only provided good agreement with the Hyperbolic method after a certain cutoff time interval (Δt) and the data range after 60 % consolidation state. Therefore, the ultimate settlement prediction by the Asaoka method for creep-susceptible soft estuarine clay requires scrutiny.

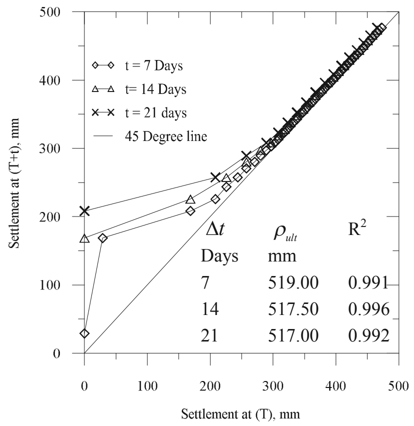
In the Hyperbolic method, the relationship between settlement (ρ_t) at any time (t) is given by Eqn. 3 and the ultimate settlement by Eqn. 4.

$$\frac{t}{\rho_t} = \beta + mt \tag{3}$$

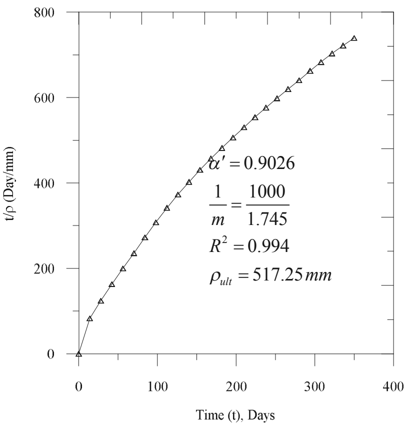
This is a linear straight line in a plot of $\frac{t}{\rho_t}$ against t.

$$\rho_{ult} = \alpha \frac{1}{m} \tag{4}$$

where α, β, m are the co-efficients representing the theoretical slope factor, intercept and slope of the straight line respectively



(a)



(b)

Figure 6. Observational approaches for predicting settlement: (a) Asaoka; and (b) Hyperbolic

In the NBR embankment field study, the ultimate settlement predicted using the Asaoka and Hyperbolic methods were almost identical (517.00 mm and 517.25 mm respectively). On the other hand, it was observed that the measured field settlement at 360 days was about 480 mm, whereas the predicted settlements obtained from the MCC and EVP analyses at 360 days were 425.55 mm and 498.00 mm respectively.

5 CONCLUDING REMARKS

The long-term performance of the instrumented preloaded NBR embankment founded on a soft sensitive estuarine clay was numerically modelled using the MCC and creep-based EVP models. It was observed from the field monitoring data through a settlement plate and piezometer that creep-based settlement was ongoing and, after 360 days, ultimate settlement was not attained. Although the MCC model ignored creep, which resulted in it under-predicting ultimate settlement. Measured settlement was well captured by the creep-based EVP model. On the other hand, the MCC model captured the excess pore water pressure prediction better than the creep-based EVP model, particularly up to 73 days. As there may have been rain after 73 days, which would have raised the ground water level, further study is being undertaken to ascertain the excess pore water pressure. To predict ultimate settlement through an observational approach, two methods were used: the Asaoka and Hyperbolic, with their predictions compared with those from FEA-based MCC and EVP analyses. It was observed that the modified calculation of the Asaoka method predicted almost identical magnitudes of ultimate settlement as the Hyperbolic method and FEAs.

6 ACKNOWLEDGEMENTS

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