Calculated and observed long term performance of Leneghans embankment

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A relatively high road embankment was constructed over the three swamps near Leneghans Drive, which is located approximately 150 km north of Sydney, by the Roads and Traffic Authority (RTA) New SouthWales, Australia in the mid 1990s. Among the three swamps the embankment was built over, the middle one posed the greatest geotechnical challenges and is the subject-matter of this paper. The embankment was well instrumented and was monitored for over nine years. Lo et al. (2008) reported the observed long term behaviour, i.e., the settlement, lateral displacement, geotextile reinforcement strains, excess pore water pressure (pwp) developed with time along with the material properties and construction details. They also presented a one dimensional unitcell FE consolidation analysis for the prediction of settlement and excess pwp in the centre-line zone of the embankment. The foundation soil was modelled using elastoplastic MCC model. Although the unitcell analysis is inherently biased to predict on the conservative side, the predicted settlement in the central region was found to be underestimated even after one year. The final settlement after 9 years was under predicted by about 19% in their analysis. This discrepancy was attributed to the fact that, the time dependent (creep or secondary compression) behaviour of the foundation soft soil was not taken into account.

Detailed long term consolidation tests were subsequently carried out to verify the time dependent nature of the foundation soft soil and was reported by Karim et al. (2010a). The settlement and excess pwp response near the centreline of the embankment was well captured by the EVP analyses presented by them.

To assess the overall predictability of the long term performance of the full embankment, a set of 2D plane strain coupled FE analysis of this embankment were carried out adopting both EVP model and elastoplastic MCC model for the foundation soil. A relatively simpler Kutter and Sathialingam (1992) EVP model was selected for this analysis. The model was selected because of its relatively simpler mathematical formulation and requirement of less number of material parameters and also all the material parameters being conventional ones. The details of the analyses and obtained results in comparison with field performance monitoring data are discussed in this paper.

Brief description of Leneghans embankment

Detailed description of the embankment construction and soil properties and instrumentation can be found in Lo et al. (2008). A brief description is presented here for the sake of completeness.

The cross-section of Leneghans embankment and details of the instrumentation used for performance monitoring are shown in Figure 1. To measure the foundation settlement, a series of hydrostatic profile gauges (HPGs) were installed at two instrumentation lines (namely line 1 and line 2). Several inclinometer casings were installed to monitor the lateral displacement profiles with depth at selected locations (also shown in Figure 1).

Site investigation report indicated that the natural ground level varied from reduced level (RL) +0.5 to +0.9 m and the ground water table fluctuated between RL +0.55 m to +1.17 m with an average value of +0.75 m. The subsoil mainly consisted of very soft to soft alluvial clay of up to 16 m thick with the top (≈ 3 m) layer being a firm crust and was reported to have higher permeability. The foundation soil was classified as high plasticity clay (CH) and its saturated unit weight varied from 14.8–16.2 kN/m^3. The natural water content of the soil was found to be ranging between 77% to 99%. Atterberg limit tests on samples exhibited the liquid limit for the soil to be 82–94% and plastic limit to be ranging between 28–37% with plasticity index of 54–63%. The alluvial clay was underlain by extremely to highly weathered siltstone.

A number of measures were taken to confirm the stability of the embankment. It included the use of prefabricated vertical drains (PVDs) to improve the load bearing capacity of the foundation soft soil by accelerating the consolidation, geogrid reinforcement to stabilize the embankment, the use of light weight fill, wide stabilizing berms, staged constructions, surcharging and observational approach with extensive instrumentations.

PVDs were first installed through the whole soft clay stratum at 1.5 m spacing and in a triangular pattern. A rectangular mandrel system that minimizes the disturbance and smear of the surrounding soft clay was used. A sand blanket was first placed up to R.L. +1.1 m to allow for drainage and to facilitate the placement of geogrid reinforcement and other construction equipments. The force developed in the geogrid reinforcements were monitored with load bolts (LB) and they were put in place at this time. The settlement profiler gauge (HPG) was also placed at this level at each instrumentation section/line.

The embankment was constructed in 3 stages allowing rest period between them to confirm stability. The existing Leneghans drive embankment acted as a toe berm on the eastern side of the embankment. At the end of one year, the embankment reached a height of RL +5.5 m. After one and half year since the construction started, the embankment was surcharged with a meter of soil and after about two years (from the date of start of construction) the surcharge was excavated back to RL +5.5 m.

Numerical details

The material models adopted to represent different part of the finite element mesh and other details are discussed here.

Foundation soil

As discussed before, two 2D plane strain analyses were carried out. One of them used a modified form of Kutter and Sathialingam (1992) model to represent the foundation soil. In the other analysis the foundation soil was modelled using elastoplastic MCC model. The material parameters used for both the coupled analyses are discussed in more detail in a later section and the MCC material parameters along with soil vertical permeability parameters are presented in Table 1.

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