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Research Paper

Numerical prediction and back-calculation of time-dependent behaviour of Ballina test embankment

Hans P. Jostad [a,*], Francesca Palmieri [a], Lars Andresen [a], Noel Boylan [b]

[a] NGI (Norwegian Geotechnical Institute), Sognsveien 72, 0855 Oslo, Norway
[b] NGI (Norwegian Geotechnical Institute), Level 7, 40 St. Georges Tce, Perth, WA 6000, Australia

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ABSTRACT

The paper describes the Class A prediction and Class C back-calculations of the Ballina test embankment using the finite element program Plaxis and the Soft Soil Creep Model (SSCM). The prediction underestimated the measured settlement 3 years after construction by about 20%. This was mainly due to too high stiffness in the transition zone beneath the clay and that SSCM underestimated the shear deformation of the clay. Furthermore, the horizontal permeability of the clay was overestimated. In the back-calculation, it was possible to obtain a excellent match with the measured settlements by reasonable modifications of the input parameters.

1. Introduction

Settlements of foundations and embankments on soft ground in geotechnical engineering are often calculated using idealized 1D methods with simplified assumptions or elastic analytical solutions of load spread distribution with depths, pure vertical pore pressure dissipation, and permeability and compressibility parameters from oedometer tests. Time dependent creep deformations are added by a simple secondary consolidation term, e.g. Mesri [1]. However, in some projects more accurate settlement predictions are required. In these cases, 2D or 3D analyses using a fully coupled displacement and pore water flow (consolidation) finite element (FE) program with a proper material model may be used.

In order to improve the accuracy and reliability of more advanced numerical analyses, the FE calculation models and the process of determining parameters need to be validated against results from well defined and instrumented field cases. This was the purpose of the test embankments constructed at the National Soft Soil Testing Facility (NFTF) in northern New South Wales, Australia.

The Australian Research Council (ARC) Centre of Excellence for Geotechnical Science and Engineering invited practising engineers and academics to make predictions of the time dependent settlement, pore pressure dissipations and lateral displacements of the test embankment.

NGI delivered two different predictions, one based on hand-calculation and on based on advanced numerical analyses using the finite element program Plaxis (www.plaxis.nl). This paper describes the numerical Class A prediction together with a Class C back-calculation.

2. Background information

2.1. Test site and embankment

Two test embankments were constructed at the NFTF. Several sampling, laboratory and in situ testing campaigns have been performed to characterize the soil [2,3]. Based on geophysics, cone penetration (CPTU) and shear vane tests, it has been demonstrated that the stratigraphy is rather uniform across the site.

Seasonal groundwater variations of about ±1 m cause the in situ pore pressure to vary with time. The average ground water level is about 0.5 m below the ground. Data obtained from vibrating wire piezometers (VWP) installed within the Ballina clay below the footprint of the western embankment (i.e. the one with vertical drains) show that the groundwater is almost hydrostatic with depth.

The depositional history suggests that the ground is likely to be geologically normally consolidated as substantial erosion is unlikely to have occurred. However, some overconsolidation through the seasonal changes in groundwater levels and creep have occurred.

* Corresponding author.
E-mail address: hpj@ngi.no (H.P. Jostad).

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The prediction is based on the stratigraphy deduced from CPT soundings and boreholes Inclo1, Mex1 and Inclo2, and aims at reproducing the settlement of the cross section 2 of the western embankment. The soil layering of this cross section comprises of about 1.4 m thick alluvial clayey sandy silt, underlain by a 9.4 m thick estuarine clay layer, a 3.3 m thick transition zone, a 5 m thick sand layer and then a stiff Pleistocene clay layer.

In order to build the embankment, a working platform approximately 95 m long by 25 m wide and 0.6 m thick was initially constructed. On top of this a 0.4 m thick sand layer was placed, before the wick drains were installed. Lastly, a 2 m thick top earth fill comprised of highly plastic clay was constructed on top of the sand layer. The final crest of the embankment was 80 m long by 16 m wide. The slope of the sides was 3:2 (H:V).

3. Finite element analyses

3.1. Finite element model

The numerical analyses are carried out by using the finite element (FE) program Plaxis 2D version 2016.01 (www.plaxis.nl). Fig. 1 shows the finite element model used in the Class A prediction. The model consists of 8 soil layers, the 0.6 m thick working platform, 0.4 m thick sand drain and the 2 m thick top embankment. The model covers a total horizontal distance of 140 m. This model is found to be sufficiently large enough such that end effects do not affect the settlement beneath the embankment and the horizontal displacement at the edge of the embankment. The bottom boundary is taken at the top of the stiff Pleistocene clay. The ground water table is in the Class A prediction taken at 1.2 m below the original terrain in order to fit the effective stress profile given in [2].

The effect of the wick drains is modelled by the vertical drain elements available in Plaxis, starting from the sand layer 1.0 m above the ground continuing down to 14.9 m below the ground, with a selected center distance of 3.2 m. When activated, the drains force the nodes with pore pressure degree of freedom along the geometrical line to have a head equal to a specified value. In the Class A prediction the head is set equal to 0 m, i.e. the nodes are forced to have a hydrostatic pore pressure starting from the original ground level. The corrected horizontal permeabilities used for the soil between the drains are calculated in Section 3.3.

The ground is assumed to be horizontal even though the borings shows some small variations. Displacements along the bottom of the model is fully fixed while the vertical boundaries are free to move in the vertical direction and fixed in the horizontal direction. Pore water flow is prevented through the bottom and the vertical boundaries of the estuarine layer. The other soil layers are considered to be drained and thus pore water flow through their vertical boundaries are allowed.

In the analyses an updated mesh formulation is used. This means that after each calculation step, the nodal points are moved according to the calculated incremental displacements. The main purpose of the updated mesh analyses is to account for that the excess weight of the embankment is gradually reduced as the material settle below the ground water table. This is accounted for by using the "Updated water pressures" option in Plaxis.

The 15-node triangular element and the medium mesh option are selected in the calculation, leading to a total of 2151 elements. This model is found to be fine enough to not be affected by any discretization errors.

3.2. Soil models and properties

The compressibility and shear deformation of the estuarine clay are modelled with the Soft Soil Creep Model (SSCM) [4]. This model accounts for the stress dependent stiffness of the soil within the framework of hardening plasticity. In addition, the model takes into account the time-dependent behaviour of the deformation, i.e. creep. The hardening law of SSCM does not include directly the strain-induced destructuration such as for instance in Creep-SCLAY1S [5,6]. Instead, the parameters are selected in order to model the significant stiffness reduction seen for this clay beyond the yield (pre-consolidation) stress in the stress range of interest. Thus, a strain independent value of the modified compression index λ^* is assumed to be appropriate to describe the material compressibility. The SSCM uses an associated plastic flow rule based on an isotropic CamClay type cap surface as shown in Fig. 2 (left). The hardening law is controlled only by the plastic volumetric strain. This means that the additional shear deformation due to slightly higher shear mobilisation than the K_o^{NC} -state may be different than predicted by the isotropic SSCM model. However, in order to control the shear deformation one need to include one additional parameter that change the shape of the yield surface between the K_o^{NC} -line and the failure line M. In the paper by Sivasithamparam et al. [7], one such model is proposed.

The input parameters to the SSCM that controls the compressibility are the modified compression index, λ^*, the modified swelling index, κ^*, the unloading/reloading Poisson ratio, ν_{ur}, the modified secondary compression index, μ^*, and the vertical effective yield stress, σ_{vc}'. The yield stress is defined by the overconsolidation ratio, $OCR = \sigma_{vc}'/\sigma_{vo}'$, or pre-overburden pressure, $POP = \sigma_{vc}' - \sigma_{vo}'$. In SSCM, it is assumed that all plastic strain is time dependent. This means that the yield stress given by the intersection between the elastic compression line and the elasto-plastic virgin compression line is rate dependent, see Fig. 2 (right). The creep rate along the virgin compression line is $d\varepsilon/dt = \mu^*/t_{eqv}$, where t_{eqv} is given by a vertical strain increment (distance) from a reference line corresponding to $t_o = 24$ h, i.e. $\Delta\varepsilon_{v,creep} = \mu^* \ln(t_{eqv}/t_o)$. Therefore, when interpreting the input parameters to SSCM from a constant rate of strain (CRS) oedometer test, one need to account for the actual strain rate used in the CRS-test. Fig. 3 shows back-calculation of the CRS-test of the specimen from depth 5.49 m in boring Inclo2 using SSCM with $\lambda^* = 0.263$, $\kappa^* = 0.042$, $\mu^*/\lambda^* = 0.03$, $\sigma_{vo}' = 40.5$ kPa and $\sigma_{vc}' = 64$ kPa (at the reference

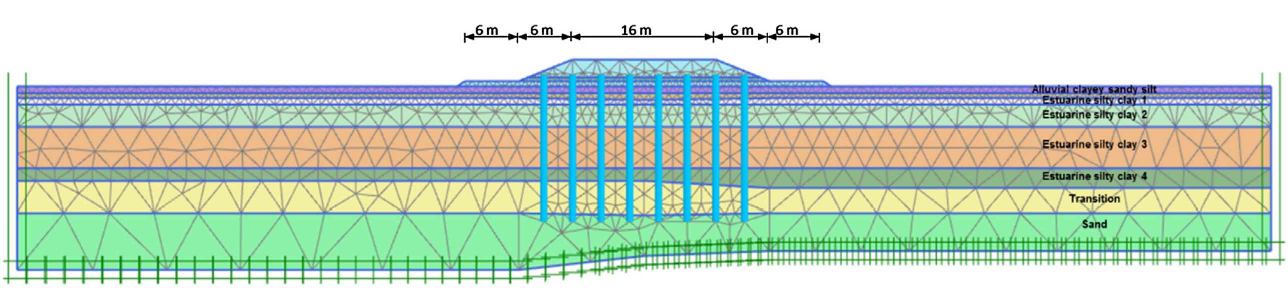

Fig. 1. Finite Element Model of cross section 2 used in the Class A prediction.