PREDICTED AND MEASURED BEHAVIOUR OF AN EMBANKMENT ON PVD-IMPROVED BALLINA CLAY

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Keywords: settlement analysis, embankment, finite-difference method, soft soil, consolidation, hand calculation.
Abstract

This paper presents Class-A and Class-C predictions of the behaviour of an embankment built on soft Ballina clay improved with prefabricated vertical drains. Predictions were carried out using hand calculations and the finite-difference method. The latter approach allowed the variation of soil parameters and stress levels with depth to be considered in the analyses. An alternative systematic procedure for estimating soil parameters based on high-quality laboratory data is described. Class-A predictions highlighted some disagreements with the measured total settlements and pore pressure dissipation rates. For Class-C predictions, the choice of geotechnical parameters used in the analyses was guided by a systematic assessment of the stress states undergone by soil elements underneath the embankment centreline. This led to a better agreement between predicted and measured data, which demonstrates the potential of the proposed procedure for future analyses of embankment behaviour on soft Ballina clay.

1. INTRODUCTION

The prediction of settlements for embankments built on soft ground is a classical problem in soil mechanics. Yet, this remains a challenging task for geotechnical engineers despite significant advances in laboratory and in situ testing, as well as numerical and constitutive modelling. Natural soft soil deposits typically display low undrained shear strength and stiffness, high compressibility, low permeability and weak structure as a result of complex physico-chemical interactions that take place during soil deposition. Poor understanding of the behaviour of natural soft soil deposits (mainly due to poor site characterization) may lead to erroneous selection of soil parameters. A key aspect for the selection of representative soil parameters is to consider the particular stress path imposed by the embankment [1]. The selected method of analysis, and constitutive model, also play significant roles on the ability to predict settlements, lateral displacements and excess pore water pressures [2]. The use of ground improvement techniques, such as prefabricated vertical drains (PVDs), requires additional factors to be accounted for in such analyses.

The Embankment Prediction Symposium (EPS) organized by the ARC Centre of Excellence for Geotechnical Science and Engineering (CGSE) has offered a great opportunity to assess current practice in predicting settlements caused by the construction of an embankment improved with PVDs at the National Soft Soil Testing Facility in Ballina, Australia [3]. Class-A predictions were presented and then compared to the field data during the symposium. Class-C predictions (back analyses) were also requested by CGSE to match the measured behaviour. Class-C predictions aim at improving the accuracy and reliability of the methods used to predict the behaviour of embankments constructed on soft estuarine clays, which are commonly found along the east coast of Australia.

This paper describes Class-A and Class-C predictions of embankment behaviour. Predictions were carried out using one-dimensional calculations along with a semi-empirical approach and the finite-difference method (FDM). One-dimensional methods were used due to their simplicity and common use in current practice. Emphasis was given in this paper on
the method used to interpret in situ and laboratory data for the selection of geotechnical parameters. The selected method was based on considerations of soil stress conditions under the embankment. Predictions of settlements and excess pore water pressures were then compared with field data. Finally, a sensitivity analysis was carried out to assess the uncertainty associated with the selection of soil parameters on the predictions made. Details of the methods and analyses used are described in the next sections.

2 SUBSOIL CONDITIONS AND EMBANKMENT CHARACTERISTICS

Figure 1 summarises the main index and mechanical properties of the soil profile at the Ballina site obtained from in situ and laboratory tests [3, 4]. The soil profile may be divided into five main layers as follows: (a) a shallow upper crust mainly composed of silty sands (z < 2 m), (b) a transition zone composed of silty clays with high shell content (2 < z < 4 m), (c) homogenous soft Ballina clay layer (4 < z < 11 m), (d) a transition sandy layer (11 < z < 14 m), and (e) a Pleistocene stiff clay layer (z < 14 m). Ballina clay displays high plasticity with liquid limit values slightly higher than its natural water content (Figure 1a). Dry density reduces with depth (void ratio increases) and remains almost constant below a depth of 4 m. This is consistent with the increase in the clay fraction which is, mainly composed of illite, kaolinite and interstratified illite/smectite. In the soft Ballina clay layer, the soil is slightly overconsolidated (1 < YSR < 2, Figure 1b). The coefficient of consolidation of the clay layer \( (c_v) \) at yield stress \( (\sigma_{\text{yield}}') \) varies between approximately 1 and 4 m\(^2\)/year (CRS and IL-Creep tests). The horizontal coefficient of consolidation \( (c_h) \) estimated from CPTu and piezoball tests [3] shows a similar range of variation, which suggests small permeability anisotropy in Ballina clay. The undrained shear strength \( (s_u) \) increases with depth. \( s_u \) is greatest in triaxial compression, intermediate in vane shear and least in triaxial extension. The sensitivity estimated from field vane tests varies between 1.5 and 5.5 [3].

The embankment is nominally 80 m long by 15 m wide at the crest, with a total height of 3 m and nominal inclination of 1.5H: 1V. The working platform is approximately 95 m long by 25 m wide, with a height of 0.6 m. The embankment is divided into 3 sections, two 30 m long with PVDs and one 20 m long with Jute PVDs. Vertical drains were installed in the soil underneath the working platform after its completion. The spacing between drains is 1.2 m with a square grid. Vertical drains are 100 mm wide and 3 mm thick. The mandrel has a width of 120 mm by 60 mm in a rectangular shape. Instrumentation of the embankment includes inclinometers (INCLO), settlement plates (SP), magnetic extensometers (MEX), vibrating wire piezometers (VWP) and hydraulic profile gauges (HPG). Figure 2 shows the embankment geometry and the location of the instrumentation. SP3 is located at section 1, MEX1, VWP5, INCLO 1 and 2 are located at section 2, whereas SP2 is located at section 3. SP2 and SP3 are located at the top surface and measure the total settlement of the embankment. MEX1 measures the localised vertical settlements at depths of 2, 5, and 8 m, VWP5 monitors the pore pressure dissipation at depths of 2, 6 and 9.5 m and INCLO 1 and 2 measure the lateral displacement at the toe on each side of the embankment. The unit weight of the fill used to construct the embankment, estimated via nuclear density tests, is 20.9 kN/m\(^3\). The total vertical surcharge applied by the embankment at the ground surface is 63 kPa. As explained in Appendix A, elastic solutions were used to determine the stress
increments with depth. From the surveying data, the topography of the embankment during construction and the position of the instruments were obtained using AutoCad Civil3D® software. Figure 3 shows the schematic vertical cross section of the embankment. Table 1 summarises the geometry of the vertical cross sections used in the predictions for the two stages of embankment construction. The embankment dimensions vary slightly at each location, especially the embankment slope on both sides.

As a minimum, symposium participants were asked to provide at least, the following estimations:

- Time-settlement curves for SP2 and SP3;
- Time-settlement curves at three distinct depths for MEX1;
- Variation of total pore pressures over time for VWP5; and
- Lateral deformation for INCLO 1 and 2 at the end of construction and after three years from the start of construction.
Figure 1: (a) Main index and (b) mechanical properties of the soil profile at the Ballina site.
**Figure 2:** Nominal dimensions of the embankment.

**Figure 3:** Schematic embankment cross section used for vertical stress increment calculations.

**Table 1:** Dimensions of the sections based on the location of instruments.

<table>
<thead>
<tr>
<th>Section</th>
<th>Instruments</th>
<th>Embankment Dimensions (m)</th>
<th>Stage 1</th>
<th>Stage 2</th>
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<td></td>
<td></td>
<td></td>
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<td>b</td>
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<tr>
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<td>2.1</td>
<td>3.6</td>
</tr>
<tr>
<td>3</td>
<td>SP2</td>
<td>32.6</td>
<td>2.9</td>
<td>3.7</td>
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</table>

Note: Stage 1 corresponds to completing the working platform including the sand blanket. Stage 2 refers to completing the embankment with PVD after completion of Stage 1.
3 PREDICTION METHODS USED IN THE ANALYSIS

The upper 10.5 m of soil was considered in the predictions. This includes the shallow sandy layer, the transition silty clay and the soft Ballina clay below 10.5 m. The sandy layer and the stiff Pleistocene clay undergo negligible deformations. For each cross section of the embankment, the 10.5 m deep soil profile was divided into 21 sublayers. The geotechnical parameters for each 0.5 m thick sublayer were estimated from laboratory test results [4, 5]. Two methods of analysis were used to predict vertical settlements, excess pore water pressures and lateral displacements in the presence of PVDs. The first approach combines 1D consolidation theory [6] with the semi-empirical method proposed by Barron and Hansbo [7, 8] (BH method). In the second approach, 1D consolidation theory is used in conjunction with FDM. Although the adopted methods do not explicitly account for particular features of soft clay behaviour such as rate effects, soil anisotropy and soil destructuration, they may provide reliable estimates of the global embankment behaviour. Their simplicity is advantageous as they may be easily implemented in practice without requiring an excessive number of parameters. They can also be used to validate complex numerical simulations. The methods were implemented in a spreadsheet using Microsoft Excel®. In both Class-A and Class-C predictions, the following estimations were made:

1. End of primary consolidation settlement due to the weight of the working platform and the embankment using hand calculations;

2. Time variation of excess pore water pressure, degree of consolidation and consolidation settlement due to the construction of the working platform (without PVD) using the FDM;

3. Time variation of excess pore water pressure, degree of consolidation and consolidation settlement due to the construction of the embankment after installation of PVDs using the BH method;

4. Same as in step 3 but using the FDM;

5. Secondary compression settlement from 1D consolidation theory; and

6. Lateral displacements using empirical correlations from Tavenas et al. [9].

A brief description of each calculation step used in the analysis is given below.

3.1 End of primary consolidation settlement due to working platform and embankment

The end of primary consolidation settlement was determined from 1-D consolidation theory. Ballina clay is slightly over-consolidated with a yield stress ratio typically lower than 2. In cases where the final vertical effective stress was less than the \( \sigma'_{\text{yield}} \), Equation 1 was used. For these cases the final state of the soil is overconsolidated \( (s_{\text{final-OC}}) \). For sublayers loaded to a final vertical effective stress larger than the \( \sigma'_{\text{yield}} \), Equation 2 was employed instead. In this case, the final state of the soil is normally consolidated \( (s_{\text{final-NC}}) \). The total
final settlement at a given depth was estimated as the sum of the settlements in the underlying sublayers.

\[
s_{\text{final-OC}} = \frac{C_r}{1+e_0} h_z \log \left( \frac{\sigma'_{v0} + \Delta \sigma}{\sigma'_{v0}} \right)
\]

\[
s_{\text{final-NC}} = \frac{C_c}{1+e_0} h_z \log \left( \frac{\sigma'_{\text{yield}}}{\sigma_{v0}} \right) + \frac{C_r}{1+e_0} h_z \log \left( \frac{\sigma'_{v0} + \Delta \sigma}{\sigma'_{\text{yield}}} \right)
\]

\( s_{\text{final}} \) = settlement of sub-layer (either OC or NC) (m)
\( e_0 \) = initial void ratio
\( C_r \) = recompression index
\( C_c \) = compression index
\( h_z \) = height of sublayer (m)
\( \sigma'_{v0} \) = initial vertical effective stress (kPa)
\( \sigma'_{\text{yield}} \) = yield stress (kPa)
\( \Delta \sigma \) = stress increment due to embankment weight (kPa)

**3.2 Consolidation settlement due to the construction of the working platform only (without PVDs) using FDM**

Classical 1-D consolidation theory does not account for variations in soil parameters due to the non-homogeneity of the layer (among other factors), especially \( c_v \). The \( c_v \) varies with depth and stress level. Assuming a single value for the entire soil layer may lead to non-reliable predictions of the settlement. Therefore, the FDM was used here to account for variations in soil parameters across the sublayers [10, 11] and estimate the dissipation of excess pore pressure generated by the embankment construction (Equation 3). Due to the division of the soil profile into sublayers with different \( c_v \) values, Equation 3 was modified to account for the difference in soil properties at the interface between two separate layers (see Equation 4[12]). Equation 4 was employed to obtain the excess pore pressure due to the construction of the working platform (without PVDs) where the drainage path is preferentially vertical. The initial excess pore pressure \( (u_0) \) was assumed to be equal to the stress increment in the layer imposed by the embankment (see Appendix A). The boundary conditions used in Equation 4 were full drainage \((u = 0)\) at \( z = 0 \) and \( z = 10.5 \) m.

From the excess pore pressure, the degree of consolidation in the vertical direction was calculated using Equation 5. The settlement of the embankment, \( s_t \), at any time, \( t \), after completing the working platform was calculated via Equation 6 [13, 14].

\[
u(i, j+1)=u(i,j)+ \beta[u(i-1,j)+ u(i+1,j)-2u(i,j)] \text{ where } \beta= \frac{c_v \Delta t}{h_z^2}
\]
\[
\begin{align*}
\frac{u(i,j+1)}{u(i,j)} &= \frac{c_{v,1} \Delta t}{h^2} \left[ 1 + \left( \frac{k_1}{k_2} \right) \left( \frac{c_{v,1}}{c_{v,2}} \right) \right] \times \\
& \left[ \frac{2k_1}{k_1+k_2} u(i-1,j) + \frac{2k_2}{k_1+k_2} u(i+1,j) - 2 u(i,j) \right] + u(i,j)
\end{align*}
\]

\[U = \frac{u_0 - u(i,j)}{u_0} \quad (5)\]

\[s_t = U \times s_{\text{final}} \quad (6)\]

\[s_{\text{final}} = \text{final settlement (Eqs. 1 and 2) (m)}\]

\[s_t = \text{settlement at any time, } t \text{ (m)}\]

\[U = \text{average degree of consolidation in the particular sublayer}\]

\[u_0 = \text{initial excess pore pressure (kPa)}\]

\[u(i,j) = \text{excess pore pressure at node } i \text{ at time } j \text{ (kPa)}\]

\[h_z = \text{height of sublayer (m)}\]

\[c_v = \text{coefficient of consolidation (m}^2/\text{year)}\]

\[c_{v,1} = \text{coefficient of consolidation of top layer at the interface between two layers (m}^2/\text{year)}\]

\[c_{v,2} = \text{coefficient of consolidation of bottom layer at the interface between two layers (m}^2/\text{year)}\]

\[k_1 = \text{hydraulic conductivity of top layer at the interface between two layers (m/s)}\]

\[k_2 = \text{hydraulic conductivity of bottom layer at interface between two layers (m/s)}\]

### 3.3 Consolidation settlement due to the working platform and embankment (with PVDs) using the BH method

In the presence of PVD, water may flow along vertical and horizontal directions. However, for a relatively thick clay layer (> 5 m), it is reasonable to assume that drainage occurs mainly in the horizontal direction [10, 14]. Therefore, the average degree of consolidation is assumed to be similar to the average degree of consolidation in the horizontal direction \(U_r\) (Equations 7-17). The determination of \(U_r\) accounts for the influence of smear (Equation 11) and the influence of the diameter and equivalent diameter of the PVDs (Equations 12-15). Variations in the efficiency of the PVDs with time (e.g. due to clogging) are not considered in this paper. The settlement, \(s_t\), at any time \(t\), is calculated from Equation 6. This set of equations is based on the works of Barron [7] and Hansbo [8]. The approach has been successfully used in the design of embankments on very soft soils [14].

\[U = U_r \quad (7)\]

\[U_r = 1 - e^{-\frac{8T_r}{T}} \quad (8)\]

\[T_r = \frac{c_{v,1}}{d_c} \quad (9)\]
As an alternative to the BH method, FDM was used to estimate the dissipation of excess pore pressure due to the embankment construction, in the presence of PVDs. Only radial drainage was considered, an assumption also adopted in previous studies [10, 14]. In this method, four nodes were allocated from the vertical axis of a reference PVD to the centre line between two adjacent PVDs (Figure 4). The boundary condition at the vertical axis of the PVD corresponded to zero excess pore pressure. At the centre line between two PVDs, the excess pore pressure was maximum and dissipated over time. The pore pressure was assumed
to vary linearly with radial distance. Equation 18 was used to estimate the radial dissipation of excess pore pressure at the centreline between two PVDs. The degree of consolidation and settlement were then obtained from Equations 5 and 6, respectively. Unlike the BH method, no correction for smear effects was made in the FDM. Therefore, a slight overestimation of the settlement may be expected.

Figure 4: Variation of excess pore pressure across different nodes for radial drainage.

\[
u(i, j+1) = u(i, j) + \beta_r \left[ u(i-1, j) + u(i+1, j) + \frac{u(i+1, j) - u(i-1, j)}{2 \left( \frac{\Delta r}{r_i} \right)} - 2u(i, j) \right] \quad \text{where} \quad \beta_r = \frac{c_h \Delta t}{\Delta r^2}
\]

\( u(i, j) \) = excess pore pressure at node i at time j (kPa)
\( c_h \) = coefficient of consolidation in the horizontal direction (m²/year)
\( r_i \) = radial distance from the centreline in the horizontal direction (m²/year)
\( \Delta r \) = distance between two nodes (m)

### 3.5 Secondary compression settlement

The secondary compression settlement was calculated according to the method presented by Mesri and Choi [15]. For Class-A predictions, secondary compression was assumed to occur after 95% of the primary consolidation was achieved in each sublayer. It was also assumed that the secondary compression coefficient \( C_a \) remained constant under constant stress and was not affected by drainage conditions. In reality, secondary compression settlement occurs concurrently with primary consolidation due to the viscosity of the soil structure [14]. This aspect is discussed in the back-analysis carried out in the Class-C predictions. The secondary compression settlement was computed according to Equation 19:
\[ s_{sec} = \frac{h_z}{(1+e_0)} C_{\alpha} \left( \frac{t}{t_p} \right) \]  

\( h_z \) = sub-layer height (m)  
\( s_{sec} \) = secondary compression settlement (m)  
\( t_p \) = time corresponding to 95% degree of consolidation in the sublayer (day)  
\( t \) = time (day)  
\( C_{\alpha} \) = secondary compression index from incremental loading (Creep) test

3.6 Lateral displacement

The lateral displacement at the toe of the embankment was predicted using the empirical expression proposed by Tavenas et al. [9], which was derived from field observations of 21 different embankments. This empirical expression relates the maximum lateral displacement to the settlement via a parameter, \( \alpha \) (Equation 20). The parameter \( \alpha \) takes different values depending on the soil state (OC or NC) [9]. The \( \alpha \) values adopted in the Class-A and Class-C predictions are discussed in Section 4.1 and 5.1, respectively.

\[ y_m = \alpha s \]  

\( y_m \) = maximum lateral displacement (m)  
\( \alpha \) = ratio of maximum lateral displacement to vertical displacement  
\( s \) = total settlement (m)

3.7 Selection of soil parameters

The profiles of mechanical properties shown in Figure 1b include estimates for both initial and yield stress conditions in situ. This information is typically insufficient to properly reproduce the behaviour of geotechnical structures under different loading scenarios. The selection of soil parameters requires proper understanding of the stress paths imposed by the embankment at different locations in the soil profile. The 1D methods used in this paper provide approximate tools to predict the behaviour of the soil underneath the embankment. Strictly speaking, they apply solely to soil states under the embankment centre line, where no lateral deformation occurs.
In general, the settlement caused by the construction of embankments on soft soils is controlled by: (i) the overconsolidation ratio (OCR or YSR), (ii) the coefficients of consolidation \( c_v \) and \( c_s \), (iii) the compressibility index \( (C_c) \), (iv) creep effects (e.g., \( C_r \)) and embankment geometry. While this may often be overlooked, the rigorous selection of soil parameters requires a deep understanding of soil behaviour and proper knowledge of *in situ* and laboratory testing techniques. With naturally structured soft soils, like Ballina clay, the highly non-linear response observed during 1D compression (due to soil destructuration) poses additional challenges to the selection of a (unique) set of soil parameters to be used in design.

To illustrate this aspect, Figure 5 shows the results of two 1D compression tests carried out on Ballina clay specimens obtained from 5.22 m (IL test) and 5.49 m (CRS test) depths. Although both CRS and IL tests may be used to study soil compressibility, the CRS test has the advantage of providing a continuous compressibility curve \( (e^{-\log \sigma'_v}) \). This simplifies the estimation of the \( \sigma'_\text{yield} \) but also provides continuous variation of the \( C_c \) with stress level. Creep effects can be directly assessed from IL tests which also provide good estimates of other keys including \( c_v \) and hydraulic conductivity \( (k_w) \). In Figure 5, the vertical stress increment \( (\Delta \sigma_v) \) imposed by the embankment at a depth of 5.5 m is indicated by the shaded area. As observed in Figure 5a the stress state moves from an overconsolidated (OC) to a normally consolidated (NC) state as a consequence of the stress increment applied by the embankment. Based on \( \sigma'\text{yield} \) values reported in [3], similar situations occur throughout the soil profile. The strong non-linearity of the compressibility curve highlights the challenges associated with any attempt of estimating a unique value for the \( C_c \). Alternatively, one may represent the NC part of the compressibility curve by defining two compressibility indices: (i) \( C_{c-1} \) for stresses in the range \( \sigma'\text{yield} < \sigma'_v < \sigma'_v=100\text{kPa} \) and, (ii) \( C_{c-2} \) for effective vertical stresses larger than 100 kPa. Inspection of Figure 5b indicates that, although \( C_{c-2} \) may represent the \( C_c \) at large stresses with high confidence, the adoption of an intermediate value \( C_{c-1} \) clearly does not capture the strong soil non-linearity. The \( c_v \) as well as the \( C_a \) are strongly affected by the stress increment applied by the embankment (see Figures 5c and 5d). \( c_v \) reduces dramatically after yielding, whereas \( C_a \) reaches a maximum value around \( \sigma'\text{yield} \) and then decreases with increasing stress level.

It is important to note that it will take some time for each soil layer to reach equilibrium in effective stresses, due to the consolidation experienced by the clay. This aspect introduces some degree of judgement in the definition of the stress level used for the selection of soil parameters. In this paper, a simple methodology was used to estimate the behaviour of the soil under the embankment. The criterion used here is based on the definition of a reference vertical effective stress at which soil parameters are assessed thus requiring less judgement, which may be highly subjective. As discussed next, the definition of the reference vertical effective stress is refined in the Class-C predictions (back-analysis) to consider the strong influence of soil destructuration on the soil response, particularly regarding \( c_v \).
Figure 5: Compressibility and consolidation properties of Ballina clay: (a) Compressibility curve. (b) Variation of $C_c$ with stress level. (c) Variation of $c_v$ with stress level and testing method. (d) Variation of $C_\alpha$ with stress level.
4 CLASS-A PREDICTIONS

4.1 Determination of soil parameters for the Class-A predictions

Figure 6 shows a typical compressibility curve (e-log $\sigma'_v$) obtained from CRS testing on Ballina clay. Initial in situ ($\sigma'_v0$), yield and final (i.e., post construction) vertical effective stresses are identified in this figure using arrows. For the Class-A predictions, the reference vertical effective stress used for the estimation of soil parameters is defined as: $\sigma'_{v,\text{ref-A}} = (\sigma'_v0 + \sigma'_v\text{final})/2$ (Figure 6). The values of $\sigma'_{v,\text{ref-A}}$ for each sublayer are listed in Table 2.

Table 2 summarises the soil parameters estimated from the laboratory characterization study reported in [4, 5]. Table 2 includes:

- $e_0$ and $\rho_{\text{bulk}}$: estimated from Tables 1-4 [4].

- OCR: These values were computed as the ratio of $\sigma'_{\text{yield}}$ to the $\sigma'_v0$ reported in Table 2 [4]. Yield stresses were previously corrected by rate effects [4] using a correction factor of 0.84.

- $C_r$: for simplicity, the $C_r$ was assumed equal to the swelling index ($C_r = C_s$). Values of $C_s$ were estimated for each sublayer from IL test results summarised in Table 3 [4].

- $C_c$: values of $C_c$ were estimated from CRS tests. The variation of $C_c$ with the effective vertical stress is highly nonlinear due to progressive soil destructuration (see Figure 9 in [4]). Therefore, $C_c$ was estimated from Figure 9 [4] for the particular $\sigma'_{v,\text{ref-A}}$ of each sublayer.

- $c_v$: although estimations from CRS and IL tests seem to follow the same trend, only IL (Creep) test results had been used in the estimation of $c_v$. CRS tests were not considered here due to the scatter observed, mainly between 2 m and 4.5 m depths, which is attributed to the small excess pore water pressure measured during the tests for stresses lower than $\sigma'_{\text{yield}}$. Therefore, the IL results reported in Figure 13 (and Table 3) [4] were fitted using an exponential function from which values of $c_v$ were estimated at $\sigma'_{v,\text{ref-A}}$ for each sublayer.

- $C_{\alpha}$: These values were determined from Figure 12 [4] for the particular $\sigma'_{v,\text{ref-A}}$ of each sublayer.

- $\alpha$ values of 0.18 (OC state) and 0.91 (NC state) were adopted in these predictions, in agreement with the data set reported by Tavenas et al. [9].
Figure 6: Example of the estimation of $\sigma'_{v,ref-A}$ for the selection of soil parameters (data from INCLO2-CRS-5.49 m [3]).
Table 2: Soil parameters used in Class-A and Class-C predictions

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Bulk unit weight of soil, γ (kN/m³)</th>
<th>OCR/YSR (%)</th>
<th>Gᵥ/sec (kPa)</th>
<th>Initial void ratio, eᵥ</th>
<th>Cᵥ</th>
<th>Cₛ</th>
<th>Cₐ</th>
<th>cᵥ, cₛ (m²/yr⁻¹)</th>
<th>Reference stress, σᵥ/ₐᵥ/ₐᵥ₀ (kPa)</th>
<th>Initial void ratio, eᵥ</th>
<th>Cᵥ</th>
<th>Cₛ</th>
<th>Cₐ</th>
<th>cᵥ, cₛ (m²/yr⁻¹)</th>
<th>Reference stress, σᵥ/ₐᵥ₀/Cₐᵥ₀ (kPa)</th>
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4.2 Comparison of Class-A predictions with measured data

The embankment was completed in two stages. First the working platform with sand blanket was built in one day. Then, after installation of the PVDs, the embankment was also constructed in one day, 20 days after the completion of stage 1. In the figures below, the measured data is labelled as MD. The predicted behaviour for the Class-A predictions are labelled as A-BH (Barron and Hansbo method) and A-FDM (Finite-difference method), depending on the method used.

Figure 7 compares the measured and predicted settlements at the location of settlement plates SP2 and SP3. Some discrepancies are observed between measured and predicted settlements. A faster settlement rate was predicted by the FDM than for the BH method. This faster settlement rate is due to the fact that smear effects due to PVD installation were not considered in the FDM. The FDM overestimated the initial part of the settlement curve (t < 400 days) but underestimated the total settlement after 1090 days by 10%. Two different responses were predicted by the BH method at the SP2 and SP3 locations, reflecting the embankment asymmetry at the end of construction. However, the measured settlements at SP2 and SP3 show almost identical responses. Closer inspection of Figure 7 shows high settlement rate was actually measured between 50 and 100 days, indicating that a 30-day delay took place between the end of the embankment construction (at 20 days) and the settlement mobilization. This delay is not captured in the Class-A predictions. For t > 800 days, the predicted settlement rates were lower than the measured one in all cases.

Comparison between measured and predicted pore water pressures at depths of 2, 6 and 9.5 m is presented in Figure 8. In the Class-A predictions, it was assumed that negligible excess pore pressure was generated during the construction of the first 1 m of fill (sand layer and working platform) due to the slow rate of construction. While this assumption seems to be valid only for shallow (2 m) and deeper (9.5 m) layers due to their proximity to the drainage boundaries, the same assumption was adopted for the entire soil profile. Additionally, the influence of the settlement of VWP5 on the pore pressure response was not considered in the Class-A predictions. The predicted dissipation rates at depths of 2 and 6 m was faster than the field measurements. At a depth of 9.5 m, the predicted dissipation rate matched reasonably well with the measured data. The FDM method predicted faster dissipation of excess pore water pressure than the BH method, due to the fact that smear effects were not considered in the FDM method. The measured response observed at a depth of 6 m indicates much lower dissipation rates in the field and highlights the differences between the in situ and the adopted coefficients of consolidation.

Figure 9 shows the relative settlement curves, measured and predicted, at depths of 2, 5 and 8 m. FDM predictions overestimate the settlement magnitude and rate as previously observed in Figure 7. The BH method shows closer agreement with the field data including the settlement rate at a deeper depth of 8 m.

The predicted and measured lateral displacements at the locations of inclinometers INCLO 1 and INCLO 2 are shown in Figure 10. The empirical method outlined by Tavenas et al. [9]...
overestimates both short and long-term responses, with the maximum lateral displacements being overpredicted by more than 100%. This behaviour suggests that the values of the parameter $\alpha$ recommended by Tavenas et al. [9] are not appropriate to predict the response of the embankment at the Ballina site. Reasons for such a discrepancy may be attributed to the variations in the soil profile, embankment geometry as well as the presence of the PVDs. This aspect is analysed in more detail in the following sections.

There are significant differences in the predictions obtained using the BH and FDM approaches, which may be explained, in part, by the fact that the FDM does not account for smear effects associated with PDV installation. Comparison between predicted and measured behaviour indicates that the adopted coefficients of consolidation were inadequate to properly predict the dissipation of excess pore pressures.

**Figure 7:** Comparison between predicted (Class-A) and measured surface settlements with time.

**Figure 8:** Comparison between predicted (Class-A) and measured total pore pressures over time.
Figure 9: Comparison between predicted (Class-A) and measured relative settlements.

Figure 10: Comparison between predicted (Class-A) and measured lateral displacements at two different times, T1 after completion of embankment and T2 after three years of construction.
5 CLASS-C PREDICTIONS (BACK-ANALYSES)

The comparison between Class-A predictions and measured data highlighted the uncertainties associated with the estimation of settlements and excess pore water pressures in Ballina clay. This is in consistent with previous studies that have highlighted the difficulties associated with predicting embankment behaviour accurately [16-18]. A significant (and surprising) aspect to remark about the measured data is that the excess pore water pressures have not totally dissipated after 1000 days. Obvious differences during the settlement and pore pressure dissipation rates were observed between the predicted and measured data for the last 700 days of the monitoring period. This behaviour suggests that \( c_v \) and \( k_w \) have been strongly reduced due to the stress increment applied by the embankment. Therefore, values of \( c_v \) and \( c_h \) at \textit{in situ} or even \( \sigma'_{\text{yield}} \) levels may not be appropriate to capture the real phenomena happening after completion of the embankment.

The Class-C predictions (back-analyses) described in this paper were aimed at assessing the predicting methods described above to identify key aspects of the soil behaviour that should be carefully considered to generate reliable predictions. An attempt is made here to develop a practical methodology to select soil parameters aimed at reducing subjective inputs. The method of analysis and the adopted soil parameters both contribute to the level of accuracy and reliability associated with settlement and pore-water pressure predictions produced by a given analysis. These two aspects are discussed in detail in the following sections. In the Class-C predictions, the completion of the embankment was changed from day 20\textsuperscript{th} to day 50\textsuperscript{th}, which corresponds to the actual completion day reported by the CGSE. Additionally, the excess pore pressures generated during the construction of the sand blanket and the working platform were included in this analysis.

5.1 Revision of soil parameters for Class-C predictions

In the Class-A predictions, \( \sigma'_{v,\text{ref-A}} \) was defined as the average between the initial \textit{in situ} and final effective stresses was used for the selection of soil parameters. The use of the initial \textit{in situ} vertical effective stress in the estimation of \( \sigma'_{v,\text{ref-A}} \) tends to provide values close to the \( \sigma'_{\text{yield}} \), at which soil destructuration starts. Therefore, \( C_v \) tends to be underestimated, whereas \( c_v \) tends to be overestimated (see Figure 5). A new definition of the reference vertical stress was adopted in the Class-C predictions, based on the following observation. Measured pore water pressures show that around 50 % of the maximum excess pore water pressure dissipated within a few months after embankment completion. Correspondingly, during this timeframe, the effective stresses increased from initial \textit{in situ} to yielding conditions. After that, the stress state of the soil slowly approaching its final state. Therefore, the reference vertical effective stress used in the Class-C predictions is redefined as the average between the yield and final effective vertical stresses: \( \sigma'_{v,\text{ref-C}} = (\sigma'_{v,\text{yield}} + \sigma'_{v,\text{final}})/2 \) (see Figure 6).
Parameters such as $OCR$, $c_v$, $C_c$ and $C_\alpha$ play a key role on settlement and pore water pressure predictions. $OCR$ and $C_c$ control the maximum settlement, whereas $c_v$ and $C_\alpha$ control the dissipation of excess pore water pressure and the settlement rate. After a critical review of the soil parameters used in the Class-A predictions, some modifications were made for the Class-C predictions, as described below and summarised in Table 2.

- $OCR$: no modifications were made to the OCR values adopted in the Class-A predictions.

- $c_v$: the higher pore pressure dissipation rates predicted in the Class-A analysis compared to the measured data suggests that the adopted values of $c_v$ were larger than the in situ ones. A key feature of Ballina clay is the strong reduction in $c_v$ (and $k_w$) after yielding. This reduction is due to soil destructuration and the significant decrease in $e_0$ induced by small increments in vertical effective stress. $c_v$ reduces by about one order of magnitude as the vertical effective stress increases from the $\sigma'_y$ yield to the final stress (Figures 5a and 5c). This suggests that adopting an approach that distinguishes between OC and NC soil states may be useful. Therefore, a two-part $c_v$ profile was selected that combines a section for OC states and another one for NC states (Figure 11). This figure shows the variation of $c_v$ with effective vertical stress for specimens from depths equal to 2.81, 6.43 and 9.81 m, tested in IL tests (grey region), as well as values obtained from CRS tests at $\sigma'_v,0$. The $c_v$ profile for OC states is obtained by averaging the values retrieved from IL and CRS tests and extends up to a vertical stress of 60 kPa, the maximum in situ vertical stress in the profile (continuous line in Figure 11). The $c_v$ profile for NC states is given by the results from IL tests at a depth of 6.43 m and applies to stresses larger than 60 kPa. Based on these profiles, the function “VLOOKUP” in the Excel® spreadsheet was used to select the value of $c_v$ at $\sigma'_v,ref-C$ in each sublayer. Open symbols are used in Figure 11a to represent the values of $c_v$ selected in the Class-C predictions. Figure 11b compares the values of $c_v$ used in both predictions. The $c_v$ profile for NC states reduces by about around one order of magnitude, in agreement with the behaviour shown in Figure 5c. The incorporation of these two $c_v$ profiles into the BH and FDM approaches is described in Section 5.2.
Figure 11: (a) $c_i$ curves from experimental data and (b) comparison of coefficients of consolidation from Class-A and Class-C predictions.

- $C_c$: in the Class-A predictions, values of $C_c$ were estimated from plots of $C_c$ vs. $\sigma'_v/\sigma'_\text{yield}$ at a depth of interest (Figure 9 in [4]). Such a procedure was impractical considering the number of sublayers used in the analysis. Moreover, some scatter was also observed due to the natural variability of the samples. An improved selection procedure was attempted for the Class-C predictions. Figure 12a compares plots of $C_c$ vs. $\sigma'_v/\sigma'_\text{yield}$ obtained from CRS tests (borehole Inclo 2) [4]. The three main trends observed in this figure are consistent with the three layers composing the soil profile. Curves from depths equal to 1.86, 3.14 and 7.75 m were selected to represent the shallow sandy layer, the transition layer and the soft clay layer, respectively. Values of $C_c$ were obtained at the effective reference stress, $\sigma'_{v,\text{ref-C}}$ of each sublayer by using the function “VLOOKUP” in the Excel® spreadsheet. Estimated $C_c$ values are indicated in Figure 12 (a) using solid symbols. Figure 12b shows the comparison between $C_c$ values adopted in Class-A and Class-C predictions. The new $C_c$ profile shows a consistent trend and less scatter compared to the original one.
• $C_\alpha$: the method used to determine $C_\alpha$ was simplified in the Class-C predictions by selecting only four curves from Figure 12 [4] to represent the behaviour of the three main layers that compose the entire profile. These correspond to specimens from depths of 0.91 m ($z < 2$ m), 2.81 m ($2 \text{ m} < z < 4.5$ m) as well as 5.22 m and 6.43 m ($4.5 \text{ m} < z < 10.5$ m). Similar estimates of $C_\alpha$ were obtained with the new selection method (Figure 13a) compared to those used in the Class-A prediction.

• $C_r$: in the Class-A predictions $C_r$ was assumed equal to $C_s$. Such an assumption may be useful in cases where the estimation of $C_r$ is doubtful due to the strong influence of sample disturbance (which is not the case of the specimens tested [4]). However, this assumption may not be realistic in weakly structured soils like Ballina clay, as important soil destructuration takes place if yielding occurs. In the recompression zone, an open fabric dominates the soil response, whereas at large stress levels, the original fabric is destroyed due to soil destructuration and reduction in void ratio. Therefore, differences between $C_r$ and $C_s$ may be significant. Inspection of CRS and IL data [4] shows that $C_r$ is slightly higher than $C_s$ ($\sim 0.17$ compared to $\sim 0.10$). Values of $C_r$ used in the Class-C predictions are shown in Table 2 and Figure 13b. The lowest value in Figure 13b corresponds to the upper sandy layer.

• $e_0$: the void ratio estimated at unstressed conditions ($\sigma_v = 0$) rather than the initial in situ void ratio (i.e., void ratio at $\sigma_v'^0$) was used in the Class-A predictions. Despite its minor influence on the predicted behaviour due to the good quality of the tested specimens, $e_0$ was re-assessed for each sublayer using the ratio $\Delta e/e_0$ (sample quality assessment) summarised in Table 2 [4]. Although small differences are observed in Figure 13c, void ratios at initial in situ stresses were used in the Class-C predictions.
• $\alpha$: in the absence of *in situ* data for Ballina clay, the after construction response was fitted by increasing the value of $\alpha$ from 0.18 to 0.26. On the other hand, good representation of the measured data (within 3 years) was obtained by reducing $\alpha$ from 0.91 to 0.36. Although the back-analysis of lateral displacements was merely based on a fitting process, rather than a rational approach, the adjusted values lie within the range reported by Tavenas et al. [9].

5.2 Review of the methods of analyses used for the Class-C predictions

Two important features revealed by the measured data were the very high settlement rates immediately after the completion of the embankment and the slow dissipation of excess pore water pressures for elements in the centre of the clay layer. Some adjustments to the method of analysis were made in the Class-C predictions aiming at better representing these two characteristics of the soil response. These adjustments can be summarised as:

1. As described above, two $c_v$ profiles were selected to simulate OC and NC soil states. The selection of $c_v$ values for each sublayer was based on: (i) the initial stress in the case of OC states, or (ii) the reference stress in the case of NC states.

2. The calculation of the degree of consolidation (NC states) was modified in the BH method due to the adoption of two $c_v$ profiles. The adjustment was implemented as:

$$U_{S2} = U_{S1} + (1-U_{S1}) \times U_{c_v,NC}; \quad U_{S1} = U_{c_v,OC}$$  \hspace{1cm} (21)
where:

\[ U_{S1} \] = average degree of consolidation up to yield stress;
\[ U_{S2} \] = average degree of consolidation from yield stress to final effective stress;
\[ U_{c_v,OC} \] = average degree of consolidation based on \( c_v \) in OC state; and.
\[ U_{c_v,NC} \] = average degree of consolidation based on \( c_v \) in NC state.

3. Creep settlements were assumed to occur just after yielding. This implies that creep may also occur during primary consolidation, which is consistent with the so-called Hypothesis B described in [19].

4. The ratios \( d_s/d_m \) was decreased from to 2 to 1.75, while \( k_h/k'_{h} \) was increased from 1 to 1.75. These modifications are justified by the data provided by Almeida et al. [14], who reported ranges of variation between 1.5 and 5 and from 1 to 5 for \( d_s/d_m \) and \( k_h/k'_{h} \), respectively.

5. Smear effects were considered in the FDM. Thus, \( c_h \) was factored by 0.57 to be consistent with the ratio \( k_h/k'_{h} \). It was assumed that the permeability of the smear zone controls the dissipation of excess pore pressure in the horizontal direction.

6. The effect of settlement, which results in an increased distance between VWP5 and the static water table, was included in the predictions of the pore pressures.

5.3 Revised procedure for Class-C predictions

The modifications made in the Class-C predictions were implemented following a step-by-step approach that used Class-A predictions as a baseline. The approach is summarised in Table 3. This sequential approach was useful to evaluate the key factors affecting the settlement and pore pressure responses in Ballina clay. Figure 14 summarises the evolution of the settlement and pore pressure predictions (P1 to P9) according to the procedure described above. For simplicity, only predictions from the BH method are presented in Figure 14. Measured data and Class-A predictions are included in Figure 14 for comparison. The shift in the starting date from the 20th day to the 50th day is shown in predictions P1 (Figure 14a). An important improvement in the predictions of settlement and pore pressure is obtained in P2 by adopting the two \( C_v \) profiles described above (Figure 14b). This demonstrates the key role played by \( c_v \) on the rate of settlement. The total pore pressure response at a depth of 6 m is remarkably well predicted in P2. Refinements made to the selection of \( C_v \) result in a better predictions of the settlement response in P3 (Figure 14c). The pore pressure response remains unchanged when using the new \( C_v \) profile. Assuming that creep starts right after the \( \sigma_{yield} \) is exceeded does not affect the pore pressure response, but leads to a slight overestimation of the total settlement (prediction P4, Figure 14d). As the pore pressure response is also controlled by the smear zone created by the mandrel, two minor modifications were also made to the method of analysis as described above. The adoption of some degree of permeability anisotropy (P5) shown in Figure 14e led to even better predictions overall, in particular of the
total pore pressure response. In prediction P6, the modification in $d_s/d_m$ results in a slightly reduced total settlement (with respect to P4) and better predictions of the total pore pressure (Figure 14f). The last three modifications made in P7 ($C_u$, Figure 14g), P8 ($C_r$, Figure 14h) and P9 ($e_0$, Figure 14i) result in minor refinements to the predictions of total settlements and total pore pressures.

Table 3: Approach used and changes made for Class-C predictions

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<td>14(d)</td>
<td>P4-Cα-time</td>
<td>Creep starts immediately after yield stress is exceeded</td>
</tr>
<tr>
<td>14(e)</td>
<td>P5-k_h/k'_h</td>
<td>Ratio of permeability in horizontal direction to permeability of smear zone in horizontal direction was changed from 1 to 1.75</td>
</tr>
<tr>
<td>14(f)</td>
<td>P6-d_s/d_m</td>
<td>Ratio of smear zone diameter to equivalent mandrel diameter was changed from 2 to 1.75</td>
</tr>
<tr>
<td>14(g)</td>
<td>P7-C_u</td>
<td>Secondary compression index values refined</td>
</tr>
<tr>
<td>14(h)</td>
<td>P8-C_r</td>
<td>Recompression index values refined</td>
</tr>
<tr>
<td>14(i)</td>
<td>P9-e_0</td>
<td>Initial void ratio values refined</td>
</tr>
</tbody>
</table>
5.4. Comparison of Class-C predictions with measured data

Figure 15 compares the measured and predicted surface settlements at two different sections. It can be seen that the use of revised parameters as well as the additional modifications implemented into the calculation procedure led to an excellent agreement between predictions and the field data. Similar results were obtained regardless of the method of analysis employed (BH and FDM). Figures 15 and 16 suggest that the predicted
settlements, either on the surface or at depth, tend to be underestimated between 280 to 500 days. The maximum difference, however, is less than 10 % and, at later times, the Class-C predictions and measured data tend to converge.

Figure 17 shows that the predicted initial pore water pressure overestimates the values measured at depths of 2 and 6 m. This is due to the fact that the construction of the 1-m thick working platform and 2-m thick embankment was assumed to have taken place in one single day, which does not represent the real construction history. This assumption implies undrained loading. Therefore, the maximum initial excess pore water pressure in the predictions is equal to the stress increment imposed by the embankment at a depth of interest. On the other hand, the small excess pore water pressure measured after embankment completion (around 50 and 60 % of the theoretical vertical stress increment) agrees with the behaviour discussed in Leroueil et al. [16]. They reported minor excess pore pressure development before yielding. This pore pressure response is captured to some degree in the predictions by the fast pore pressure dissipation occurring after peak, which is controlled by the $c_v$ in OC state. After yielding, the value of $c_v$ is reduced, leading to a reduced rate of dissipation. This response is controlled by the $c_v$ profile in NC states. Overall, the use of two profiles for representing the pore water pressure response seems to capture the measured behaviour in a consistent way.

![Figure 15: Comparison between predicted (Class-C) and measured surface settlements for SP2 and SP3.](image)

Figure 18 shows that the predicted lateral displacements agree reasonably well with the measured data. However, the predictions cannot account for different lateral displacements on each side of the embankment.
**Figure 16:** Comparison between predicted (Class-C) and measured relative settlements at three different depths.

**Figure 17:** Comparison between predicted (Class-C) and measured pore pressure responses over time at three different depths.
6. SENSITIVITY ANALYSIS

A sensitivity analysis was performed to evaluate the influence of different soil parameters on the predicted behaviour. To some extent, this provides insight on the uncertainty associated with the selection of soil parameters either due to poor interpretation of test results or inappropriate assessment of the stress path experienced by the soil profile at the centerline due to the embankment construction. Several combinations of parameters may lead to similar settlement predictions. However, parameter selection should have a rational justification based on available data. Eight different scenarios were evaluated in this analysis. These are summarised in Table 5. To keep the analysis as simple as possible, the remaining parameters/methods were kept identical to those used in the Class-C predictions (Table 2). Only the BH method was used for the sensitivity analysis described here.

Surface settlements from the analyses were compared with the measured data at four different timeframes: 122<sup>nd</sup> day, 248<sup>th</sup> day, 501<sup>st</sup> day and 1090<sup>th</sup> day. Predicted and measured pore water pressures (6 and 9.5 m depths) were compared at 248<sup>th</sup> day, 501<sup>st</sup> day and 1090<sup>th</sup> day. Figure 19a compares the predicted surface settlements for the eight scenarios with the measured data. Class-C predictions were also included for comparison. The inspection of Figure 19b shows that total settlements can be over-predicted by more than 30 % and under-predicted by about 40 %. An upper bound is given by T4-<i>c_v</i> where <i>c_v</i> was adopted from a CRS test at a depth of 3.14 m. The higher <i>c_v</i> used in this case resulted in faster dissipation of excess pore pressure. This suggests that creep settlements started earlier as soil consolidated at a faster rate. When the upper bound parameters for <i>C_α</i> (T8-C<sub>α</sub>) are used, total settlements calculated were increased by 10 % as to the measured data. There is a tendency for underestimating the total settlements as it was also observed in Class-A predictions presented in the EPS.
The lowest predictions for the total settlements arose from T2\(-\)Cc where the lower bound profile of \(C_c\) (CRS at depth 3.14 m) was used. Values of \(C_c\) were approximately 60 % lower than the values adopted in the Class-C predictions. The upper bound values for \(C_c\) (T3\(-\)Cc), which tend to be similar to the adopted \(C_c\) profile, resulted in 10 % differences compared to the measured data. The use of uncorrected \(\sigma'_{\text{yield}}\) values reduced the predicted total settlements by 30%. It is important to note that \(\sigma'_{\text{yield}}\) is also used in the selection of \(C_c\) and \(c_v\). Therefore, the error involved may be much higher.

<table>
<thead>
<tr>
<th>Cases</th>
<th>Changes</th>
<th>Justification</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1-OCR</td>
<td>OCR uncorrected by rate effects</td>
<td>(\sigma'_{\text{yield}}) reported in [2] is uncorrected by strain rate effects</td>
</tr>
<tr>
<td>T2-C(_c)</td>
<td>(C_c) from CRS data at depth 3.14 m</td>
<td>(C_c) from this test data represents lower bound solutions</td>
</tr>
<tr>
<td>T3-C(_c)</td>
<td>(C_c) from CRS data at depth 5.89 m</td>
<td>(C_c) from this test data represents upper bound solutions due to the large variation of (C_c) with increasing stress level</td>
</tr>
<tr>
<td>T4-c(_v)</td>
<td>(c_v) from CRS data at depth 3.14 m for stage 2 only</td>
<td>(c_v) from this test data represents upper bound solutions</td>
</tr>
<tr>
<td>T5-c(_v)</td>
<td>Average (c_v) values used</td>
<td>Three average values of (c_v) were used for three different layers in each stage, representing typical approach taken by practitioners</td>
</tr>
<tr>
<td>T6-t(_{90})</td>
<td>Secondary compression starting after 90 % consolidation</td>
<td>A common approach adopted in practice</td>
</tr>
<tr>
<td>T7-C.</td>
<td>(C_a = 0.002) assumed for all layers</td>
<td>This average value of (C_a) represents lower bound solutions</td>
</tr>
<tr>
<td>T8-C.</td>
<td>(C_a = 0.06) assumed for all layers</td>
<td>This average value of (C_a) represents upper bound solutions</td>
</tr>
</tbody>
</table>

![Figure 19](image-url)  
*Figure 19*: Comparison of total settlements predicted for different cases with measured data and Class-C predictions: (a) Evolution of settlements with time (b) Percentage variations of the settlements for each case from the measured data.
Figure 20 shows the outcomes of pore pressures evaluated at 6 and 9.5 m depths. There is a tendency for underestimating the total pore pressure response. As expected, only $c_v$ has a major influence on the predicted pore water pressure. Case T4-$c_v$ resulted in lower predictions as a consequence of the higher rate of excess pore pressure dissipation. Overall, these analyses suggest that the uncertainties for predicting total settlements can vary within $\pm 35\%$ while for total pore pressure response, it can vary within $\pm 10\%$.

Figure 20: Comparison of total pore pressures predicted for different cases for Class-C predictions with measured data.

7. CONCLUDING REMARKS

Simple hand calculations coupled with the finite-difference method were used to predict the behaviour of an embankment built on PVD-improved Ballina clay. Four soil parameters were identified to control the settlements and pore pressure responses: OCR, $c_v$, $C_c$ and $C_\alpha$. Comparisons of Class-A predictions with the measured data showed important discrepancies, mainly in terms of settlement rate and pore pressure dissipation rate. These two aspects were strongly affected by the values of $c_v$ selected from laboratory tests, which seemed to be too high compared with the in situ value of $c_v$ after construction. The selection of $c_v$ was also affected by the choice of OCR, as OCR influences the value of $\sigma_{yield}^\prime$ used in the analyses. $C_c$ and $C_\alpha$ altered the settlement magnitude but did not affect the pore pressure response. The time assumed for creep to start, which affected the analyses involving the $C_\alpha$ parameter, influenced the settlement rate.

Class-C predictions were made with prior knowledge of the performance of the embankment. Soil parameters were revised, in particular $C_r$, $C_\alpha$, $C_c$ and $c_v$. Class-C predictions showed good agreement with the measured data, indicating the importance of considering realistic parameters for the calculations. It was demonstrated that good predictions were obtained by using two separate $c_v$ profiles (OC and NC states). The sensitivity analysis, which was carried out by varying key soil parameters, yielded a range of responses within $\pm 40\%$ for the total surface settlement and $\pm 10\%$ for the total pore pressures, with respect to the measured values.

This paper demonstrates that simple techniques such as hand calculation and finite-
difference methods are still relevant in current practice to predict settlements of embankments. The paper described a methodology that can be used to carefully select soil parameters from laboratory and in situ data. The use of this methodology led to predictions of settlements and total pore pressure response that were in good agreement with measured data.

ACKNOWLEDGEMENTS

The work presented here forms part of the activities of the Centre for Offshore Foundation Systems (COFS), currently supported as a node of the Australian Research Council Centre of Excellence for Geotechnical Science and Engineering (grant CE110001009). The authors would like to thank Professor Barry M. Lehane for some valuable discussions on industry practice.
APPENDIX A. Stress increments imposed under the embankment

Traditionally, vertical stress increments have been estimated using elastic solutions which vary depending on particular loading types and boundary conditions. The soil is assumed to be a semi-infinite elastic, homogeneous, isotropic and weightless material. For the stress calculation, in this case, simplified solutions [20] based on Boussinesq’s theory have been used. The load from the toe of the embankment to the crest level of the embankment for both sides is assumed to vary linearly, while the load at the crest level is assumed to be uniform. Thus, the stresses from the embankment are assumed to be the combination of uniformly and linearly loaded infinite strips.

The geometry of the embankment used for the estimation of the vertical stresses with depth is given in Figure A.1. The increment of vertical stress at depth \( z \) was determined by multiplying the calculated influence factors by the loading from the embankment as shown in Equation A1[20]. Two sides from left and right were determined separately due to the asymmetrical geometry of the embankment for the two different construction stages.

![Figure A.1: Vertical stress increment under an embankment.](image)

\[
\Delta \sigma_z = q I_{q,1} + q I_{q,2}
\]

\[
I_{q,1} = \frac{1}{\pi} \left[ \frac{p + r}{p} \tan^{-1} \left( \frac{p}{1 + r^2 + 2pr} \right) \right] ;
I_{q,2} = \frac{1}{\pi} \left[ \frac{x + y}{x} \tan^{-1} \left( \frac{x}{1 + y^2 + xy + 2xy} \right) \right]
\]

\( \Delta \sigma_z \) = vertical stress increment at depth \( z \)  
\( q = \gamma h_e \) (kPa)  
\( \gamma \) = bulk unit weight of the fill (kN/m³)  
\( h_e \) = height of the embankment (m)  
\( I_{q,1} \) = influence factor based on the depth and geometry of the embankment from left side  
\( I_{q,2} \) = influence factor based on the depth and geometry of the embankment from right side  
\( p = \frac{a_1}{z} ; r = \frac{b_1}{z} ; x = \frac{a_2}{z} \) and \( y = \frac{b_2}{z} \).
Figure A2 shows the distribution of total vertical stresses at three depths underneath the embankment: 0.5 m, 5 m and 10 m. According to this approach, the total vertical stress increment at the centreline of the embankment reduces from 63 kPa at the ground surface to only 53 kPa at 10 m depth ($\Delta \sigma_{ave} \approx 58$ kPa). This suggests that all soil layers considered in the problem move from a slightly over-consolidated to a normally consolidated state due to the loads imposed by the embankment.

**Figure A2:** Vertical stress distribution underneath the embankment at three different depths.
REFERENCES