Class A and C predictions for Ballina trial embankment with vertical drains using standard test data from industry and large diameter test specimens

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Abstract
Consolidation using prefabricated vertical drains has been one of the most popular ground improvement methods in the past few decades. In this paper, a benchmarking exercise is performed to compare the accuracy of the available methods through both Class A and Class C predictions for a trial embankment stabilised by prefabricated vertical drains (PVD) at the site of the National Field Testing Facility (NFTF). In this paper, the main difference between Class A and Class C is that additional soil properties specially required to represent the visco-plastic behaviour or creep have been included, which were absent in the authors’ original Class A predictions made before the availability of field data. Naturally, the inclusion of visco-plastic behaviour improves the settlement and excess pore water pressure predictions significantly, especially after about 1-1.5 years. The site is located in Ballina, NSW and is owned by the Roads and Maritime Services (RMS) of NSW. A trial embankment was constructed over this soft Holocene clay under the auspices of the ARC Centre of Excellence in Geotechnical Science and Engineering (ARC-CGSE). Prefabricated vertical drains were installed with an array of field instrumentation including inclinometers, piezometers, settlement plates and total pressure cells. Large scale consolidometer testing was also carried out on undisturbed soil specimens (350 mm in diameter) retrieved from the site to obtain additional consolidation parameters needed for radial consolidation analysis. Class A and Class C predictions were performed via numerical and analytical approaches. The results show, that for this case history, the suggested approaches can match the observed field performance well, with the exception of long term excess pore water pressures (say beyond 1.5 years) and the lateral displacement approaching the ground surface.

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CLASS A AND C PREDICTIONS FOR BALLINA TRIAL EMBANKMENT WITH VERTICAL DRAINS USING STANDARD TEST DATA FROM INDUSTRY AND LARGE DIAMETER TEST SPECIMENS

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EMBANKMENT WITH VERTICAL DRAINS USING STANDARD TEST
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Key words: embankment, prefabricated vertical drain, radial consolidation, soft soil.

ABSTRACT: Consolidation using prefabricated vertical drains has been one of the most
popular ground improvement methods in the past few decades. In this paper, a benchmarking
exercise is performed to compare the accuracy of the available methods through both Class A
and Class C predictions for a trial embankment stabilised by prefabricated vertical drains
(PVD) at the site of the National Field Testing Facility (NFTF). In this paper, the main
difference between Class A and Class C is that additional soil properties specially required to
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inclusion of visco-plastic behaviour improves the settlement and excess pore water pressure predictions significantly, especially after about 1-1.5 years. The site is located in Ballina, NSW and is owned by the Roads and Maritime Services (RMS) of NSW. A trial embankment was constructed over this soft Holocene clay under the auspices of the ARC Centre of Excellence in Geotechnical Science and Engineering (ARC-CGSE). Prefabricated vertical drains were installed with an array of field instrumentation including inclinometers, piezometers, settlement plates and total pressure cells. Large scale consolidometer testing was also carried out on undisturbed soil specimens (350 mm in diameter) retrieved from the site to obtain additional consolidation parameters needed for radial consolidation analysis. Class A and Class C predictions were performed via numerical and analytical approaches. The results show, that for this case history, the suggested approaches can match the observed field performance well, with the exception of long term excess pore water pressures (say beyond 1.5 years) and the lateral displacement approaching the ground surface.

1 INTRODUCTION

Accelerated consolidation facilitated by prefabricated vertical drains (PVDs) and surcharge has been considered as one of the popular and economical ground improvement methods. The method is employed to promote radial consolidation by reducing the drainage length. The shear strength of soil is increased with reduced post-construction deformation once the soil is consolidated. Numerous analytical solutions based on the unit cell approach have been initially proposed to consider the various aspects affecting the soil consolidation behaviour. Barron [1] derived classical axisymmetric solutions for radial consolidation considering a constant permeability in the disturbed region of the soil adjoining the vertical drain (smear
zone). This solution is based on the following assumptions: (a) the soil is fully saturated, (b) only vertical compressive strain within the soil occurs uniformly, (c) the outer boundary of the unit cell is assumed to be circular, (d) permeability of the drain is significantly higher than that of the soil, (e) linear Darcy’s law is valid, and (f) the small strain theory is adequate. Subsequently, various other solutions incorporating different assumptions and boundary conditions were proposed by Yoshikuni and Nakanodo [2], Holtz et al. [3], Hansbo [4], Zeng and Xie [5], Chai et al. [6], Zhu and Yin [7], Leo [8], Indraratna et al. [9], Walker and Indraratna [10], Lu et al [11], Kianfar et al. [12], and Lei et al. [13] among others. To analyse an embankment with an array of vertical drains, several researchers have conducted both 2D and 3D finite element modelling (e.g. [14, 9, 15, 16]). They show that both techniques can provide reasonable predictions depending on the embankment geometry, although 3D modelling is significantly more time consuming compared to 2D plane strain. In both cases, a fine mesh discretisation is usually required to generate individual vertical drain and adjacent smear zones as integrated unit cell domains to represent a complete multi-drain analysis.

During the installation of PVDs, soil properties including compressibility and permeability are altered in the smear zone. It was pointed out by Onoue et al. [17], Sharma and Xiao [18], and Sathananthan and Indraratna [19] that after PVD installation, the variations of moisture content and permeability can be employed to characterize the smear zone. For the first time, Indraratna and Redana [20, 21] characterised the smear zone based on the lateral to vertical permeability ratio of reconstituted specimens as measured in a large (500mm diameter) radial drainage chamber. They demonstrated analytically and experimentally that the lateral permeability is significantly reduced as the stress in the vertical and lateral direction is
increased by mandrel driving and this affects the rate of dissipation of excess pore water pressure within the smear zone. This was later confirmed using an independent elliptical cavity expansion theory [22]. Subsequently, an elegant study by Zhou and Chai [23] proposed the concept of an equivalent smear due to non-uniform consolidation.

Indraratna et al. [24] obtained soil samples from various locations around a PVD installed at the Ballina site. Vertical drains were installed using a mandrel (140mm x 90mm) using a static approach. Based on a series of oedometer tests, they found that the soil compression curves are significantly affected due to soil destructuration, apart from the change in the permeability and water content of the soil. The smear zone diameter was about 0.8m and the permeability decreased linearly within the smear zone. The average ratio of permeability in the undisturbed zone to that in the smear zone (k_u/k_s) and the extent of the normalised smear zone (d_s/d_w) for this particular study were 2.3 and 7.8, respectively.

2 PREDICTION CLASSIFICATION

Lambe [25] classified the prediction and benchmarking exercises into three distinct categories depending on the nature and timing of the predictions.

Class A: The predictions are carried out before the construction event with the available soil properties and site investigation data,

Class B: Predictions are made during the construction event, so that they can be influenced or adjusted based on the initial field data, and

Class C: The analyses are carried on after the construction event when the complete set of field data is made available; they can also be used as a back-calculation to determine the appropriate soil properties by curve matching. In this paper, the Authors have conducted both
Class A and Class C consolidation analyses incorporating the soil properties obtained from: (a) laboratory testing using large diameter samples [26], (b) previous Pacific Highway embankment works in Ballina where the field data have been used to interpret soft soil embankment behaviour [27, 28, 29], and (c) field data obtained for characterising the smear zone [24]. The 1st Author conducted these computations even before the embankment was built. Using soil parameters from published industry literature for nearby the Ballina sites does not make these predictions deviate from being Class A albeit accuracy may be affected. It is noteworthy that the original laboratory and site investigation data by industry standards alone cannot be used to obtain the most accurate predictions, because in the field, the mandrel-driven PVDs cause considerable smear and soil destructuration which require further investigation to characterise the alterations to the original soil properties especially the permeability and compressibility parameters. In this paper both a unit cell analysis containing a cylindrical soil with a single vertical drain and a multi drain analysis were considered as described in the following. The Authors did not use laboratory and in situ test data provided by the organiser, because, the organisers requested the 1st Author to use the soil parameters available from industry reports and published literature to his best ability, in order to represent different design perspective. These parameters obtained for nearby Pacific Highway sites are expected to have certain anomalies at various depths that may be in some conflict with the subsequent parameters obtained by Pineda et al [30] through more carefully controlled conditions specifically for this trial embankment site.

3 SOIL PROFILES

A Geological survey of New South Wales (1:250,000 Moreton Map) indicates that the Ballina flood plain contains Holocene sediments of low strength and high compressible silty clay with
shell and sand partings, typical of estuarine deposits. The soil profile was established based on: (a) available literature [27, 28, 29], (b) large-scale 350mm diameter specimen testing [26], and (c) field approach to characterise the smear zone [24]. Figure 1 summarizes the consolidation and basic soil properties, as provided by Indraratna et al. [29].

As described by Indraratna et al. [29], the sub-soil consists of an approximately 2-2.7 m of thick alluvium soft soil followed by 12 m thick very soft clay, which is underlain by a firm silty clay layer of 15 m in thickness. In general, this deposit can be classified as highly compressible marine clay with very low permeability and high plasticity (CH). The natural water content is very close to the liquid limit at most depths of the upper Holocene layer. The groundwater level is assumed to be almost at the ground surface. Given that this area is a low-lying floodplain region, and often the groundwater table is elevated by seepage into the floodplain following heavy precipitation, flash flooding and routine tidal influx. While the water table has been found to fluctuate 0.5m-1m below the ground surface, much of the soil layer above water table is still made saturated due to the discharged water from PVDs of 500m$^3$ during consolidation settlement and the effect of capillary rise, which could be estimated to exceed 800 mm based on the soil properties used for the uppermost Holocene layer using a formulation provided by Peck et al. [31].

4 DESCRIPTIONS OF THE ANALYTICAL TOOLS AND PREDICTION APPROACHES

4.1 Integrated unit cell analytical approach

The following 4 approaches were adopted in the predictions made by the Authors. Cases A-C are considered to be Class A as these predictions were made before the field data was made
available, and the 1st Author was invited to provide Class A predictions based on soil properties available through past industry projects at the Embankment Prediction Symposium held in Newcastle in September 2016. For Case D, the effect of the elasto-visco plastic soil properties was captured to predict the long-term deformation. All relevant mathematical formulations for each method are given in Appendix B, and only the fundamental concepts are explained below. A single set of soil parameters was adopted for analysis in which superscripts are used to indicate the parameters that have been used for each individual case of modelling.

Indraratna et al. [9] demonstrated how an analytical solution could be used to determine settlement for multilayer soil by integrating the single layer theory with varying soil strata with depth. Assuming that the drain is relatively long compared to its spacing, the flow in the radial direction is the most dominant and the vertical flow between the layers has much less influence. Therefore, the settlement of an individual soil layer using a single layer theory can be applied for each individual soil stratum and subsequently integrated with depth to obtain the total settlement with insignificant error.

**Case A (CLASS A prediction):** This method is based on the radial consolidation properties obtained from large diameter samples of undisturbed Ballina soil tested at the University of Wollongong using a large-scale consolidation chamber (350mmx 700mm) under an applied surcharge of 80kPa. For these large cylindrical specimens, the mean lateral soil permeability \( (k_h) \) in the smear zone was determined to be half of that in the outer undisturbed zone.

Both the prefabricated vertical drain and the mandrel were scaled down for the appropriate unit cell simulated herein in relation to the field drain spacing of 1.2m. The scaled-down drain (25 mm in width) was pushed through the centre of the soil sample with the aid of the
steel mandrel. Two pore water pressure transducers (one at the centre and another at a
distance of 96mm from the centre) were installed to capture the change in pore water pressure
and to measure the coefficient of horizontal consolidation ($c_h$).

Based on the pore water pressure readings, a plot of the flow velocity ($v$) against the
hydraulic gradient ($i$) indicates a deviation from the traditional linear Darcy’s law, where the
observed flow is non-linear as shown in Fig. 2. Based on this non-Darcian flow behaviour
[12], the two specific power law constants $\alpha_c$ and $\beta$ were evaluated as $5.28 \times 10^{-10}$ m/s and
1.28, respectively. The extent of the smear zone was found to be about 7.5 times the
equivalent diameter of PVD, which is significantly greater than what has been found from
past laboratory studies for fully remoulded kaolinite clays. The corresponding soil parameters
are shown in Table 1. This Class A prediction has been slightly modified using the exact
history of staged construction (for initial 60 days) and reported as Class C prediction.

**Case B** (*CLASS A prediction*): This model [32] captures the effect of soil disturbance caused
by mandrel driving with variations of permeability and compressibility within the smear zone,
thereby improving the original methods proposed by Indraratna and Redana ([20]; [33]). The
available soil properties were based on previous Pacific Highway embankment works in
Ballina (e.g., [27, 28, 29]). The corresponding soil properties are tabulated in Table 1.

**Case C** (*CLASS A prediction*): In this case, a large strain consolidation model recently
introduced by Indraratna *et al.*, [34] was used. This model also captures non-Darcian flow
with varying compressibility and permeability. The relevant soil parameters sourced from
past Pacific Highway projects in Ballina [27] are listed in Table 1, as amended where
warranted to represent non-linear flow behaviour.

**Case D** (*CLASS C prediction*): To include long-term deformation (creep) in the analysis, an
elastic visco-plastic (EVP) model proposed by Yin and Graham [35] is incorporated in the consolidation equation to compute the settlements and excess pore water pressures using a one-dimensional finite difference method, the details of which have been described in Appendix B. The corresponding soil parameters are shown in Table 2. The creep parameters were determined using UOW’s large-scale (350mm diameter) specimens via long term consolidation tests with a central PVD. It was felt that given the random heterogeneous features typical of estuarine floodplains including miniature sand partings and marine relics etc., larger test specimens would be more representative of the actual soil. For typical inorganic clays, the creep parameter \((c_\alpha \text{ or } \mu^*)\) is found to be in the range of 1-5% of the consolidation coefficient \((c_c \text{ or } \lambda^*)\) based on extensive research conducted over many years [36]. For the 350mm diameter Ballina test specimens, this creep compression ratio is 1.8-2.8 %, hence well within this range given by Terzaghi et al. [36] for inorganic estuarine clays.

4.2 Lateral deformation predictions based on empirical formulations integrated unit cell analytical approach

The prediction of lateral displacements is based on the formulations proposed in past literature using the ratio between the lateral displacement and the settlement at the corresponding depths. Based on a 4.75m high Muar clay embankment in Malaysia with 1.3m vertical drain spacing in a triangular pattern, Indraratna et al. [37] reported three different stability factors \((\alpha, \beta_1 \text{ & } \beta_2)\), and recommended their values as 0.123, 0.034 and 0.274 respectively, where,

\[
\alpha = \text{ratio of maximum lateral displacement at the toe to the maximum settlement at the centreline (}\beta_1/\beta_2); \\
\beta_1 = \text{ratio of maximum lateral displacement to the corresponding fill height;}
\]
\( \beta_2 \) = ratio of maximum settlement to the corresponding fill height.

Indraratna et al. [38] reported the ratio of lateral deformation/maximum settlement (\( \alpha \)) to be about 0.2 for the Port of Brisbane (Australia), while Chai et al. [39] proposed the term NLD (normalized lateral displacement) and RLS (ratio of index pressure to representative shear strength) so as to predict the lateral displacement associated with embankment loading. Subsequently, Xu and Chai [40] proposed a co-relationship between NLD and RLS based on 18 different embankments (case histories) as:

\[
NLD = 0.067 \times RLS + 0.11 \quad (0.05 < RLS < 3.0) \pm 0.05
\]

(1)

Based on the above, two limits for minimum and maximum RLS were used to predict the lateral displacements. The distribution of vertical stress followed the method of Osterberg [41].

Also, Tavenas et al. [42] proposed a relationship between the normalized depth of soft soil (\( Z = z/D \)) and the normalized lateral deformation (\( Y = y/y_m \)) given by:

\[
Y = 1.78Z^3 - 4.7Z^2 + 2.21Z + 0.71
\]

(2)

In the above, \( D \) is depth of soft soil and \( y_m \) is the maximum lateral deformation. The term \( y_m \) is the function of maximum settlement at the centreline of the embankment. Ladd [43] proposed the ratio of maximum lateral deformation to maximum settlement (i.e. \( y_m/s \)) as 0.2 during the fill placement and this value is reported as 0.16 by Tavenas and Leroueil [44] for any embankment with a safety factor greater than 1.3.

Both 2D and 3D simulations were also used to predict the lateral displacements. Since the lateral deformation can be obtained directly from the simulations, there is no further elaboration on how to obtain these results in this section.
4.3 Multi-drain analysis based on Finite Element Modelling

In order to predict the consolidation behaviour of the entire PVD-improved clay foundation, a multi-drain approach is considered in the FEM model to predict the settlement, lateral displacement, and excess pore water pressures over the discretised mesh. In this regard, the unit cell theory is integrated over the entire discretised mesh as explained elsewhere by Indraratna and Redana [33] and Sathananthan and Indraratna [45]. This approach can be used to predict the overall consolidation in a large project where several hundreds of drains are often installed. Complete 3D FEM analysis in such cases may become cumbersome and computationally expensive, and a quicker approach is the transformed equivalent 2D plane strain method, where the strain in the longitudinal direction can be considered very small compared to that of the transverse direction. The transformation includes not only the geometric factors, but also the permeability functions to ensure the same time-settlement profile [20]. Both plane strain (2D) with transformed permeability properties [20] and true 3D analyses constituting individual vertical drain were carried out using commercially available PLAXIS 2D and 3D software [46]. The large deformation calculations were adopted through an updated mesh option available in PLAXIS. The values of $c_k$ have been reported in Table 3. Multi-drain analyses were divided into 3 categories as listed below:

**Case E (CLASS A Prediction):** 2D plane strain analysis using Soft Soil Model – no creep [47]

**Case F (CLASS A prediction):** 3D analysis using Soft Soil Model – no creep [47], and

**Case G (CLASS C prediction):** 2D plane strain analysis - Soft Soil Model with creep [47].

In all of the above FEM analyses, the Hardening Soil Model was adopted for the compacted embankment fill and for the highly over-consolidated surface crust.

The corresponding soil properties are tabulated in Table 3, and the key features of the
Simulations are elucidated below.

Mesh and Material Properties: The water table was assumed to be at the ground surface. Therefore, the soil layers underneath the embankment were assumed to be fully saturated. PVDs of appropriate length were modelled using a drain element [46]. A 6-noded 2D triangular element (Fig. 3a) was used in the plane strain analysis, while a 10-noded tetrahedron 3D soil element (Fig. 3b) was used in 3D analysis. A total of 170432 elements and 229533 nodes formed the mesh discretisation for the 3D simulations, and 11096 elements and 17096 nodes were used in the plane strain simulations. The simulated staged construction followed the actual construction stages in the field.

Plane strain permeability conversion from the true axisymmetric condition in conjunction with the geometric conversions, if carried out properly, can provide accurate pore pressure measurements. Such mathematical conversions for geometry and permeability (e.g. Indraratna and Redana [33]) have been successfully applied for many past case studies of Class A and C predictions (Indraratna et al. [9], [48]).

Following the approach by Indraratna and Redana [33], the horizontal permeability in the undisturbed zone for the plane strain model could be determined based on the in-situ permeability using the following relationship:

$$\frac{k_{h,ps}}{k_h} = \frac{2/3}{\ln(n) - 0.75} \quad (1a)$$

Permeability in the smear zone can be estimated using the following equation:

$$\frac{k_{h,ps}'}{k_{h,ps}} = \frac{\beta}{k_{h,ps}} \left[ \ln(n) + \left( \frac{k_h}{k_{h,ps}} \right) \ln(s) - 0.75 \right] - \alpha \quad (1b)$$

where $k_{h,ps}$ & $k_{h,ps}'$ are the undisturbed horizontal and corresponding smear zone equivalent
permeabilities for the plane strain model, respectively. The geometric parameters $\alpha$ and $\beta$ are then given by:

$$
\alpha = \frac{2}{3} \frac{(n-s)^3}{n^2 (n-1)}
$$

(1c)

$$
\beta = \frac{2(s-1)}{n^2 (n-1)} \left[ n(n-s-1) + \frac{1}{3} (s^2 + s + 1) \right]
$$

(1d)

$$
n = \frac{d_e}{d_w} \quad & s = \frac{d_s}{d_w}
$$

(1e)

Based on these equations, the permeability for the smear zone and undisturbed zone could be determined (Table 3).

5 CHARACTERISTICS OF THE EMBANKMENT AND PVDS

An embankment, 3m high with an 80m long by 15 m wide crest and a side slope set to be 1.5H: 1V was constructed. The embankment was divided into 3 sections: two sections are 30 m long and consist of conventional PVD and Jute PVD (bio-degradable drain); the third section is 20 m long and consists of conventional PVD with a synthetic drain and a geotextile layer instead of a sand drainage layer. Jute drains were installed in one section to compare its performance with the conventional synthetic drains. Construction was carried out in stages and was completed within 60 days. The compacted density of the drainage layer (0.6 - 1m thick) was 15.9kN/m$^3$ whereas the density of the remaining compacted fill was 20.6kN/m$^3$, resulting in a total surcharge load of 59.8kN/m$^3$.

At each half of the embankment, two types of vertical drains including synthetic drains and natural fibre (jute) drains were installed using a rectangular mandrel with a 120mm x 60mm
cross-section to a depth of 14-15m via a static push from an 80t drain stitchers. Rectangular plates (190mm × 90mm) were used as drain anchors while installing the drains in a square pattern at 1.2m spacing. The properties of the conventional (polymeric) wick drains are tabulated in Table 4, and the relevant properties of the jute drains are given in Table 5.

6 FIELD RESULTS AND DISCUSSION

6.1 Embankment centreline behaviour using unit cell analysis

In this section, the field measurements together with a compilation of predictions are compared and discussed. Figure 4 presents the stage construction history, surface settlement and excess pore water pressure at a depth of 6m, close to the centreline of the embankment. The predicted settlement by all methods is acceptably close to the field values (Fig 4b). As expected, Class C prediction using an elastic visco-plastic model yields higher long-term settlement compared to other predictions including Class A, which did not consider the creep effects due to the absence of reliable data from past industry reports. Figure 4c presents the excess pore pressure readings based on the time-dependent settlements at the corresponding depth. However, these measurements are ‘uncorrected’ because of the unavailability of benchmark data at that location. After about 350 days, the rate of dissipation of excess pore pressure becomes considerably retarded. The predicted rate of excess pore water pressure dissipation is much higher compared to the field observations. Certainly, the inclusion of the viscous effect has significantly improved the accuracy of the excess pore water pressure prediction. The comparison in Fig. 5 shows that the viscous soil behaviour can contribute to a retarded dissipation rate of excess pore water pressure. This is because, at a given mean effective stress, the viscous nature of clay causes additional soil compression for a certain
period of time, i.e. without further excess pore water dissipation. However, the remaining (higher) excess pore pressure suggests that there may be other mechanisms to influence the rate of pore pressure build up and/or its dissipation.

**Vibrating Wire Piezometer (VWP) Response:** Based on Figs. 4 and 6, calculated excess pore pressures using analytical methods are relatively lower than those computed by numerical analysis for this particular trial embankment, while the opposite trend has been observed for some other past case studies. In general, the rate of excess pore water pressure dissipation computed using software such as PLAXIS is usually greater compared to the field observations in most case studies conducted by the 1st Author. Sathananthan and Indraratna [45] have shown that the excess pore pressure predictions made by analytical solutions are expected to be different from those computed by numerical analysis. This is because, the excess pore pressure results obtained from analytical solutions are usually averaged at a given depth, while those from numerical analysis are determined at a particular node (point); in this case, the mid-distance between 2 PVDs. The adoption of plane strain equivalent permeability (e.g. Indraratna and Redana [20]) can be employed for direct comparison with pore pressure measurements if the exact location of piezometer is known even at large displacements (measuring tip can move with soil deformation) and also assuming there are no other factors affecting the measurements. An inability to predict excess pore pressure accurately can be attributed to a number of reasons, and we have mentioned a number of possible factors that can affect both the predictions and measurements. Based on our experience in similar floodplain sites in both NSW and QLD where soft Holocene clay governs the foundation behaviour, discrepancies can be attributed to various factors such as characteristics of measuring devices and field monitoring approaches, instrumentation installation techniques.
and associated soil disturbance, seepage and groundwater level fluctuations, routine tidal influx and common incidences of flooding in low-lying floodplains, apart from the obvious assumptions embedded in the adopted theoretical/numerical models and the computational limitations [49, 50, 51].

Figure 7 shows a comparison of a selected VWP response at the Port of Brisbane (POB) with the current study, where both embankments were raised on similar soft upper Holocene clay. In both cases, the piezometer data were recorded at similar depths in the upper Holocene clay at the embankment centreline below the groundwater table, so that the single drain (unit cell) approach could be adopted. For the purpose of comparison, the excess pore water pressure (EPWP) has also been normalised to the maximum fill height ($h_m$). For this selected VWP at POB, the dissipation of EPWP can be predicted very well using the elastic visco-plastic model over 2.5 years, while for the VWP at the Ballina trial embankment site, the same choice of soil model could only match the EPWP up to about 1 year, beyond which the measured data remain higher than the computed values and this disparity increasing with time. There can be various reasons for this marked difference between these two sites. Given that the Ballina site is located within a well-known acid sulphate floodplain with marked levels of iron cations carried by acidic groundwater, one possibility might be that partial clogging due to precipitation of iron compounds may occur in the proximity of measuring devices at low pH values (e.g. Guy et al. [52]). In this respect, future studies to further investigate the reasons for such discrepancies are recommended including the inspection of piezometer filter materials and the possible need for refining the computational models accordingly.
6.2 Multi-drain analysis

Settlements at the centreline of the embankment and the excess pore water pressure obtained from numerical modelling are compared with the measured settlement and excess pore water pressure in Fig. 6. *Class A* settlement predictions by the numerical analysis shown in Fig 6b agree well with the measured data. The inclusion of a more realistic smear zone obtained from the field [24] further improves the accuracy of the predictions. As shown in Figure 8, an additional analysis to examine creep was conducted without PVDs. The creep parameter adopted herein in *Class C* predictions increased the settlement at any given time slightly compared to *Class A*, but did not increase the long term settlements significantly. In the opinion of the authors, this is attributed to the accelerated radial drainage, and in particular the corresponding rapid initial settlement occurring during the first few months promoted by PVDs. Therefore, in the authors’ opinion, unlike in the case without PVD the observed creep settlement would not be pronounced in the longer term (>1.5 years), since accelerated consolidation has already occurred in the immediate to short term. As shown in Fig 6c, the predicted excess pore pressure dissipation rates are very close to the field observations for the initial consolidation period of 1 year, and then tends to deviate. Additional analysis in relation to ground water fluctuation was performed. This is now illustrated in Fig. 9 together with the original *Class C* predictions where the ground water table was assumed to coincide with the surface and 0.5m below the ground surface; the increased excess PWP trend is marginal.

The lateral deformation after 3 years has been predicted for the different cases by various methods, including the Stability Factor Method [37], the Stress Path Method [53], the lower and upper bounds proposed by Xu and Chai [40], and the approach of Tavenas *et al.* [42] as plotted in Fig. 10a. In contrast, the finite element analysis is shown in Fig. 10b. The observed
absolute value of lateral displacement becomes a maximum at a depth of 5-6 m where the softest upper Holocene clay layer is located. The observed lateral displacements do not match the model predictions in the upper layer of soil approaching the surface. However, in the modified stability factor method [37], the value of $\beta$ could be calibrated with the ultimate settlement and then applied to determine the lateral displacement profile giving a much better match with the field data.

There still exist some discrepancies of observed lateral deformation in the uppermost crust (0-2m) and deeper region of the clay layer (12-15m), which cannot be interpreted properly at this stage from these predictions. In the middle of the very soft clay layer (4-5 m deep), the predictions from Case G (Class C) are the closest to the field measurements. With regard to Class A predictions, the approach using the lower bound of Xu and Chai [39] and the stability factor method proposed by Indraratna et al.[37] under-predict the lateral displacement in the soft upper clay layers, while a better match is obtained for the deeper clay deposits. The discrepancy between the Class A models and the field data is more evident towards the top weathered crust (0-2 m). It is found that the lateral strain obtained based on Case E FEM plane strain model is at least 30% greater at 2-6 m deep than that computed from 3D modelling (Case F). This is not surprising, because, in 2D plane strain modelling a zero strain is prescribed in the longitudinal direction, which in turn results in an increased strain in the transverse direction. The inclusion of soft soil creep (Case G) provides similar lateral displacement profile to that of Case E. Based on the field measurements, the back calculated values of $\alpha$, $\beta_1$ and $\beta_2$ [37] are 0.04, 0.124 and 0.51, respectively. Irrespective of the different properties of jute drains compared to polymeric wick drains, the measured settlement, excess pore water pressure and lateral displacement for the two types of drains are very similar to
each other.

7 COMPARISON OF SETTLEMENT IN TERMS OF PAST INDUSTRY DATA AND CGSE SITE DATA

Further analyses (Class C) have been performed to evaluate the settlement of the trial embankment using different sets of soil properties. Figure 11 illustrates these Class C predictions in comparison with the field data (i.e. denoted by open circles), as described below:

(a) Soil properties based on past industry data [27; Table 1&2] transferred to the actual CGSE site profile (i.e. 10 m of Holocene clay followed by a transition layer to the underlying sand), but ignoring creep.

(b) Same as above but including creep properties evaluated from large-scale undisturbed specimens [26].

(c) Carefully evaluated stress path data from for the CGSE site including creep properties based on Pineda et al [30].

The comparisons clearly show that when creep is captured, there is little difference between (b) and (c), and both plots are in close agreement with the measured settlement. This is not surprising because for a 3 m high embankment, the distributed vertical stress (Boussinesq elastic theory) below 10 m of depth is quite small (less than 30 kPa), and as a result whether the soil at that depth (> 10 m) is dense Holocene sand (CGSE site) or relatively stiff Holocene clay (RTA-Pacific Highway report) does not make a significant influence on the settlement predictions. The comparison further suggests that past industry standard data was good enough to provide an acceptable match with the field measurements (open circles) during the initial year or so, whereas the use of Pineda [30] data provides an improved comparison with
field data during later stages when the long-term creep tends to be pronounced.

8 CONCLUSION

The application of prefabricated vertical drains (PVD) combined with surcharge preloading is an effective method for accelerating soft soil consolidation. In this study, the radial consolidation behaviour of the soft estuarine clay foundation beneath the Ballina trial embankment (3m surcharge fill) could be predicted using available analytical and numerical tools (Class A and Class C).

Integrated unit cell (centreline-single drain) and multi-drain analyses were carried out adopting the geotechnical properties available from past published works including standard testing reports provided by industry, as well as through more sophisticated experimental testing on both conventional and large-diameter undisturbed soil specimens. The smear zone could be adequately characterised using the undisturbed samples obtained at various radii from a vertical drain. The ratio of the permeability in the undisturbed zone to that in the smear zone (k_u/k_s), and the normalised size of smear zone (d_s/d_w) for this trial embankment were evaluated to be 2.3 and 7.8, respectively.

For the unit cell (single drain) analysis, various approaches considering non-Darcian flow, non-linear variation of soil compressibility and soil permeability, large-strain condition and elastic, visco-plastic properties were considered in making these Class A and Class C predictions. It was shown that the centreline settlement predictions agreed generally well with the field data (Class A), but the ultimate settlement could only be matched correctly when the visco-plastic (creep) behaviour was captured in the analytical and numerical methods.

The build-up of excess pore water pressure upon construction loading, its peak and initial dissipation trend could be predicted reasonably well once the visco-plastic nature of the soft
clay was incorporated. In spite of the relatively closely spaced drains, the excess pore water pressures at some piezometer locations may not dissipate as fast as one expects. In particular, beyond a period of say 1-1.5 years of consolidation, further refinement seems prudent through future efforts to insightfully interpret the correct rate of pore pressure dissipation. Moreover, where warranted various factors that may influence measurements in a given site may be investigated more to improve our ability to better interpret the observed field measurements as well as to refine the computational models accordingly. For multi-drain analyses, the settlement predictions agreed well with the field data for both 2D and 3D cases, albeit the measured excess pore pressures remaining at significantly higher levels than the predicted values, especially after 1-1.5 years as mentioned earlier. The accuracy of predicting the lateral movement at the embankment toe relies on the careful selection and evaluation of soil properties. The comparisons show that the empirical formulations, analytical models and numerical analyses presented here still require further refinement to accurately predict the lateral displacement profile, especially nearing the surface (compacted) crust. Not surprisingly, the maximum lateral displacement was observed at a depth of 5-6 m in the softest clay layer. Overall, based on the field measurements and comparisons with the predictions, the magnitudes of parameters $\alpha$, $\beta_1$ and $\beta_2$ (stability factor method, [37]) in the order of 0.04, 0.124 and 0.51, respectively, provided an acceptable Class C prediction in relation to the measured settlement, excess pore pressure and lateral displacement.

9 ACKNOWLEDGMENT

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10 REFERENCES


Appendix A: Nomenclature

$\bar{C}_c$  average compression index
$C_c$ Compressibility index
$C_p$ coefficient of radial consolidation
$C_k$ permeability index
$C_r$ Re-compression index
$C_s$ recompression index
$C_v$ coefficient of vertical consolidation
$e_0$ initial void ratio
$d_w$ diameter of drain
$d_s$ diameter of smeared zone
$D_e$ equivalent diameter of the influence zone
$\bar{e}_0$ average initial void ratio
$k_s$ permeability in smear zone
$k_h$ horizontal coefficient of permeability for axisymmetry in undisturbed zone
$k_h'$ horizontal coefficient of permeability for axisymmetry in smear zone
$k_{h,ps}$ horizontal coefficient of permeability for plane strain in undisturbed zone
$k_{h,ps}'$ horizontal coefficient of permeability for plane strain in smear zone
$R$ radius of axisymmetric unit cell
$r_s$ radius of smear zone
$r_w$ radius of vertical drain
$n$ ratio of equivalent diameter of soil cylinder to drain diameter
$s$ ratio of smeared diameter to drain diameter
$l$ length of vertical drain
$LL$ liquid limit
**PL**  plastic limit

**w_p**  natural water content

**PSD**  particle size distribution

**m_v**  coefficient of volume compressibility

**γ**  unit weight of soil

**P_{av}**  constant as a function of over consolidation ratio

**P, P_0**  vacuum pressure (negative)

**Q**  fill surcharge loading

**q_w**  discharge capacity

**R_{td}**  Excess pore pressure ratio

**t_i**  time (when \( \sigma'_v = \bar{\sigma}'_{vt} \))

**T_{ho} T_{ho}**  dimension-less time factor

**u_{t,j}^i**  pore water pressure at i,j co-ordinate at time t.

**C^I C^{III}**  parameters used in P-R FDI analysis.

**ΔT_r**  constant used in EVP analysis in r-direction

**ΔT_v**  constant used in EVP analysis in v-direction

**V**  specific volume

**λ**  slope of reference time line

**κ**  slope of instant time line

**ψ**  slope of fitted creep curve

**g(u, ε_z)**  parameter used during EVP analysis

**U_i**  degree of consolidation at time t

**\bar{u}_0**  average pore water pressure

**α_c**  non-Darcian flow parameter

**α_c/α'_c**  ratio of smeared zone in non-Darcian flow
Appendix B: Summary of mathematical functions for the governing equations of Cases A-D

CASE A: Kianfar et al. [12] presented a radial consolidation model to capture non-linear relationship between the flow velocity and the hydraulic gradient (non-Darcian law). They also consider the non-linear relationship of soil compressibility and permeability with the void ratio. The average excess pore water pressure ($\bar{u}$) can then be determined as:

$$
\bar{u} = \left\{ (1 - \beta) \left( -\frac{2\alpha \varepsilon}{m_w} \right) \frac{n^2 - 1}{2n^2} \frac{R^2}{\eta w} \right\}^{\beta} t + (\bar{u}_0)^{(1-\beta)} \right\}^{1/(1-\beta)}
$$

(A.1)
\[
\varepsilon = -m_p \left\{ \left(1 - \beta \right) \left( -\frac{2a_c}{m_p} \right) \left[ \frac{m^2-1}{2m^2} \frac{R^2}{\eta \gamma_w} \right]^{\beta} \left[ t + \left( \bar{u}_0 \right)^{(1-\beta)} \right]^{\frac{1}{1-\beta}} - \left( \bar{u}_0 \right) \right\} 
\]  
(A.2)

\[
n = \frac{R}{r_w} 
\]  
(A.3)

\[
\eta_n = \sum_{i=0}^{\infty} \left( \frac{1}{i!} \right) \left( -1 \right)^i \left( R^2 \right)^{\frac{1}{i-1}} \left[ r^j + \left( c_a - 1 \right) r^j_s - c_a r^j_w \right] 
\]  
(A.4)

\[
c_a = \left( \frac{k}{k_s} \right)^{\frac{1}{\beta}} 
\]  
(A.5)

where \( \bar{u} \) is the average excess pore water pressure in the unit cell, \( m_p \) is the coefficient of the soil volume compressibility, \( \bar{u}_0 \) is the initial average excess pore water pressure in the unit cell, \( k_s \) and \( \beta \) are constants which depend on the type of soil and flow relationship in the smear zone, \( r_w \) and \( r_s \) are the radii of the drain and smear zone, respectively. \( \gamma_w \) = unit weight of water.

CASE B: Perera et al. [32] captured the effect of soil disturbance caused by mandrel driving including the variations of permeability and compressibility in the smear zone, and the corresponding role of void ratio, thus improving on the original derivations of Indraratna and Redana [20,33]. The excess pore water pressure ratio at any time \( t \) at depth \( z \) can be determined as follows:

\[
R_u = \left( \frac{u_0}{\Delta \sigma} \right) \times \exp \left\{ - \left[ \frac{\sigma'_y}{\sigma_0'} \right]^{1-\left( \frac{\varepsilon_f}{c_k} \right)} + 1 \right\} \frac{4 \tau_{ha}}{\mu} \quad \sigma' \leq \sigma'_y, t \leq t_i, \quad \text{ (A.6)}
\]

\[
R_u = \left( \frac{\varepsilon_0 + u_0 - \sigma'_y}{\Delta \sigma} \right) \times \exp \left\{ - \left[ \frac{\sigma'_y}{\sigma_0'} \right]^{1-\left( \frac{\varepsilon_f}{c_k} \right)} + 1 \right\} \frac{4 \tau_{ha}}{\mu} \quad \sigma' > \sigma'_y, t > t_i \quad \text{ (A.7)}
\]

\[
t_i = \frac{\mu d^2}{4 c_{ha} \left[ \frac{\sigma'_y}{\sigma_0} \right]^{1-\left( \frac{\varepsilon_f}{c_k} \right)} + 1} \ln \left( \frac{u_0}{\sigma'_y + u_0 - \sigma'_y} \right) \quad \text{ (A.8)}
\]
\[
\mu = \ln \left( \frac{n}{s} \right) - \frac{3}{4} + \frac{\kappa(s-1)}{s-k} \ln \left( \frac{s}{\kappa} \right)
\] (A.9)

\[
T_{h_0} = P_{av,0}T_{h_0} = 0.5 \left[ \frac{\sigma'_{f}}{\sigma_{o}} \right]^{1-\left(\frac{c_{s}/c_k}{\sigma'_{y}}\right)} + 1 \] \quad \bar{\sigma}' \leq \bar{\sigma}'_{y}
\] (A.10)

\[
P_{av,0} = 0.5 \left[ \frac{\sigma'_{f}}{\sigma_{o}} \right]^{1-\left(\frac{c_{s}/c_k}{\sigma'_{y}}\right)} + 1 ; \quad \bar{\sigma}' \leq \bar{\sigma}'_{y}
\] (A.11)

\[
T_{h_i} = P_{av,i}T_{h_i} = 0.5 \left[ \left(\frac{\sigma'_{o}+\Delta\sigma}{\sigma'_{y}}\right)^{1-\left(\frac{c_{s}/c_k}{\sigma'_{y}}\right)} + 1 \right] \] \quad \bar{\sigma}' > \bar{\sigma}'_{y}
\] (A.12)

\[
P = P_{av,i} = 0.5 \left[ \left(\frac{\sigma'_{o}+\Delta\sigma}{\sigma'_{y}}\right)^{1-\left(\frac{c_{s}/c_k}{\sigma'_{y}}\right)} + 1 \right] ; \quad \bar{\sigma}' > \bar{\sigma}'_{y}
\] (A.13)

where \( R_{e} \) is the excess pore water pressure ratio, \( \bar{c}_{c} \) is the average compression index for a given stress range in a normally consolidated region, and \( c_{s} \) is the recompression index in the over-consolidation region, \( c_{k} \) is the permeability index, \( \bar{\sigma}'_{y} \) is pre-consolidation stress (yield stress) of the average curve, \( \sigma'_{o} \) is effective vertical stress at initial stage, \( u_{o} \) is excess pore water pressure, \( \Delta \sigma' \) is total effective stress change, \( t_{i} \) is the time required for soil to change from an over-consolidated state into a normally-consolidated state.

**CASE C:** Indraratna et al. [34] recently proposed an analytical solution for radial consolidation based on the large strain concept incorporating the non-Darcian flow. According to the traditional 1D large strain consolidation [54], the relationship between Lagrangian coordinate \( a \) and the convective coordinate \( \xi \) can be expressed as:

\[
\frac{\partial \xi}{\partial \mu} = \frac{1+e}{1+e_{i}}
\] (A.14)

where, \( e \) is the current void ratio and \( e_{i} \) is the initial void ratio.

Based on the above relationship, radial flow with large-strain condition can be established as:
\[
\frac{1}{1+e_i} \frac{\partial e}{\partial t} \, da + v_i(r) \frac{2r}{r^2 - r_i^2} \frac{\partial \xi}{\partial a} \, da = 0
\]  
(A.15)

where \( r \) is the radius, \( r_i \) is the radius of the influential area, \( t \) is the time, and \( v_i(r) \) is the inward seepage velocity at radius \( r \).

Non-Darcian flow representing an exponential relationship between the seepage velocity and the hydraulic gradient can be given by Hansbo [55]:

\[
v_i = k_i^\beta = k \left( \frac{1}{\gamma_w} \frac{\partial u}{\partial r} \right)^\beta
\]  
(A.16)

where \( k \) is the coefficient of non-Darcian permeability (\( k = k_i \) and \( k_o \) inside and outside smear zone, respectively), \( \beta \) is the non-Darcian flow exponent, \( i_r \) is the hydraulic gradient in radial direction, \( u \) is the excess pore pressure and \( \gamma_w \) is the unit weight of water.

To consider the variations of permeability and compressibility during consolidation, the following constitutive relationships were adopted:

\[
e = e_o - C_e \log \left( \frac{\sigma'_{\gamma}}{\sigma'_{\gamma_0}} \right)
\]  
(A.17)

\[
e = e_o + C_k \log \left( \frac{k}{k_0} \right)
\]  
(A.18)

In the above, \( \sigma'_{\gamma} \) is the average effective stress within the influential area; \( C_e \) and \( C_k \) are the compression index and permeability change index, respectively; \( e_o \) is a reference void ratio; and \( \sigma'_{\gamma_0} \) and \( k_0 \) are the effective stress and coefficient of permeability corresponding to \( e_0 \).

By substituting Eqs. (A.16)–(A.18) into Eq. (A.15) and subsequent mathematical manipulation, a
governing equation that accounts for the large-strain with non-linear variations of soil compressibility and soil permeability is obtained:

$$\frac{\partial \sigma'}{\partial t} = 2\ln10k_{hi} \frac{\sigma'}{C_e} \Gamma (\sigma'_n + Q - P - \sigma')\beta$$  \hspace{1cm} (A.19)

where $k_{hi}$ is the initial coefficient of horizontal permeability outside the smear zone,

$$\Gamma = \exp\left[ \ln10\left( \frac{1}{C_k} - \frac{1}{C_e} \right)(e - e_i) \right](1 + e), \quad Q \text{ and } P \text{ (negative value)} \text{ are the fill surcharge loading and vacuum pressure, respectively, and } \Theta \text{ is given by:}$$

$$\Theta = \frac{2n^2}{n^2 - 1} \frac{\gamma c \varphi}{r_c^2}$$  \hspace{1cm} (A.20)

$$\varphi = r_c^{\frac{1}{n^2}} \sum_{n=0}^{\alpha} \left( 1 + \frac{1}{n^2} \right)^k \left[ \left( k_{hi} \right)^2 \right] \left( 1 - 2s \right)^{-1} \left( k_{hi} \right)^{\frac{1}{2}}$$  \hspace{1cm} (A.21)

In the above, \( \frac{1/\beta - 1}{i!} \) for \( i > 0 \); \( j = 2i - 1/\beta + 1 \), \( n = \frac{r_s}{r_e} \), \( s = \frac{r_s}{r_e} \), \( r_s \) is the radius of the smear zone, and \( k_s \) and \( k_h \) are the horizontal coefficients of permeability inside and outside the smear zone, respectively.

**CASE D**: Yin and Graham [35] proposed an elastic visco-plastic model (EVP) based on the equivalent time line concept, which has since been successfully calibrated by others using laboratory and field data. The model has been further modified by Baral [26] using non-Darcian flow for radial consolidation and solved using Finite difference method called Peaceman-Rachford ADI-FDM scheme, whereby the matrix form for predictor and corrector have been written in MATLAB to
facilitate computation of settlement and excess pore water pressure dissipation. The governing equation can be expressed as:

\[
m_v \frac{\partial u}{\partial t} - g(u, \varepsilon_2) = \frac{\alpha_c \beta}{\gamma_m} \left[ \frac{\partial u}{\partial r} \right]^{\beta - 1} \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) + \left| \frac{\partial u}{\partial z} \right|^{\beta - 1} \frac{\partial^2 u}{\partial z^2} \tag{A.22}
\]

where,

\[
g(u, \varepsilon_2) = \frac{\psi}{\nu_0 \nu} \exp \left[ - (\varepsilon_2) \frac{V}{\psi} \left( \frac{\sigma_x - u}{\sigma_{0x}} \right)^\lambda \psi \right]
\]

On further simplification using the Peaceman Rachford (P-R) scheme, the above equations can be estimated by the following steps.

The matrix form of predictor is:

\[
u_{i,j}^{t+\frac{\Delta t}{2}} = C^I u_{i+1,j}^{t+\frac{\Delta t}{2}} + C^{II} u_{i-1,j}^{t+\frac{\Delta t}{2}}
\tag{A.23}

For each \( j \),

\[
\begin{align*}
1 & -C_{1,j}^{II} & 0 \\
-\frac{1}{2} & 1 & -\frac{1}{2} \\
0 & -\frac{1}{2} & 1 \\
M & O & -\frac{1}{2}C_{N-1,j}^{III} \\
-n & -\frac{1}{2}C_{N,j}^{II} - C_{N,j}^{III} & 1
\end{align*}
\]

\[
\begin{bmatrix}
u_{1,j}^{t+\frac{\Delta t}{2}} \\
u_{2,j}^{t+\frac{\Delta t}{2}} \\
u_{3,j}^{t+\frac{\Delta t}{2}} \\
u_{N-1,j}^{t+\frac{\Delta t}{2}} \\
u_{N,j}^{t+\frac{\Delta t}{2}}
\end{bmatrix}
= 
\begin{bmatrix}
1 & C^I_{1,j} & u_{0,j+\frac{\Delta t}{2}}^{t+\frac{\Delta t}{2}} \\
C^I_{2,j} & C^I_{2,j} & M \\
C^I_{3,j} & C^I_{3,j} & M \\
C^I_{N-1,j} & C^I_{N-1,j} & M \\
C^I_{N,j} & C^I_{N,j} & M
\end{bmatrix}
\begin{bmatrix}
u_{1,j} \\
u_{2,j} \\
u_{3,j} \\
u_{N-1,j} \\
u_{N,j}
\end{bmatrix}
\]

Where,
\[
C^I = \frac{(1 - 2\Delta T_v)}{(1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r})} u_{i,j}^t + \frac{\Delta T_v}{(1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r})} (u_{i,j+1}^t + u_{i,j-1}^t)
+ \frac{\Delta t}{(1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r})} g(u, \varepsilon_{i,j})
\]
\[
C^{II} = \frac{\Delta T_r \cdot (1 + \frac{\Delta r}{r})}{(1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r})}
\]
\[
C^{III} = \frac{\Delta T_r}{(1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r})}
\]

The matrix form of corrector is:
\[
\begin{bmatrix}
1 & -C^{II}_{i,1} & 0 \\
-C^{III}_{i,2} & 1 & -C^{III}_{i,3} \\
0 & -C^{III}_{i,1} & 1 \\
\end{bmatrix}
\begin{bmatrix}
u_{i,1}^t \\
u_{i,2}^t \\
u_{i,3}^t \\
\end{bmatrix}
\begin{bmatrix}
1 & 0 & 0 \\
0 & 1 & 0 \\
0 & 0 & 1 \\
\end{bmatrix}
\begin{bmatrix}
u_{i,j}^{t+\Delta t} \\
u_{i+1,j}^{t+0.5\Delta t} \\
u_{i-1,j}^{t+0.5\Delta t} \\
\end{bmatrix}
= \begin{bmatrix}
C^{I}_{i,1} + C^{III}_{i,1} u_{i,0} u_{i+2M} \\
C^{I}_{i,2} \\
C^{I}_{i,3} \\
\end{bmatrix}
\begin{bmatrix}
1 & 0 & 0 \\
0 & 1 & 0 \\
0 & 0 & 1 \\
\end{bmatrix}
\begin{bmatrix}
C^{I}_{i,M} \\
C^{I}_{i,M-1} \\
\cdots \\
C^{I}_{i,1} \\
\end{bmatrix}
\]

where,
\[
C^{I} = \frac{(1 - 2\Delta T_v - \Delta T_r \cdot \frac{\Delta r}{r})}{(1 + 2\Delta T_v)} u_{i,j}^t + \frac{\Delta T_v}{(1 + 2\Delta T_v)} \frac{t+\Delta t}{2} u_{i,1,j}^t + \frac{\Delta T_r}{(1 + 2\Delta T_v)} \frac{t+\Delta t}{2} u_{i-1,j}^t
+ \frac{\Delta t}{m_v (1 + 2\Delta T_v)} g(u, \varepsilon_{i,j})
\]
\[
C^{II} = \frac{\Delta T_v}{(1 + 2\Delta T_v)}
\]
\[
C^{III} = \frac{\Delta T_r}{(1 + 2\Delta T_v)}
\]
Figure 1: Basic soil properties of Ballina clay (Source: Indraratna et al. [29])
Figure 2: Radial flow characteristics using large-scale consolidometer

\[ v = 5.3 \times 10^{-10} \text{ m/s} \]

\[ R^2 = 0.985 \]
Figure 3: Mesh discretisations (a) 2D Plane strain model and (b) 3D Model
Figure 4: Unit cell analyses: (a) stage construction (b) surface settlements and (c) excess pore pressures near the centreline at 6m below the ground surface.
Figure 5: Excess pore pressure retardation due to viscous nature of clay
Figure 6: Multi-drain analyses: (a) stage construction (b) surface settlements and (c) excess pore pressures near the centreline at 6m below the ground surface
Figure 7: Prediction of VWP behaviour using EVP-FDM model at the POB and current Ballina embankment

[Note: VW piezometers are located in upper Holocene clay at a depth of -6m]
Figure 8: Effect of radial consolidation on surface settlements using Soft Soil (SS) and Soft Soil Creep (SSC) model.

Figure 9: Effect of lowering of ground water table on excess pore water pressure reading at 6m below the ground surface.
Figure 10: Comparison of lateral deformation with (a) existing analytical methods (b) FEM
Figure 11: Comparison between industry standard data and laboratory tested specimens.
### Table 1 Industry standard soil properties used for CLASS A (Case A, B & C) prediction approach.

<table>
<thead>
<tr>
<th>Layer no.</th>
<th>Thickness (m) (A,B,C)</th>
<th>$\bar{\sigma}_0^{(B,C)}$ (kPa)</th>
<th>$\sigma_f^{(B,C)}$ (kPa)</th>
<th>$\bar{\sigma}_{\text{pp}}^{(B)}$ (kPa)</th>
<th>$\sigma_{\text{pp},B}^{(C)}$ (kPa)</th>
<th>$C_r^{(B,C)}$</th>
<th>$C_r^{(B)}$</th>
<th>$C_r^{(C)}$</th>
<th>$\varphi_0^{(B,C)}$</th>
<th>OCR (B,C)</th>
<th>$m_p^{(A)}$ (m$^3$/kN)</th>
<th>$\alpha_c \times 10^{-10}$ (m$^{-1}$)</th>
<th>$P_{\text{av}}$ (m/s)</th>
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<td>1</td>
<td>2.7</td>
<td>9.3</td>
<td>67.9</td>
<td>58.6</td>
<td>46.3</td>
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<td>1.40</td>
<td>1.55</td>
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<td>0.00225</td>
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<td>75.0</td>
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<td>0.27</td>
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<td>1.37</td>
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<td>2.8</td>
<td>1.2</td>
<td>0.00212</td>
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<td>3.0</td>
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<td>83.1</td>
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<td>44.1</td>
<td>0.43</td>
<td>1.33</td>
<td>1.68</td>
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<td>1.2</td>
<td>0.00038</td>
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<td>99.9</td>
<td>34.3</td>
<td>65.6</td>
<td>0.20</td>
<td>1.11</td>
<td>1.52</td>
<td>1.35</td>
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<tr>
<th>D_c (mm)(A,B,C)</th>
<th>d_w (mm)(A,B,C)</th>
<th>d_s (mm)(A,B,C)</th>
<th>$\alpha_c/\alpha_c^{(C)}$</th>
<th>$q_u$ (m$^3$/s)(A,B,C)</th>
<th>n(A,B,C)</th>
<th>s(A,B,C)</th>
<th>$\beta^{(C)}$</th>
<th>$\eta^{(C)}$</th>
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<tbody>
<tr>
<td>1356</td>
<td>51.5</td>
<td>400</td>
<td>2</td>
<td>$8 \times 10^{-3}$</td>
<td>26.33</td>
<td>7.8</td>
<td>1.3</td>
<td>0.23</td>
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Table 2 Soil properties used for Class C (Case D) prediction approach

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<tr>
<th>Layer no.</th>
<th>Thickness (m)</th>
<th>(k/\nu)</th>
<th>(\lambda/\nu)</th>
<th>(\psi/\nu)</th>
<th>(e_0)</th>
<th>(\alpha_c) (m/s)</th>
<th>(\gamma_s) (kN/m³)</th>
<th>(\sigma^{'vp}) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.7</td>
<td>0.034</td>
<td>0.148</td>
<td>0.003</td>
<td>3.1</td>
<td>5.3\times10^{-10}</td>
<td>14.5</td>
<td>46.3</td>
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<tr>
<td>2</td>
<td>3.0</td>
<td>0.062</td>
<td>0.156</td>
<td>0.004</td>
<td>2.8</td>
<td>4.9\times10^{-10}</td>
<td>13.7</td>
<td>27.2</td>
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<tr>
<td>3</td>
<td>3.0</td>
<td>0.098</td>
<td>0.192</td>
<td>0.004</td>
<td>2.8</td>
<td>4.8\times10^{-10}</td>
<td>14.2</td>
<td>44.1</td>
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<tr>
<td>4</td>
<td>3.0</td>
<td>0.107</td>
<td>0.138</td>
<td>0.004</td>
<td>2.8</td>
<td>5.5\times10^{-10}</td>
<td>14.2</td>
<td>50.8</td>
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<tr>
<td>5</td>
<td>3.3</td>
<td>0.047</td>
<td>0.180</td>
<td>0.003</td>
<td>2.7</td>
<td>6.2\times10^{-10}</td>
<td>14.2</td>
<td>65.6</td>
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</table>

<table>
<thead>
<tr>
<th>(D_s) (mm)</th>
<th>(d_w) (mm)</th>
<th>(d_i) (mm)</th>
<th>(\alpha_c/\alpha'_c)</th>
<th>(q_w) (m³/s)</th>
<th>(n)</th>
<th>(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1356</td>
<td>51.5</td>
<td>400</td>
<td>2</td>
<td>8\times10^{-3}</td>
<td>26.33</td>
<td>7.8</td>
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</table>
### Table 3 Material parameters used for multi drain analysis

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<tr>
<th>Parameter(s)</th>
<th>Embankment</th>
<th>Sand blanket</th>
<th>Layer1</th>
<th>Layer2</th>
<th>Layer3</th>
<th>Layer4</th>
<th>Layer5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material model</td>
<td>Hardening</td>
<td>Hardening</td>
<td>Hardening</td>
<td>Soft-soil</td>
<td>Soft-soil</td>
<td>Soft-soil</td>
<td>Soft-soil</td>
</tr>
<tr>
<td>Thickness of subsoil layer (m)</td>
<td>2.7</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.3</td>
<td></td>
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</tr>
<tr>
<td>Unit weight above water table ($\gamma_{unsat}$, kN/m$^3$)</td>
<td>20.6</td>
<td>15.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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</tr>
<tr>
<td>Unit weight below water table ($\gamma_{sat}$, kN/m$^3$)</td>
<td>20.6</td>
<td>15.9</td>
<td>14.5</td>
<td>13.7</td>
<td>14.2</td>
<td>14.2</td>
<td>14.2</td>
</tr>
<tr>
<td>Initial void ratio ($e_{init}$)</td>
<td>0.5</td>
<td>0.5</td>
<td>3.1</td>
<td>2.8</td>
<td>2.8</td>
<td>2.8</td>
<td>2.7</td>
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<tr>
<td>Secant stiffness ($E_{50}^{ref}$, kN/m$^2$)</td>
<td>5000</td>
<td>6000</td>
<td>1000</td>
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<tr>
<td>Tangent stiffness ($E_{oed}^{ref}$, kN/m$^2$)</td>
<td>3435</td>
<td>4328</td>
<td>434.5</td>
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<tr>
<td>Un/Reloading stiffness ($E_{ur}^{ref}$, kN/m$^2$)</td>
<td>15000</td>
<td>18000</td>
<td>3000</td>
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<td>Power for stress level (m)</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
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<tr>
<td>Modified compression index ($\lambda^*$)</td>
<td>0.156</td>
<td>0.192</td>
<td>0.138</td>
<td>0.180</td>
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<tr>
<td>Modified swelling index ($\kappa^*$)</td>
<td>0.062</td>
<td>0.098</td>
<td>0.107</td>
<td>0.047</td>
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<td>Cohesion (kN/m$^2$)</td>
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<td>0.025</td>
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<td>5.0</td>
<td>5.0</td>
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<td>Friction angle</td>
<td>32.0</td>
<td>35.0</td>
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<td>Dilatancy angle</td>
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<td>5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>Horizontal permeability ($k_x$, m/day)</td>
<td>0.65</td>
<td>1.30</td>
<td>1.24×10$^{-4}$</td>
<td>1.04×10$^{-4}$</td>
<td>1.04×10$^{-4}$</td>
<td>1.04×10$^{-4}$</td>
<td>1.28×10$^{-4}$</td>
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<tr>
<td>Vertical permeability ($k_z$, m/day)</td>
<td>0.65</td>
<td>1.30</td>
<td>0.62×10$^{-4}$</td>
<td>0.52×10$^{-4}$</td>
<td>0.52×10$^{-4}$</td>
<td>0.52×10$^{-4}$</td>
<td>0.64×10$^{-4}$</td>
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<tr>
<td>Over-consolidation ratio</td>
<td>1.2</td>
<td>1.2</td>
<td>1.0</td>
<td>1.0</td>
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<tr>
<td>At rest, lateral pressure coefficient ($K_{0NC}$)</td>
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<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
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<tr>
<td>Permeability change index ($C_u$)</td>
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<td>1.4</td>
<td>1.4</td>
<td>1.35</td>
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<tr>
<td>Equivalent permeability ($k_{h,eq}$, m/day)</td>
<td>2.76×10$^{-3}$</td>
<td>2.76×10$^{-3}$</td>
<td>2.76×10$^{-3}$</td>
<td>3.40×10$^{-3}$</td>
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<td>Creep parameter for soft soil creep model ($\mu^*$)</td>
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<td>0.004</td>
<td>0.004</td>
<td>0.003</td>
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<td>Table 4: Properties of wick drains (Source: ceteau.com/wickdrain.html)</td>
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<td>Shape of core</td>
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<td>Drainage channels</td>
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<tr>
<td>Tensile strength</td>
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<tr>
<td>Discharge capacity</td>
<td>8 × 10⁻⁵ m³/s (at 300 kPa)</td>
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<tr>
<td></td>
<td>2.5 × 10⁻⁵ m³/s (buckled: at 200 kPa and 25% deformation)</td>
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<td>6 × 10⁻⁵ m³/s (kinked: at 300 kPa)</td>
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<td>Grab tensile strength filter</td>
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<td>Tear strength filter</td>
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<td>Permittivity filter</td>
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<td>Weight (g/m²)</td>
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<td>Permeability at 100 mm water head (l/m²/s)</td>
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<td>Puncture resistance (kN)</td>
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<td>Tensile strength of fresh Jute drain (for given cross-section, kN)</td>
<td>3.4</td>
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<tr>
<td>Tensile strength of jute drain pulled out of embankment (for given cross section, kN)</td>
<td>1.5-1.8 kN</td>
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