Performance of a trial embankment at the Ballina soft soil Field Testing Facility

Performance d'un remblai d'essai sur le site expérimental de l'argile molle de Ballina

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ABSTRACT: This paper summarises findings of laboratory, in situ and full-scale tests performed at the National Soft Soil Field Testing Facility, established by the ARC Centre of Excellence for Geotechnical Science and Engineering near Ballina, New South Wales. A 9m-thick soft estuarine clay deposit dominates the subsoil profile there, which is typical of foundation conditions along the eastern and southern coasts of Australia. The variation of index and mechanical properties with depth is presented along with measurements of settlement, pore pressure dissipation and lateral deformation below a 3m-thick trial embankment on PVD-improved soil, over a period of 3 years. The presentation concludes with a short discussion on the key mechanisms governing the observed response, essential for the prediction of the behaviour of embankments on similar soft soils.

RÉSUMÉ : Cet article résume les résultats des essais au laboratoire et in situ ainsi que d'un remblai d'essai grandeur nature sur le site expérimental de l'argile molle de Ballina, établi par le Centre d'excellence de A.R.C pour la science et l'ingénierie géotechnique, près de Ballina, Nouvelle-Galles du Sud (NSW). Un dépôt d'argile molle estuarienne à une épaisseur de 9 m domine le profil du sous-sol sur ce site, qui est représentatif de conditions de fondation le long des côtes orientales et méridionales de l'Australie. La variation des indices et des propriétés mécaniques avec la profondeur est présentée ainsi que les mesures de l'tassement, de la dissipation de la pression d'eau interstitielle et de la déformation latérale au-dessous d'un remblai d'essai de 3 m d'épaisseur sur le sol amélioré par des drains verticaux préfabriqués (PVD), pour une période de 3 ans. La présentation se termine par une courte discussion sur les mécanismes clés gouvernant la réponse observée, essentiels pour la prédiction du comportement des remblais sur des sols mous similaires.

KEYWORDS: soft clays; in situ testing; laboratory tests; trial embankment; prefabricated vertical drains

1 INTRODUCTION

The establishment of a soft soil testing facility by the ARC Centre of Excellence for Geotechnical Science and Engineering (CGSE) near Ballina, New South Wales was motivated by the difficulties encountered during the construction of a nearby motorway section: Embankments with maximum fill height 14m settled up to 6.4m over a period of 3 years, while accurate prediction of settlement magnitude and rate of evolution of embankment deformations proved to be challenging (Kelly 2014). The reason is that these fills were built on estuarine soft clay deposits, commonly found along the eastern and southern Australian coastlines. Australian high- to extremely high plasticity estuarine clays exhibit low undrained shear strength and high compressibility, and are characterised by presence of electrolytes in the pore fluid, traces of organic matter and expansive minerals as well as weak cementation. All these features render modelling of their mechanical behaviour particularly demanding.

The establishment of the National Soft Soil Field Testing Facility (NFTF) by the CGSE has allowed thorough in situ investigations to be combined with advanced laboratory tests on high-quality samples to characterise a representative Australian estuarine soft clay. In addition, two full-scale trial embankments were constructed in 2013 (Kelly *et al.* 2014), one on subsoil improved with both conventional wick drains and biodegradable jute drains (PVD, Fig. 1). This embankment, the performance of which is discussed in this paper, was extensively instrumented both during and after construction. Measurements of pore pressures, vertical and horizontal deformations, as well as vertical and lateral soil pressures, were obtained at a large number of locations over a period of 3 years. Accordingly, a numerical prediction symposium was held in Newcastle, Australia on 12 and 13 September 2016, which attracted 28 Class A predictions of the performance of the abovementioned PVD embankment from Australian and international academics and practitioners. Lessons learned from this symposium are the subject of a special issue, to be published in *Computers and Geotechnics* in 2017.

In this paper, we present findings from the laboratory and in situ characterisation studies that are of key importance for compiling a detailed geotechnical model of the soft estuarine silty clay deposits, referred to hereinafter as Ballina clay. Next, elements of the performance of the trial embankment are presented in short, and we discuss certain key mechanisms that need to be properly captured in an analysis model to simulate its behaviour.

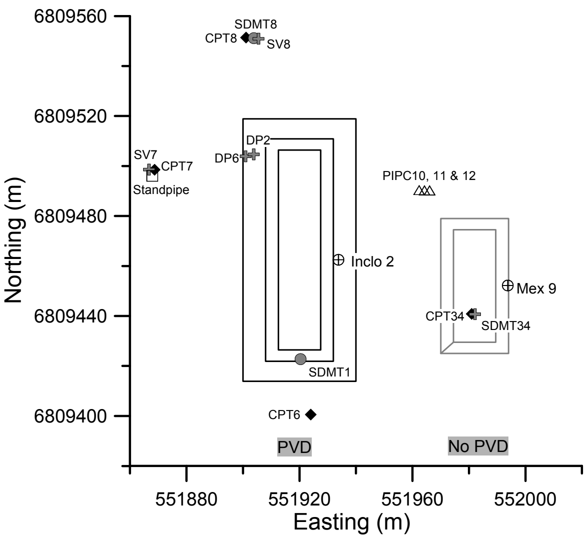

Figure 1. Plan view of the NFTF at Ballina, NSW, Australia.

2 SOFT SOIL CHARACTERISATION STUDY

The subsoil at the NFTF comprises of an approximately 0.2m thick surficial layer of organic material (decomposing sugar cane plants) underlain by a 1.0m to 1.3m thick layer of sandy to clayey silt alluvium deposited during flood events. The soft estuarine clay deposits (Holocene age) under the alluvium mantle infill the Richmond river valley in northern New South Wales, and are about 9m at the site. A transition zone approximately 4m thick lies below the estuarine deposits, turning into a fine sand layer that extends up to a depth of about 18m-19m. The groundwater table level fluctuates between the base of the alluvial silt layer (-1.5m) and 0.5m above ground level, due to standing water after heavy rainfall.

Index properties, as well as mechanical parameters, were obtained from laboratory tests performed on tube specimens retrieved from two continuous boreholes (Inclo 2 and Mex 9, Fig. 1). Samples were retrieved from depths up to 13 m using an Osterberg-type fixed piston sampler (89 mm external diameter). The characterisation study included: (i) the inspection of tube specimens with non-destructive imaging techniques, (ii) index characterisation testing, and (iii) advanced characterisation testing. The basic characterisation tests focused on determining index properties, composition, and fundamental parameters of the natural soil deposits. Advanced characterisation tests were carried out to determine the variation of the hydraulic and mechanical properties along the soil profile. A detailed description of the laboratory characterisation methodology can be found in Pineda *et al.* (2016).

In addition, a comprehensive in situ testing campaign (Kelly *et al.* 2017a) took place in parallel with the laboratory element tests, consisting of geophysics, cone penetrometer, seismic dilatometer, vane shear, piezocone (CPTu) and BAT permeability tests. Selected results from both laboratory and in situ tests are presented and discussed in the following sections.

2.1 *Soil state and composition*

Figure 2 summarises the main index properties obtained from the laboratory tests. Ballina clay has an organic content of around 3% and soil activity equal to 1. Its main mineral components are kaolinite, illite, quartz, illite/smectite and amorphous minerals. The natural water content increases with depth from 20% up to 120%. Differences between the liquid limit and natural water content are less than 10-15%. The plastic limit ranges between 20% and 53%, whereas the liquid limit varies from 55% to 135%. The dry density ρ_d reduces from 1.50 Mg/m^3 to 0.70Mg/m^3 with depth, and the minimum variation in ρ_d occurs between 3 m and 11 m depth. Particle size distributions exhibit some discrepancies between the two boreholes Inclo 2 and Mex 9 at shallow depths (z<2m), mainly in terms of the sand content. The clay content is predominant below 2m, with maximum values of up to 82%, while the sand content lies around 1%.

The marine nature of Ballina clay implies that the influence of the pore fluid salinity is significant and has to be considered during mechanical testing. The electrical conductivity of the pore fluid increases with depth from 5mS/cm ($z\approx$1m) up to 36.5mS/cm at around z=8m (Fig. 2). This corresponds to a solubility of 24gr NaCl /litre of pure water (i.e. 0.69 times the salinity of sea water). The salinity of the pore fluid was controlled during the mechanical tests described below to minimise potential influences on the measured and interpreted properties of the natural clay.

2.2 *Mechanical properties*

Constant rate of strain (CRS) oedometer tests were carried out to evaluate the compressibility of Ballina clay. The one-dimensional stress-strain curves obtained from CRS tests exhibited strong non-linearity, which suggests that the compression index C_c reduces with the stress level. As shown in Fig. 3, the yield stress ratio (YSR) reduces with depth, from around 3 at shallow depths down to 1.5 below 5m. Proper correction of YSR values reported in Fig. 3 for rate effects is required before using them in boundary value problem simulations (Pineda *et al.* 2016). The consolidation coefficient c_v reduces as the stress level approaches the yield stress, σ'_{yield}. Estimates of c_v at σ'_{yield} show that c_v decreases with depth from values larger than 100m^2/year (z<4 m) to an average value of 3m^2/year. Permeability k_w follows the variation of c_v described above. The estimated k_w from the CRS tests ranges between 10^{-8}m/s (z<4 m) towards 10^{-10}m/s below 6m depth. k_w estimated from IL (creep) tests shows a smoother reduction with depth from 2x10^{-9} to 2x10^{-10}m/s. Creep tests also indicate that Ballina clay exhibits secondary compression, with the ratio C_α/C_c varying in the range of 0.03 - 0.05.

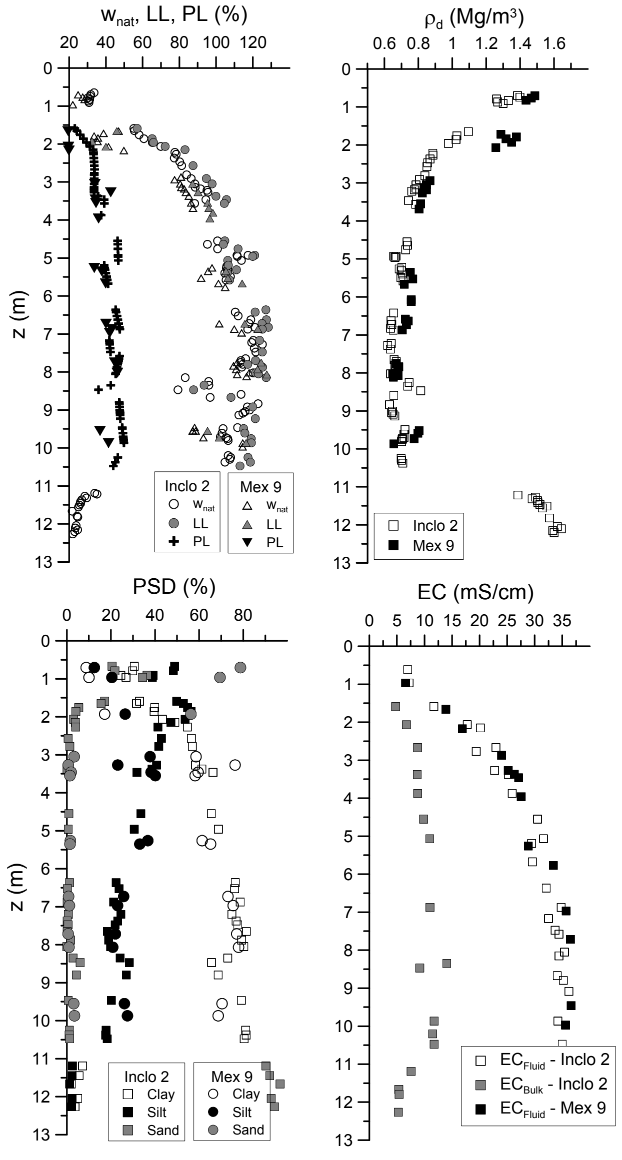

Figure 2. Variation of index properties with depth.

Comparison of c_v and k_w estimated from CRS tests against the results of CPTu dissipation and BAT tests can provide insight to the anisotropy of Ballina clay. The small differences between the horizontal and vertical coefficient of consolidation and permeability values depicted in Fig. 4 suggest low permeability anisotropy. For the soft estuarine clay deposit found between

2.5m and 10.5m depth the coefficient of consolidation at the yield stress from CRS tests c_v lies in the range 2 to 10m^2/year, whereas c_h from CPTu tests varies between 1.5 and 15m^2/year. Figure 4 also depicts good agreement between the horizontal and vertical water permeabilities obtained from in situ BAT and laboratory CRS tests, respectively.

Figure 3 also shows the variation of the undrained shear strength s_u with depth that was obtained from K_0-consolidated undrained triaxial tests (compression and extension). Notice that, as expected, there is a progressive increase in s_u with depth, not only in compression but also under extension shearing conditions. The strength in triaxial compression is about 12.5kPa at 1.8m depth, and increases up to 27.5kPa at a depth of 9.75m. Similarly, undrained strength in triaxial extension increases almost linearly with depth from 10.5kPa to 17.5kPa between 3.5m and 10m. Results from in situ tests (CPTu, SDMT and field vane tests) performed at the locations depicted in Fig. 1 agree well with the laboratory results. Although soil disturbance due to tube sampling perhaps cannot be completely avoided, the results described above show that the use of large-diameter fixed piston samplers, in combination with non-destructive methods to assess and select samples for laboratory testing, produce high quality, reliable tests results.

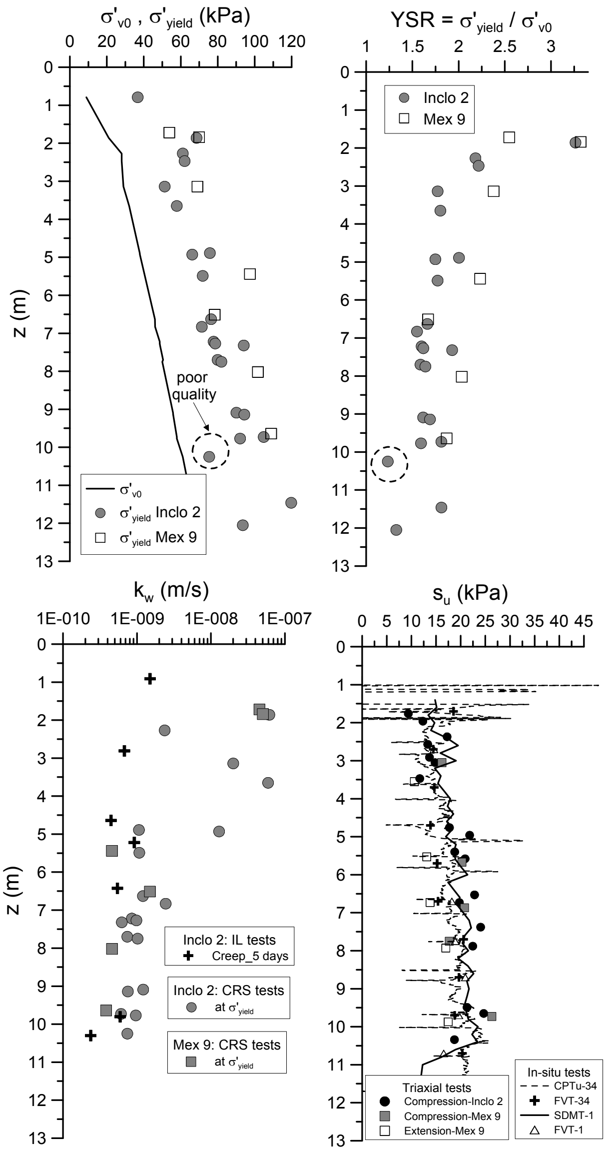

Figure 3. Variation of mechanical parameters with depth.

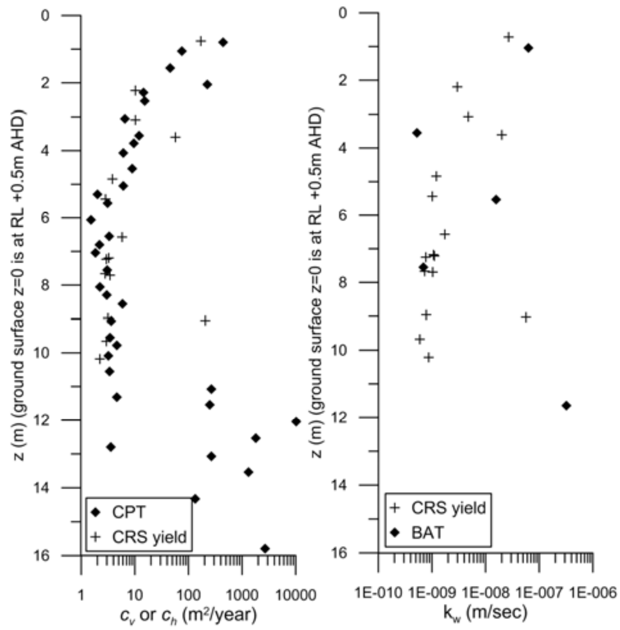

Figure 4. Comparison of field and laboratory consolidation coefficients and permeability.

3 TRIAL EMBANKMENT ON PVD-IMPROVED SOIL

Following retrieval of the soil samples and in situ tests, the construction of the trial PVD embankment commenced in July 2013 and was completed within about 2 months. A typical cross-section of its geometry is presented in Fig. 5. The crest of the embankment is 80m long by 15 m wide and the nominal inclination of the batters is 1.5H:1V. A 95m long by 25m wide working platform was initially constructed, followed by the placement of a sand drainage layer beneath the footprint of the embankment. Layers of separation geofabric were placed above and below the sand drainage layer. The average density of the fill from nuclear density tests was about 2Mg/m^3, therefore the applied load on the ground surface is of the order of 60kPa. Vertical drains were installed on a nominally 1.2m square grid from the top of the working platform, and extended below the transition zone, into the fine sand layer. An extensive instrument network was deployed to monitor the performance of the embankment during and after construction, consisting of vibrating wire piezometers (VWP), total pressure cells, magnetic extensiometers, hydrostatic profile gauges, inclinometers, push-in pressure cells and settlement plates to record settlement of the ground surface below the embankment.

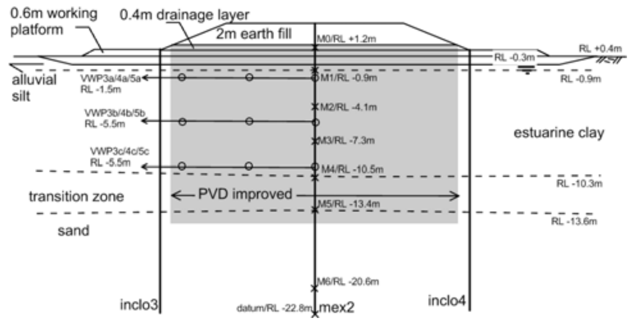

Figure 5. Embankment section and locations of indicative instruments.

3.1 *Embankment monitoring results*

Some measurements indicative of the performance of the trial embankment are presented in Figs 6-8, while the complete monitoring data are provided in Kelly *et al.* (2017b). Even 3 years after construction the rate of settlement of the ground surface has not stabilised, and significant excess pore pressures below the centreline of the embankment have not dissipated (despite the use of wick drains to accelerate consolidation). Data from magnetic extensiometers (not shown here) indicate that deformation of soil layers between magnets M1 and M4 (Fig. 5) is still occurring. These suggest that creep plays a dominant role in the behaviour of Ballina clay, even at relatively low external stress levels (which are sufficient to load the estuarine clay layer beyond yield). The slow rate of dissipation of the excess pore pressures is further delayed by the reduction in the consolidation coefficient and permeability as the stress level increases past the yield stress. Pineda *et al.* (2016) present results of CRS tests which indicated that the permeability of Ballina clay varies with void ratio as $c_k=\Delta e/\Delta \log k_w=1.125$. The ratio of the maximum lateral displacement to the total settlement is of the order of 0.14 (Fig. 8), although it is worth mentioning that the rate of development of lateral deformations is rather high.

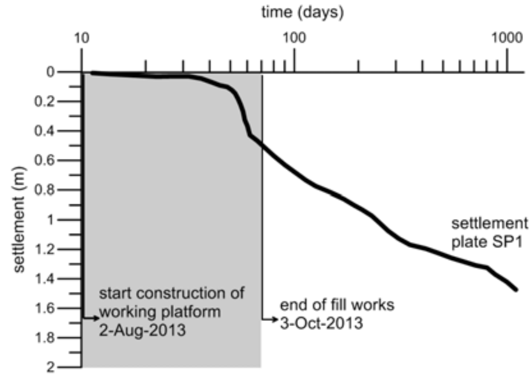

Figure 6. Ground surface settlement at settlement plate 1.

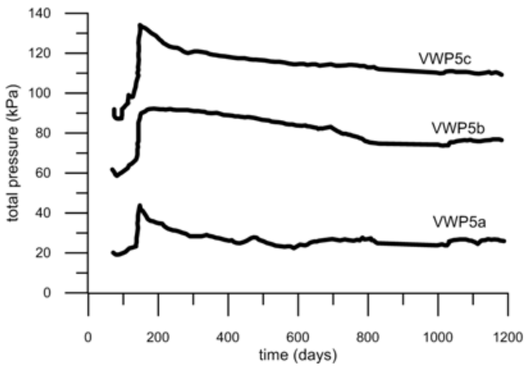

Figure 7. Pore pressures below the embankment centreline.

It is worth mentioning that groundwater levels were not constant during the 3-year period covered by the measurements, and the site was inundated with rainwater on several occasions. It is assumed that groundwater levels increased, compared to the level during construction shown in Fig. 5, although no standpipe piezometers were installed to confirm this. Analysis of total pressure cell measurements suggest a reduction in the total pressure below the embankment of the order of 15%, attributed mostly to settlement of the fill below the groundwater table and buoyancy effects. However, the uncertainties related to this are quite high, and *a priori* consideration of such phenomena is unquestionably challenging.

4 CONCLUDING REMARKS

Results of the laboratory and in situ tests, together with details about the embankment geometry and construction history discussed briefly above, were provided to academics and practitioners interested in attempting to predict the response of the PVD-improved embankment. Comparisons of their blind predictions with actual measurements exemplify the importance of high-quality sampling and in situ/laboratory testing for obtaining reliable parameters to calibrate the rate-dependent constitutive models required to capture the creep behaviour of soft, structured clays. The use of simplistic models which do not account for viscous phenomena, such as Modified Cam Clay, resulted in underprediction of the settlement magnitude, and overprediction of the rate of dissipation of excess pore pressures. Uncertainties in quantifying the role of the prefabricated vertical drains in accelerating the rate of pore pressure dissipation remain, despite extensive research in the field. Estimating the extent of the smear zone, and of its reduced permeability, in natural clays is subject to a number of unknowns, and its proper integration with standard 2D plane strain analysis models remains challenging. A detailed discussion on the above is presented in Kelly *et al.* (2017c).

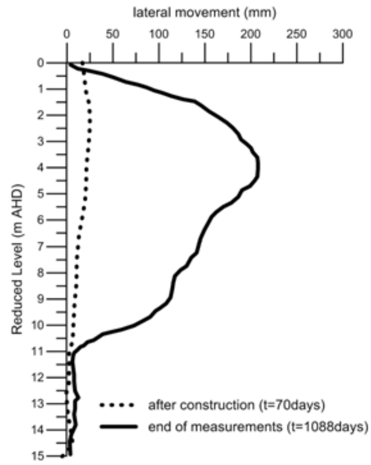

Figure 8. Lateral displacement at the toe of the embankment.

5 ACKNOWLEDGEMENTS

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