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Preloading of deep soft foundation for an Administration Building of Container Terminal at Melbourne

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ABSTRACT: An administration building was proposed as part of the Victoria International Container Terminal Project at Melbourne. The site was underlain by a highly compressible clay soil. To satisfy the design requirements, ground improvement was undertaken by preloading the site with 3 m of compacted fill surcharge with assistance of wick drains. The installed instrumentation consisted of settlement plates and vibrating wire piezometers, which allowed the target settlement to be confirmed prior to removal of the surcharge. The results of the surcharging monitoring were used to calibrate the consolidation analysis using PLAXIS 2D, leading to an increased confidence in the settlement predictions. Finally, the paper discusses the consolidation characteristics of the wick-drained preloading adopted for this project in comparison with a pre-loading without wick drains undertaken by others for a nearby site.

1 INTRODUCTION

URS Australia Pty Ltd (now part of AECOM) was engaged by Victoria International Container Terminal Ltd (VICT) to provide Engineering Services for the construction of the Victoria International Container Terminal at Webb Dock East, Port Melbourne. As part of Melbourne's $1.6 billion Port Capacity Project, the development would deliver fully-auto-mated container handling operations from the gate to the quayside to handle up to 1.4 million twenty-foot equivalent container units (TEU) annually. The development consists of a container terminal area (TA), gate control area (GCA) and empty container park (ECP). Administration Building, which provides management and control of the entire container handling operations, is located at GCA of the off-dock area, Webb Dock Drive at Port Melbourne (Figure 1). Also shown on Figure 1 is the location of a trial embankment TE1 (Golder, 2013) some 200m north east of Administration building. The trial embankment formed part of study for the development (Empty Container Park) undertaken by Port of Melbourne Corporation (PoMC). The data of the study formed part of reference data for this project.

Part of AECOM's duties were to provide the foundation design and ground improvement engineering services. The ground improvement techniques that were adopted for the project depended on the site reclamation history, compressible soil layer thickness and the operational requirements imposed by the development. Adopted techniques included a combination of the following:

- Controlled modulus columns (CMCs) for part of TA area which is located in a reclaimed area south of GCA.
- Preloading with 3 to 5 m high surcharge fill for both TA and GCA areas.
- Impact compaction. This method focused on improving shallow fill and was used in all three development areas.

Ground improvement for TA of Webb Dock development was covered in another paper (Chen, 2016). This paper focuses on the preloading design and associated instrumentation and monitoring for the Ad-ministration Building at GCA.

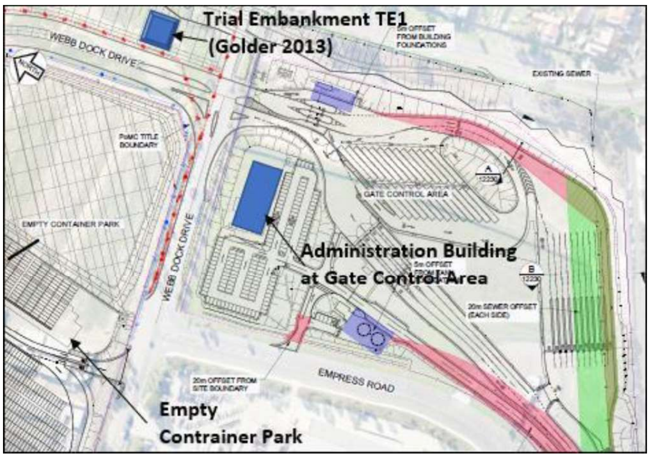

Figure 1 - Locality plan of Administration Building

2 GROUND CONDITIONS

Webb Dock is located at the head of Port Phillip Bay immediately east of the mouth of Yarra River and within the area broadly described as the Yarra Delta. The sub-surface conditions in this area comprise recent marine alluvial deposits (Port Melbourne Sand and Coode Island Silt) underlain by Tertiary marine sedimentary deposits with Melbourne Mudstone forming the basement rock at depth. While various episodes of basalt flows are reported within the sequence (Quaternary and Tertiary aged flows) (Neilson 1992), no basalt flows were found within the project area. The expected subsurface conditions based on the investigation and available geotechnical information in the adjacent area comprised a layer of uncontrolled fill to a depth of about 2.1 m, over Port Melbourne Sand (PMS) to a depth of about 9.6 m, over Coode Island Silt (CIS) to a depth of about 30.4 m, over Moray Street Gravel (MSG), as illustrated in Figure 2. The stratigraphy of the subsurface conditions underlying the proposed building are summarised in Table 1.

Table 1 Subsurface conditions for Administration Building

Unit	Depth to top of unit (m)	Thickness (m)	Consistency
Fill	0	2	various.
PMS	2.0	7.6	loose to medium
CIS	9.6	20.8	soft to stiff
MSG	30.4	-	medium to dense

The consistency of CIS ranges from soft to firm (the cone resistance q_c of 0.5 to 1.0 MPa). Towards its base CIS became firm to stiff (q_c = 0.8 to 1.2 MPa) prior to the transition to MSG. The q_c values are higher than typically expected near surface in CIS (in the range 0.2 to 0.5 MPa) due to the high confining pressure imposed by the overlaying PMS. The strength, stiffness and consolidation parameters of CIS are presented in Table 2 based on the laboratory testing undertaken for the site investigation (Chen et al 2016).

Table 2 – Strength and Consolidation parameters of CIS

Void Ratio	c' (kPa)	ϕ' (°)	C_c	C_s	C_α	k_v (m/d)	PoP (kPa)
2.04	1	25	0.72	0.1	0.02	10^{-5}	10-30

Where c' = effective cohesion, ϕ' = effective internal friction angle, C_c = compression index, C_s = swell index, C_α = secondary compression index, C_v = Coefficient of consolidation, PoP = p'_c-σ'_{zo}, p'_c = pre-consolidation pressure, σ'_{zo} = effective overburden stress.

The consolidation status of CIS was assessed by the comparison with its pre-consolidation pressure (p_c') against the current overburden stress σ'_{zo}. The $p_{c'}$ value has been estimated primarily from the Oedometer tests (Chen et al 2016) to be about 10 to 30 kPa greater than the σ'_{zo}. Value (i.e. very slightly over-con-solidated). The assessed value is consistent with the inferred value from CPT (the offset of 30 kPa at ground level from the cone resistance (qt) versus depth plot shown in Figure 2). The above assessment is also consistent with reports by others (Ervin, 1992 and Srithar, 2010).

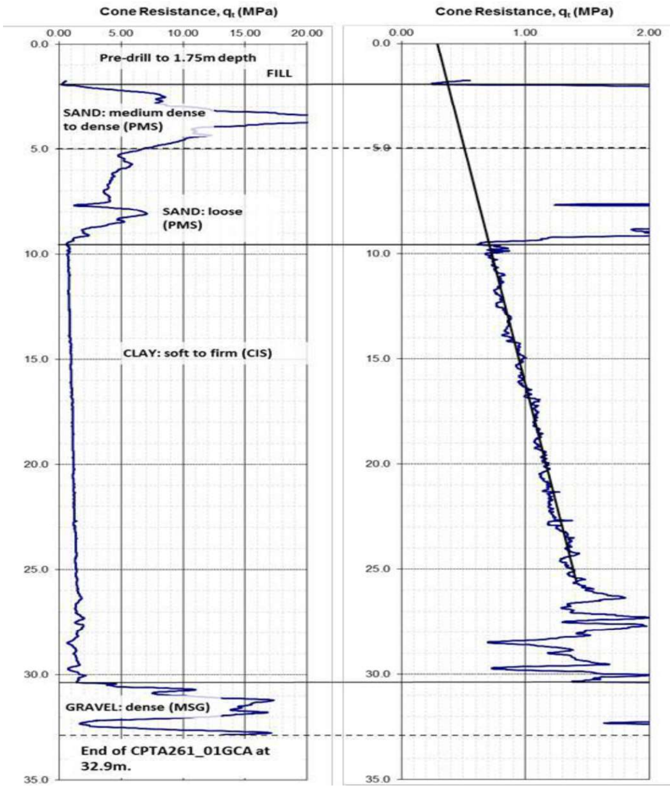

Figure 2 - Inferred *s*ubsurface *c*onditions from CPT

3 PROPOSED ADMINISTRATION BUILDING

As shown in Figure 1, the proposed building is located at the entrance to GCA. It was proposed to be a two-level structure supported on slab-on-ground footings occupying an area of 50m by 20m. It was a requirement for site drainage purposes to raise the site levels by approximately 1.4m bulk earth filling to form a building platform at RL3.5m. Total load imposed at the original ground level is about 50 kPa comprising about 30 kPa from the additional 1.4 m filling and 20 kPa from the building load (dead and live load).

The presence of CIS and its great thickness imposed special challenges for design and construction of the administration building. It was estimated that without ground improvement, the combined 1.4m filling and building dead load (10 kPa) could result in potential settlement of up to 200 mm (> 50mm differential) over the design life of the project. The estimated settlement was in excess of the building's limit of 40 mm differential settlement. Of the total settlement of 200mm, more than 2/3 contribution was estimated from the 1.4 m bulk filling and this settlement would extend beyond the footprint of the building (affected adjacent pavements and other structures). A piled solution to MSG for the building would reduce the building settlement; however, would result in differential settlement to the surrounding area leading to "mushrooming" effect, which was particularly problematic for the service connections to the building.

It was decided that the building be founded on slab on ground, effectively "floating" the building on an engineered fill platform and design a shallow footing system founded within the engineered fill. A key recommendation for the building design was for a light weight and flexible structure. The purpose of this was to reduce the load imposed by the building and likewise to make the building more tolerable to the residual differential settlement. To implement this recommendation, the potential total settlement of 200 mm had to be reduced by preloading down to 100 mm prior to the building construction so that the residual settlements post the construction was acceptable for the building structure.

4 GROUND IMPROVEMENT

Preloading is one of the most effective, economic and practical ground improvement methods that have been adopted in civil engineering worldwide. Depending on time availability and the target settlement to be removed, the preloading process can be accelerated by increasing the surcharge load or vacuuming, shortening the drainage paths using prefabricated vertical drains (Bergado et al 2002 and Indraratna 2009), or combinations of the above (Day 2007).

Prior to preloading, the entire GCA area was dynamically compacted using impact roller to improve the existing uncontrolled fill and remove potential differential settlement from this unit. Validation testing confirmed that between 30 to 80 mm settlement was removed by dynamic compaction within the entire GCA area.

Based on the analysis presented in Section Settlement analysis, a preloading with surcharge of 3m high compacted earth fill for a period of 4 to 6 months would result in more than 50% reduction in potential settlements, i.e., the potential settlement would be reduced from 200 mm to below 100 mm. However due to the constraint of the project program, it was decided to install wick drains prior to preloading to expediate the consolidation and to reduce the preloading period to approximately 2 months. Key aspects of the site preparation and preloading included:

- Improvement of the existing fill at RL 2.1 m by impact rolling
- Installation of wick drains within the Coode Island Silt to a depth of 25 m (RL -23m)
- Raising the site levels with well-compacted earth fill to a level approximately 1.4m above the existing ground level (RL 3.5m)
- Placing 3 m surcharge fill to RL 6.5 for about 2 months to remove a target settlement of 100mm prior to the construction of the building.
- Settlement and pore pressure monitoring.

See the following sections for more details of surcharge, wick drains arrangement and monitoring.

5 DETAILS OF WICK DRAINS AND INSTRUMENTATIONS

Figure 3 and Figure 4 show the layout of the proposed administration building and the extent of wick drains and instrumentation details. To minimise the differential settlement near the edge of the building both surcharge and wick drains extended 5 m beyond the building footprint.

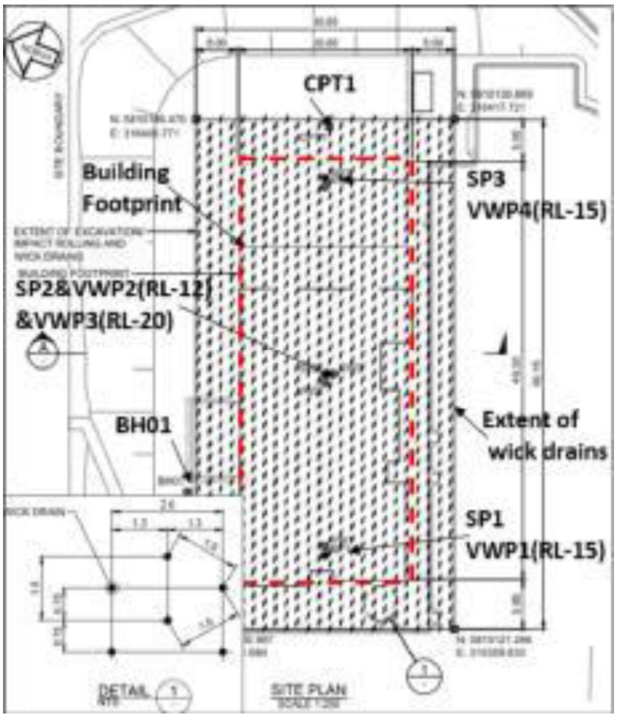

Figure 3 - Plan layout of surcharge and wick drains

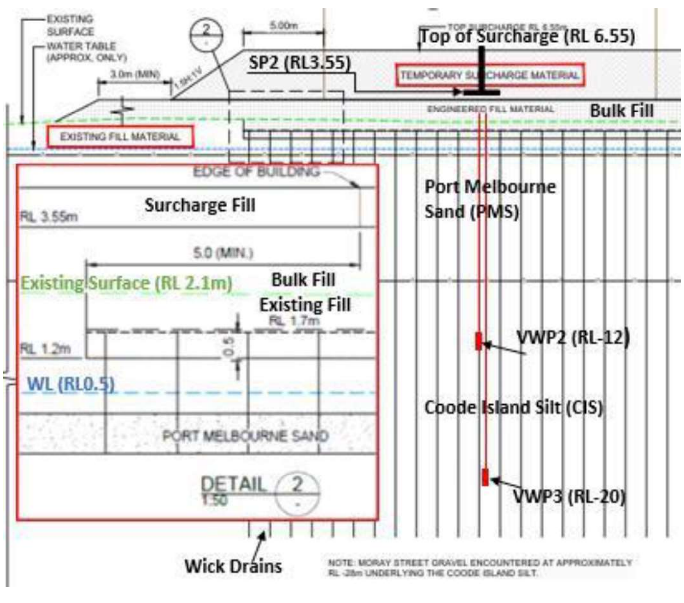

Figure 4- Section layout of surcharge and wick drains

Wick drains comprised CeTeau – 100mm wide prefabricated vertical drain (PVD). As shown on Figure 3 the wick drains are spaced at 1.5m in triangular patterns and installed to about 25 m below the existing surface. The installation of wick drains was undertaken from the existing ground level using a proprietary hydraulic driven rig as shown in Figure 5.

Figure 5 – Wick drain installation

The instrumentation consisted of three Settlement Plates (SP1 to SP3) and four Vibrating Wire Piezometers (VWP1 to VWP4) which allowed the settlement of surcharge and pore water pressures in the ground beneath the surcharge to be monitored. All settlement plates (SP1 to SP3) were installed at the bulk earth ground level (RL 3.5) prior to the placement of the surcharge. VWPs were installed at 3 locations (VWP1 & VWP4 are at the same plan locations as SP1 & SP3 respectively and VWP2 & VWP3 at the SP2 plan location) with the tip of VWPs targeting CIS at the depths ranging between 14m and 22 m. The VWPs were installed at the base of boreholes drilled to the nominated depths. Once the VWPs were in-stalled, the borehole void surrounding the piezometer tip was packed with washed fine sand to a height of between 0.7 m to 1.4 m with overlying bentonite pellet plug of between 1.2 m to 2.0 m thick. The remaining borehole length was then backfilled with cement-bentonite grout to the ground surface with tremie grouting techniques. The cables from VWPs were embedded 0.5m in a trench directed to central locations north of the surcharge for monitoring.

The installed instrumentation allowed the target settlement and the degree of consolidation to be confirmed prior to removal of the surcharge. The results of the surcharging monitoring also allowed the consolidation analysis using PLAXIS 2D to be calibrated against the measured settlements and pore pressures, leading to an increased confidence in the settlement predictions.

6 SETTLEMENT ANALYSIS

The PLAXIS model including the proposed bulk filling and surcharge geometry and soil stratigraphy is shown in Figure 6. The deformation analysis for the preloading ground improvement was undertaken using PLAXIS 2D with the CIS modelled as Soft Soil Creep model with the material parameters used for the analysis presented in Table 2. All the inputs for the analysis were directly taken from the average values of Oedometer tests (Chen et al, 2016) apart from $C\alpha$ which was taken 3% of C_c (Mesri, 1987). The parameters for other soil units are not important for the preloading design and these are not presented in this paper. The wick drains were modelled as “drain” elements with a constant head at the groundwater level. In the 2D model the drain spacing was assumed to be 2.8m which is converted from the 3D 1.5m by 1.5m triangular drain patterns with consideration of the smear zone and well resistance effects using a similar approach by Hansbo (1981).

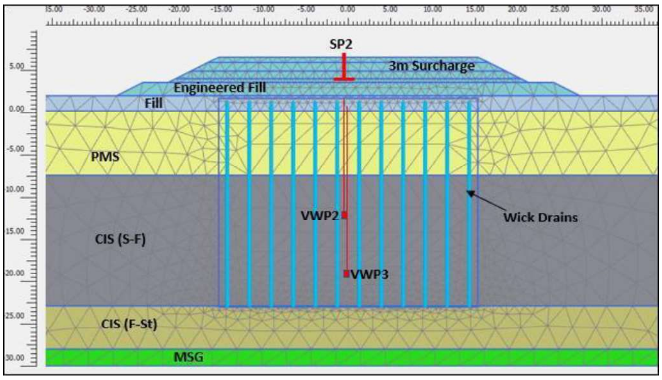

Figure 6 - PLAXIS model

Estimated settlements are summarized in Table 3. The analysis indicated that with adoption of 3m surcharge both the preloading with and without wick drains can satisfy the building’s limits of 100mm for total and 40mm for differential settlement, although the pre-loading assisted with wick drain can achieve a better result within a shorter period of time.

Table 3 – Estimated long term settlements for admin. Building

Wick drain	Surcharge		Settlement (mm)	
	Height (m)	Period(months)	Total	Differential
No	0	0	200	50 to 70
No	3	4	90	<40
Yes	3	2	<50	<30

7 SETTLEMENT AND PORE PRESSURE MONITORING

The surcharge was built to 3m high and left at full height for 54 days. Monitoring of settlements and pore pressures were undertaken during the pre-load-ing stage. The ground consolidation at each of settle-ment plates by preloading is summarised in Table 4.

Monitored data confirmed that more than 100 mm of the target long term settlement was removed.

Table 4 . Removed settlements at SP1, SP2 and SP3

Monitoring point	SP1	SP2	SP3
Ground consolidation (mm)	185	223	174

Figure 7a) presents the surcharge loading sequence and measured settlement at the center of surcharge (SP2). Figure 7b) presents the measured pore pressures at VWP2 and VWP3 (at depth of RL-12 and RL-20 below the center of surcharge.

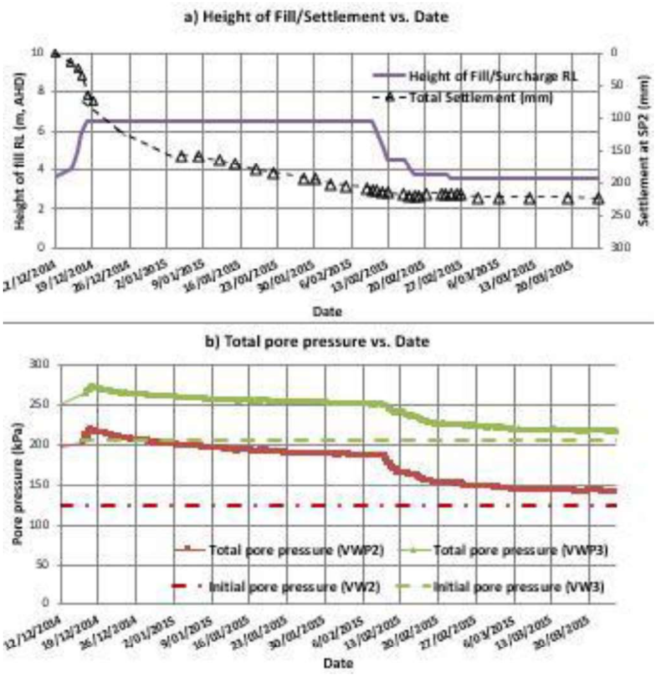

Figure 7 - Measured settlement and pore pressures

The pore pressures monitored in the CIS by VWP2 and VWP3, depicted in Figure 7b, show the progressive increase in pressure from the hydrostatic pressure during the surcharge loading period. The decline in pore pressure can be seen after the steep jumps associated with the rapid placement of surcharge material.

Following placement of the surcharge there was generally a steady decline in pore water pressure before unloading was commenced. On removal of the surcharge, the pore water pressure dropped back towards the hydrostatic pressure of 125 kPa at VWP2 and of 205 kPa at VWP3 (initial pore pressure relevant to the ground level of RL 2.1m prior to any filling).

Measured excess pore pressures (difference between the measured total pore pressure and the initial pore pressure) for all VWPs are presented in Figure 8. For the two measurements at the same depth (VWP1 and VWP4), the initial response is quite different: the VWP1 had reached an excess pore pressure of 35 kPa (only about 37% of the applied load) whereas the VWP4 had up to 60 kPa (over 60% of the load applied). Despite the difference both VWPs had converged towards the same value of 34 kPa (about 35% of the applied load (or 65% of full consolidation).

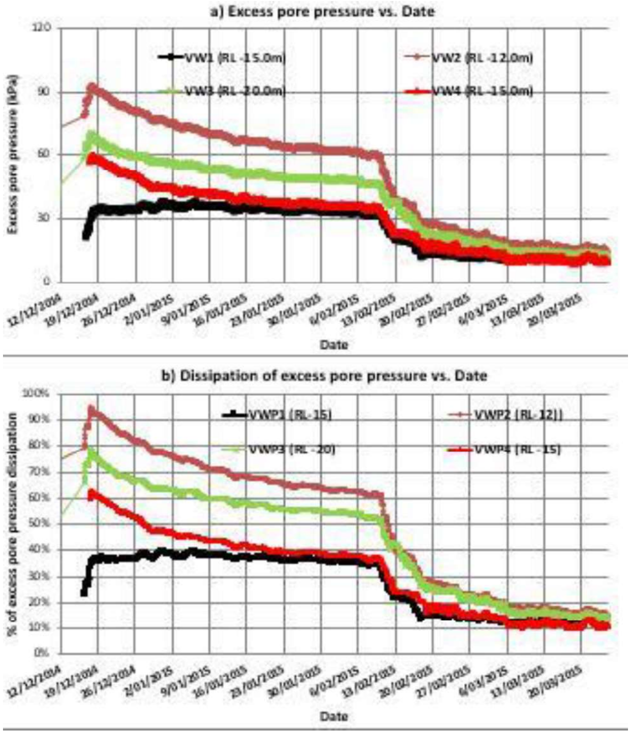

Figure 8 - Measured excess pore pressure

Table 5 summarises the pore pressure measurements. As expected, the highest excess pore pressure response occurred at VWP2 (93 kPa = 95% of the applied pressure) – the shallower piezometer located within softer CIS where the imposed stress is the greatest.

Table 5 - Details of pore pressure measurement

ID of VWP	VWP1	VWP2	VWP3	VWP4
Depth in RL	-15	-12	-20	-15
u_o (kPa)	155	125	205	155
$\Delta\sigma'_z$ (kPa)	95	98	89	95
u_{max} (kPa)	190	218	275	215
Δu_{max} (kPa)	35	93	70	60
$\Delta u_{max} / \Delta\sigma'_z$ (%)	37	95	79	63

Where u_o – initial pore pressure prior to filling and surcharge; $\Delta\sigma'_z$-maximum overburden stress applied due to filling and sur-charge; u_{max} - maximum measured pore pressure; Δu_{max} maxi-mum measured excess pressure.

Overall the degree of consolidation completed at the end of the 54 day surcharge period ranged between 34% and 61% (average of 46%) relative to the surcharge load. Inferred from the pore pressure measurement, the average settlement removed as a result of preloading of 194 mm would count for approximately 46% of the potential total settlement that would have occurred should the preload have been a permanent load. On the other hand, the 194mm of settlement removed by preloading would count for about 97% of the 200mm settlement predicted to occur without preloading. Therefore preloading with a 3 m surcharge for 54 days had removed 97% of the estimated long term settlement under the combined bulk filling and building load.

8 DISCUSSIONS

Following conclusion of the preloading, the PLAXIS model used to predict settlement was re-run replicating the as-built surcharging process. The result of the analysis for settlement at the middle of surcharge is presented in Figure *9*, together with the settlement measured at the same locations (SP2). At this location it can be seen that a very close match between the calculated vertical displacement and monitored settlements at the middle of surcharge was achieved without adjustment to the soil parameters used in the initial modelling prior to the actual preloading.

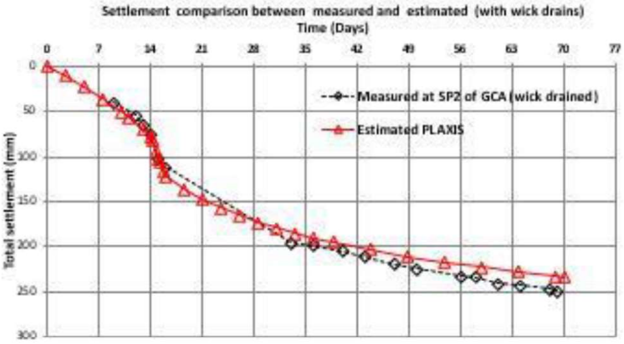

Figure 9-Settlement comparison: predicted vs measured

Figure 10 presents a comparison of the settlements measured on the GCA site where wick drains were installed with a nearby site where no wick drains were installed. Both sites have similar ground conditions and had been preloaded with 3m surcharge which was fully loaded over 14 days. It can be seen that when the full surcharge was placed by Day 14, a total settlement of about 100 mm occurred for both sites. However by Day 70, the settlement at GCA site was about 250 mm which was more than twice the settlement of 118mm at TE1 site. By projection of the TE1 settlement vs time curve, it would need an additional 530 days to achieve the same settlement of 250 mm as GCA site. This clearly shows that the installation of wick drains at 1.5m spacing of triangular grids had effectively expediated the consolidation process and resulted in significantly saving in preloading time.

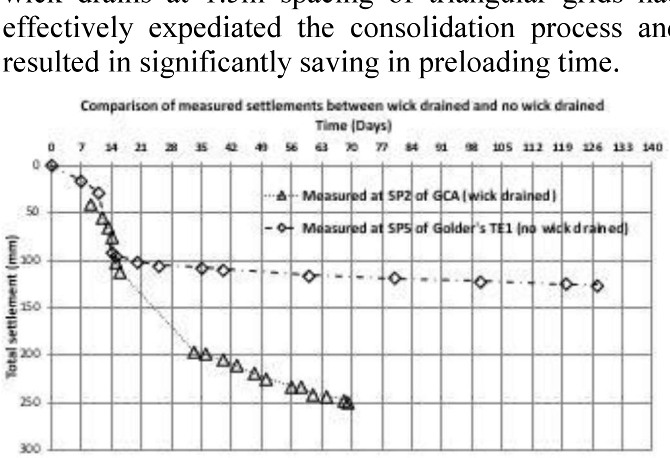

Figure 10-Settlement comparison: wick drained vs. no wick drained

9 CONCLUSIONS

Ground improvement using wick drained preload was undertaken for the administration building under-lain by soft marine clay. The monitored settlement and excess pore pressure demonstrated that approximately 46% of the primary consolidation of a relatively thick 20m layer of CIS could be achieved from a preloading of less than 2 months with assistance of wick drains. Of note, it was found that laboratory de-rived consolidation parameters, particularly compression index (C_c) and coefficient of vertical permeability (k_v), provided a good match to observed behaviour of the full scale preloading. Compared with the data obtained from a nearby site (preloading without wick drains), it was found that the preloading assisted by wick drains had removed the settlement that would have otherwise taken about 1.5 year to complete for the same surcharge without wick drains.

10 ACKNOWLEDGEMENTS

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REFERENCES

Bergado, D. T., Balasubramaniam, A. S., Fannin, R. J. & Holtz, R. D. Prefabricated vertical drain (PVD) in soft Bangkok clay: a case of NBIA project. *Canadian Geotechnical Journal, 2002. 39(2): pp. 304-315.*

Chen, K. & Roberts, S. 2016. Preloading Ground Improvement for an International Container Terminal Project at Webb Dock Port Melbourne. *2016 AGS Symposium for Ground Stabilisation Techniques for Problematic Soils and Unstable Rock*, *11 November 2016*, Sydney.

Day, R. & Woods P. 2007. Verification of consolidation parameters of a near-normally consolidate clay by back-analysis of an instrumented, wick-drained reclamation. *Proceeding of 10th Australia New Zealand conference on geomechanics*, Brisbane, Australia

Ervin, M.C, *1992* Engineering properties of Quaternary age sediments of the Yarra Delta. *Engineering geology of Melbourne, Peck, Neilson, Olds & Seddon (eds)*

Golder Associates, 2013. Webb Dock Trial Embankments – Assessment of Settlements, *Letter to Port of Melbourne Corporation, 11 November 2013*

Hansbo, S, 1981 Consolidation of fine grained soils by prefabricated drains, *Proceedings 10th ICSMFE, Stockholm, Vol. 3 pp 677-682*

Indraratna, B. 2009. Recent Advances in the Application of Vertical Drain and Vacuum Preloading in Soft Soil Stabilisation. *2009 EH Davis Memorial Lecture – Australian Geomechanics Society*

Mesri, G and Castro, A. 1987. The / C_c/C_α concept and Ko during secondary compression, *Journal of Geotechnical Engineering, ASCE, 113(3): 230-247*

Srithar, S. 2010. Settlement Characteristics of Coode Island Silt. *Australian Geomechanics Vol 45 No1, March 2010, pp55-64*