PRELOADING GROUND IMPROVEMENT FOR AN INTERNATIONAL CONTAINER TERMINAL PROJECT AT WEBB DOCK PORT MELBOURNE

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ABSTRACT

In this paper, a case study is presented for preloading ground improvement for an automated stacking container development at Port Melbourne, Victoria, a reclaimed site underlain by high compressible marine clays referred to locally as Coode Island Silt (CIS). The paper has firstly reviewed the site reclamation history, followed by consolidation back analysis using PLAXIS 2D to study the historical settlements associated with the previous reclamation and land use and their effect on the future development with and without the adoption of preloading. A decision was then made to adopt a preloading program that involved the application of up to 5.5 m high compacted earth fill (110 kPa which is equivalent to more than twice of the maximum design load for the containers) for a period of 2 to 3 months, targeting to remove over 40% of the potential total settlement under the container stacking loads. The paper also discusses some of the instrumentation, such as settlement plates and vibrating wire piezometers that was employed to verify the target settlement. The study has concluded that the design intention has been achieved by the completed preloading ground improvement program.

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-automated container handling operations from the gate to the quayside. Once fully developed, the 35.4-hectare terminal will be able to handle up to 1.4 million twenty-foot equivalent container units (TEU) annually.

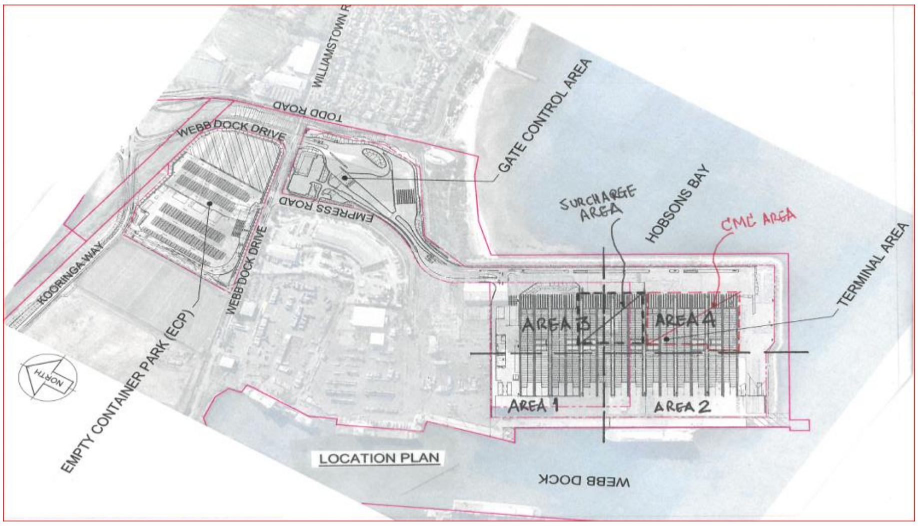


Figure 1: General area of Webb Dock International Container Terminal Development

The development consists of a container terminal area (TA), gate control area (GCA) and empty container park (ECP) (Figure 1), of which the TA is located at Webb Dock East, a site previously reclaimed from Hobsons Bay, part of Port Phillip. Both the GCA and ECP are situated in the off dock area with the most part of the ECP being located over an old sand quarry that once provided the source of fill for Webb Dock's early development. The quarry was subsequently filled with landfill waste before being capped with clay fill to current ground level.

The TA development, measuring approximately 700m by 500m, is located to the east of the existing dock, developed in four areas, namely Areas 1 to 4, by land reclamation between the 1960s and late 1990s (refer to Figure 1 for the division of these areas and Section 2 for details of the reclamation history). Existing fill depths typically vary between 6 and 8m but are up to 14 m at the south-eastern portion of the dock. The fill is underlain by Quaternary age marine and alluvial deposits of the Yarra Delta. The recent marine deposits, referred to locally as Coode Island Silt (CIS), is a highly compressible (with an initial void ratio of e_0 = 1.2 to 2.8) and very low strength (with an undrained shear strength of c_u= 5 to 30 kPa) clay soil. The thickness of CIS varies from 1 to 2 m at the west end of the development up to 5 to 7 m in the eastern portion of the site. The presence of CIS and its variable thickness imposes special challenges for design and construction of the development.

The key assets of the proposed development include automated stacking cranes (ASCs) and the plinth foundations for stacking containers. The most important design considerations for the automated container handling system included the rail foundations for the ASCs and the plinth foundations for stacking containers. The operational tolerance of ASCs imposes tight restrictions on the vertical and lateral displacements of rail foundations. The plinth foundations required to support the loads of containers stacked 5 high also have limits on the settlements induced by the loads to meet the ASC operational tolerance. These restrictions are due to the automated nature of the ASC operation.

One of the key components of AECOM's engineering services was ground improvement. 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 areas underlain by more than 5 m compressible CIS and with estimated long term settlement ≥ 350 mm;
- Preloading with 3 to 5 m high surcharge fill for areas underlain by up to 5 m compressible CIS with estimated long term settlement between 150 and 350 mm; and
- Impact compaction for areas with estimated long term settlement <150 mm (primarily to compact the upper fill materials).

This paper focuses on the preloading design and associated instrumentation and monitoring for part of Area 4, where the CIS was relatively thin and uniform.

2 RECLAMATION AND SETTLEMENT HISTORY

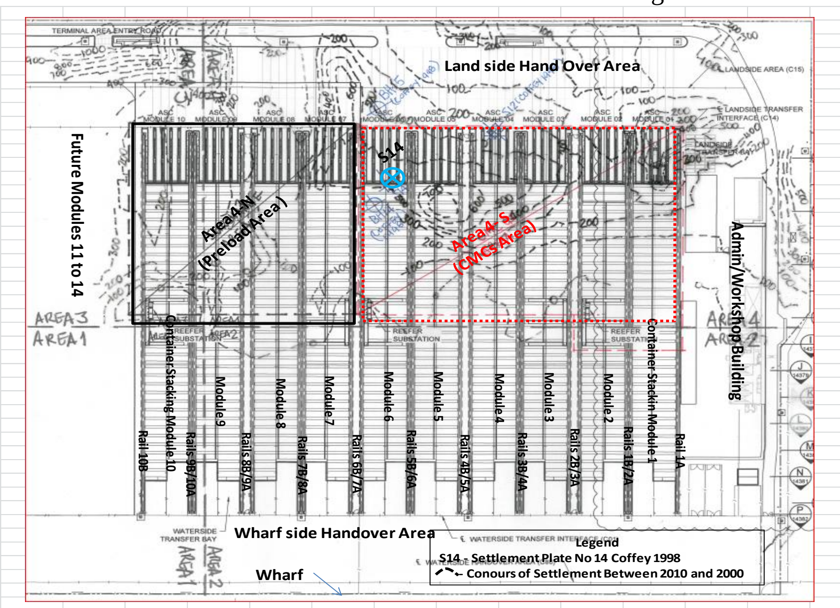
Figure 1 shows the four areas (Area 1 to Area 4) progressively developed by land reclamation between the 1960s and late 1990s. This information was sourced by reviewing historical aerial photos and surveys that were included in the tender documents from the Port of Melbourne Corporation (PoMC).

Area 1, 2 and 3 were reclaimed in the 1960s and 1970s whereas the Area 4 reclamation was undertaken over a relatively short period between late 1997 and March 1998. Previous uses of the development varies includes a car storage facility in Area 3 and Area 4 and a container terminal in Area 1 and Area 2.

The typical reclamation process for each of the areas commenced with constructing an outer bund wall using rock fill followed by internal filling using gravel, sand, silt and clay. The outer bund rock fill is understood to have been placed by the end dumping technique. The internal filling was generally undertaken using the hydraulic filling technique for Area 1 to Area 3 and the northern half of Area 4 (Area 4-N). Both hydraulic and end dumping techniques had been adopted in the reclamation works for the southern half of Area 4 (Area 4-S).

Hydraulic fill is normally placed gradually over the natural soils and causes minimal disturbance to the underlying low strength CIS. This forms a relatively uniform profile, albeit often in a very loose and under-consolidated state. On the other hand, the end dumping technique can load the weak ground quickly, leading to progressive failures within the underlying CIS and creating "mud waves" ahead of the fill, resulting in a fill comprising a mixture of imported material and in-situ soils or the formation of inclusions of weak clay zones within the fill. The worst scenario created by the displacement of CIS is where an area of CIS has been fully displaced by rock fill leaving hardly any highly compressible material below the less compressible rock fill immediately adjacent to an area where the mud waved CIS has been lifted higher than the in-situ CIS, creating a thick layer of highly compressible disturbed material. Differential settlement between these two areas could be very large and additionally a foundation strength design issue is created where the very low strength mud waved CIS is close to the underside of the footings.

Particular reference is made to the filling works in Area 4-S (see **Figure 2**



), where the reclamation was undertaken by the end dumping technique. Large mud waves were created by the end dumping of spoil won from excavation spoil comprising siltstone of varying degrees of weathering. A series of cells were created by constructing end dumped berms with the intention of minimising displacement of the weak in-situ soils. The investigation and assessment of this mud waved area was undertaken by Coffey Geosciences (1998 and 1999). The investigation indicated that the end dumping filling technique had created an extremely non-uniform and highly variable thickness of fill, both horizontally and vertically. Based on the boreholes drilled post reclamation, some rock fill was encountered at a depth of 3 to 5m below the original sea bed level while CIS was found 3 to 5m above the original sea bed level. The variability of the new fill formation was also demonstrated by a large range of observed settlements (ranging between <100mm to over 1000mm) monitored post reclamation between March 1998 and June 2000 (see Table **1**).

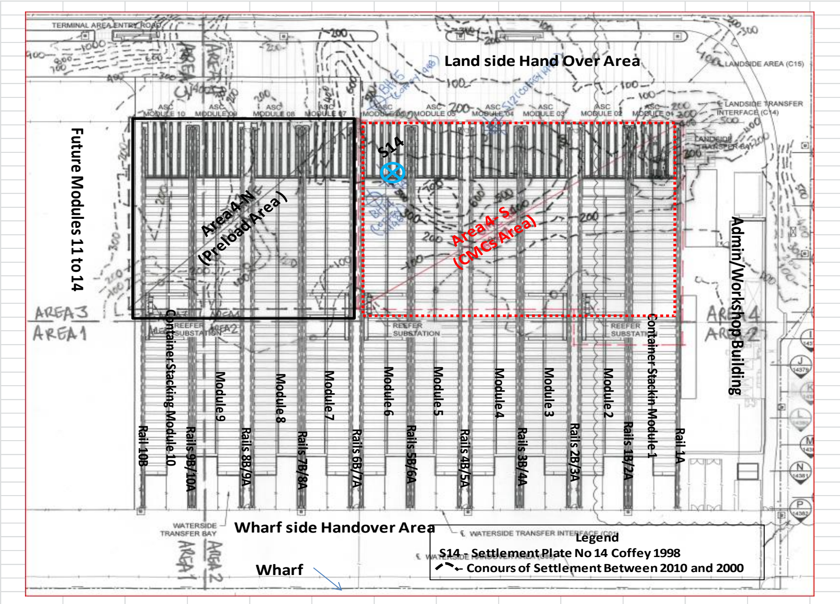
Table 1: Measured settlements in Area 4

Date	Area 4-S Settlement (mm)	Area 4-N Settlement (mm)	Note
Prior to March1998	unknown	unknown	No measurement *
March to July1998	56 to 458	33 to 116	Measured*
March1998 to June2000	102 to 690	130 to 409	Measured*
June2000 to May2010	100-700	100 to 500	Inferred from survey data
May2010 to May2015	70	50	Inferred from survey data
Sum	328 to 1918	313 to1075	

*settlement plates were installed post reclamation and therefore the measured settlement did not include the portion of settlement that occurred during the filling

The 1990s reclamation area (Area 4) was developed into an open car park in early 2000 with structural pavement fill placed over the reclaimed fill. Evidence of large settlements that occurred post pavement placement can be inferred from a series of aerial photos where several large ponds of water developed into a permanent feature in early 2000s.

A study of post development settlement at Area 4 was undertaken as part of the current site investigation program. The study included observation of the current pavement levels along with comparison of the site feature survey undertaken at different dates, including the May 2010 feature survey undertaken by PoMC and the 2014 and 2015 feature surveys undertaken by the construction contractor for VICT. The difference in pavement levels between the 2010 survey and the 2000 data identified several hot spots where settlement of over 500 mm occurred during this 10 year period (settlement contours are shown as dotted lines on **Figure 2**



). The hot spots were identified in the following areas:

- Area east of proposed container stack Modules 9 and 10 where the maximum settlement was over 1000 mm;
- Area at the eastern end of proposed container stack Modules 4 and 5 where the maximum settlement was over 600 mm;
- Area at the proposed Module 1 eastern landside handover bay where the maximum settlement was more than 500 mm.

Differential settlements in these hot spots are very significant, in the order of 500mm over a distance of 30 m.

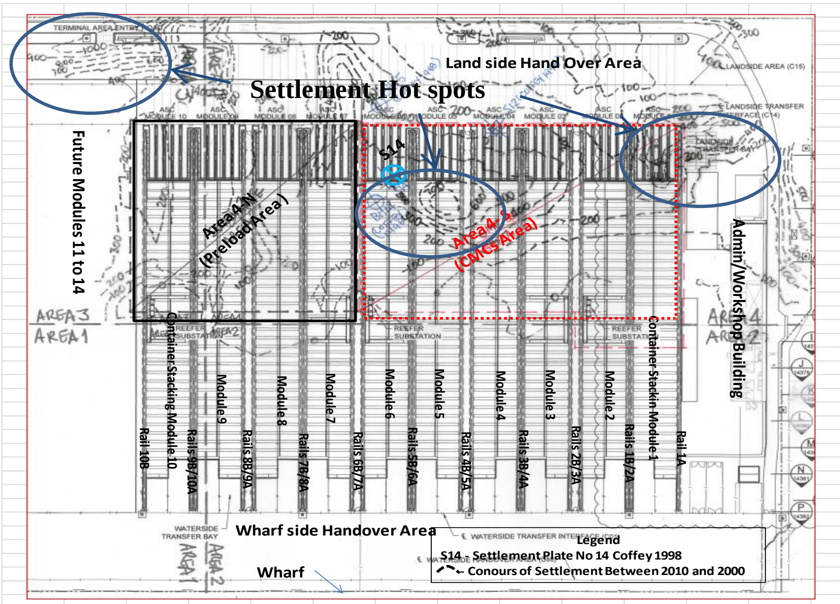
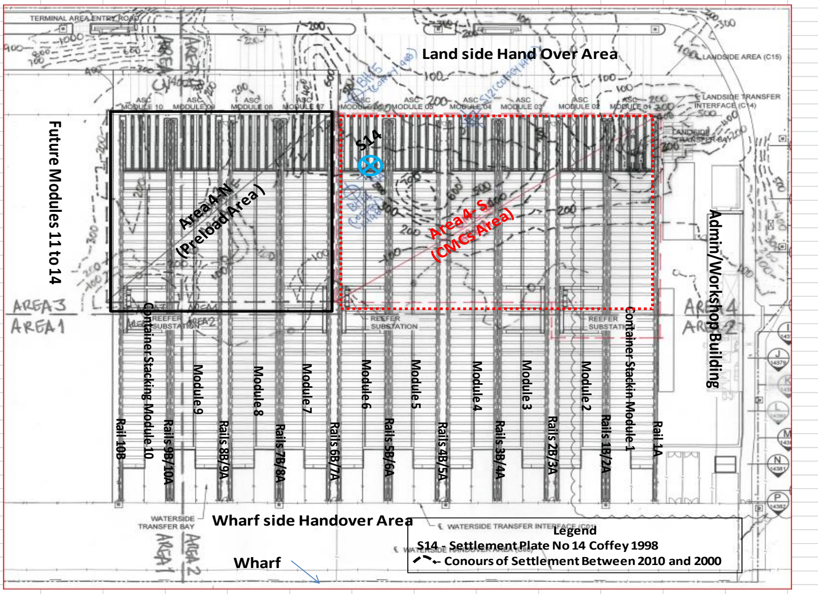


Figure 2: - Proposed terminal area

3 PROPOSED DEVELOPMENT AND CHALLENGES

The current TA development measures approximately 700m by 500m and accommodates the following components (as shown in **Figure 2**):



- A wharf to accommodate ship-to-shore cranes at the western margin of the development;
- Container stacking area with automated stacking cranes (ASCs) in the central part of the development;
- Wharf side handover area with automated container carriers (ACCs) west of container stacking area;
- Landside handover area east of the container stacking area; and
- Administration and workshop buildings to the south of the container stacking area.

Two of the most important design considerations for the automated container handling system are the rail foundations for the ASCs and the plinth foundations for stacking containers. As each ASC runs on two parallel rails to deliver containers to and from the container stacking areas, they have small operational tolerances. The operational axial loads of ASCs are 347 kN/wheel and the operational tolerances are:

- Gauge changes between two rails of the same crane (31.1m spacing) less than 20 mm;
- Lateral displacement of each rail less than 10 mm; and
- Differential settlements of less than 40 mm between two rails of the same crane.

In order to manage settlement exceeding the operational requirements, an adjustable rail and ballast system has been adopted. This system consists of two levels of adjustment. The first level involves levelling with adjustable hold down bolts attached to the rails that can handle up to 100 mm of settlement. The second level of levelling is undertaken after the first level adjustment can no longer be used, and involves re-ballasting of the track. The combined system can manage rail and track settlement of up to around 350 mm.

The container plinth foundations are required to support the loads of corner casts of containers stacked 5 high while also ensuring the settlements induced by the loads are limited to meet the ASC operational tolerance. These settlement restrictions are due to the automated nature of the ASC operation. The operational tolerances for the plinth foundations are:

- Differential settlement between plinths and the adjacent ASC rails is less than 40 mm; and
- Differential settlement between the plinths is less than 20 mm.

It was estimated that without ground improvement the current development could potentially be subjected to settlement of up to 500 mm over the 27 year design life. In order to satisfy the design requirements and reduce maintenance, ground improvement was required.

The aim of the ground improvement was to limit the post construction settlement over the 25 years of TA operation to 150 mm, so that typically only one re-ballasting of crane rails would need to be scheduled. As discussed in Section 1, a range of ground treatments was adopted depending on the site reclamation history, compressible layer thickness and the operational requirements imposed by the development. The remainder of this paper focus on Area 4-N where relatively uniform thickness of CIS allowed a preloading solution to be adopted.

4 SUBSURFACE CONDITIONS

A typical ground profile for the proposed preloading area is shown in Figure 3. Based on the site investigation at Area4-N, a typical subsurface profile has been developed and is presented in **Error! Reference source not found.**. This profile is based on interpretation of in-situ sampling and testing, combined with laboratory testing, from two boreholes (BH210 and BH224 – refer to Figure 8 for locations).

The units encountered and characterised in order of increasing depth include Upper Fill, Lower Fill, Coode Island Silt (CIS), Upper Fisherman's Bend Silt (FBS-U) and Lower Fisherman's Bend Silt (FBS-L). These units are discussed below while further discussion of settlement properties is made in Section 0.

Upper Fill: Comprises predominantly firm to stiff clayey soils that appear to have been placed post reclamation. SPT N-values ranged from 3 to 11 and cone penetrometer cone resistance (CPT q_t) values ranged from 0.5 to 1.5 MPa.

Lower Fill: Is predominantly very loose to medium dense sandy fill that is believed to have been hydraulically placed as part of the reclamation. SPT N-values ranged from 1 to 22 and CPT q_t values typically ranged from 1 to 5 MPa.

Coode Island Silt (CIS): Typically 2 to 5 m thick below the proposed preloading area, the CIS was typically recovered as dark grey, soft to firm high plasticity clay (liquid limit between 57 and 82%). The unit contained occasional sand lenses with occasional thicker silty sand bands up to 2m thick were also occasionally encountered as part of the formation. CPT q_t values typically ranged between 0.4 and 0.8 MPa, which corresponds to an undrained shear strength of between 25 and 50 kPa. The CIS is highly compressible with Oedometer testing indicating an initial void ratio (e_0) of between 1.40 and 1.85 and a compression index (c_c) between 0.51 and 0.77.

Fisherman's Bend Silt (FBS-U & FBS-L): The Fisherman's Bend Silt (FBS) is a low to high plasticity clay (liquid limit between 34 and 103%) of stiff to very stiff consistency, ranging from 5m to in excess of 20m thick. The deposits of FBS in particular the thicker units are expected to contribute to settlement. Two distinct layers within this unit were identified based on their plasticity, consistency and settlement characteristics, namely, the Upper Fisherman's Bend Silt (FBS-U) and Lower Fisherman's Bend Silt (FBS-L). The boundary between the two layers has been defined at RL -17mAHD. As shown in **Error! Reference source not found.**, the CPT q_t of the FBS-U fluctuates between 1.5 and 2.5 MPa while that of the FBS-L gradually decreases from around 2 MPa to around 1.5 MPa. The compressibility varied between the FBS-U and FBS-L units with the FBS-U being significantly more over-consolidated than the FBS-L. Oedometer testing found the over-consolidation ratio (OCR) to vary from 4 to 2, the initial void ratio to vary from 0.8 to 1.5 and the compression index to vary from 0.35 to 0.7 between the FBS-U and FBS-L, respectively.

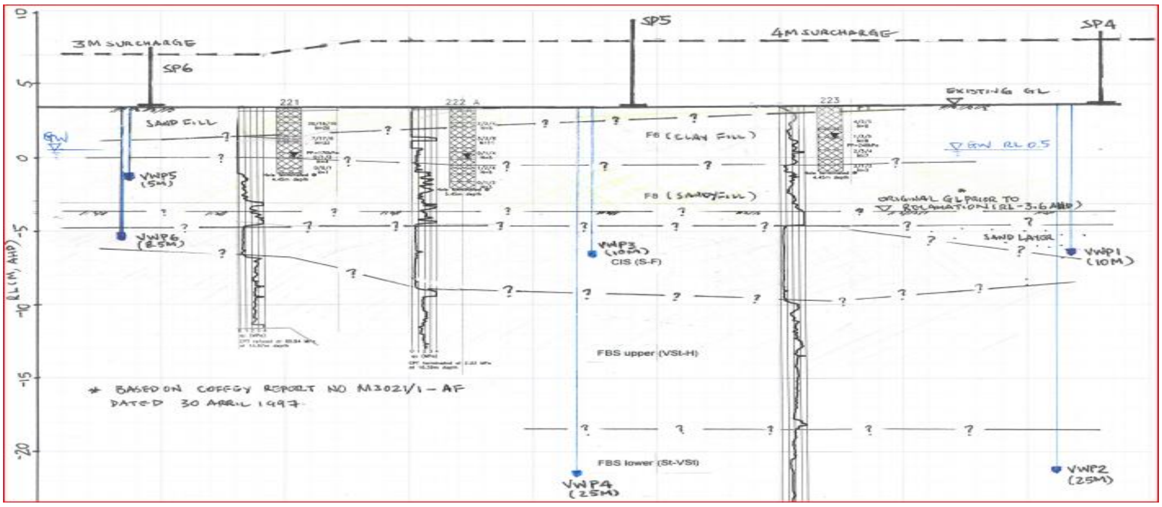


Figure 3: Typical subsurface profiles and VWP positions

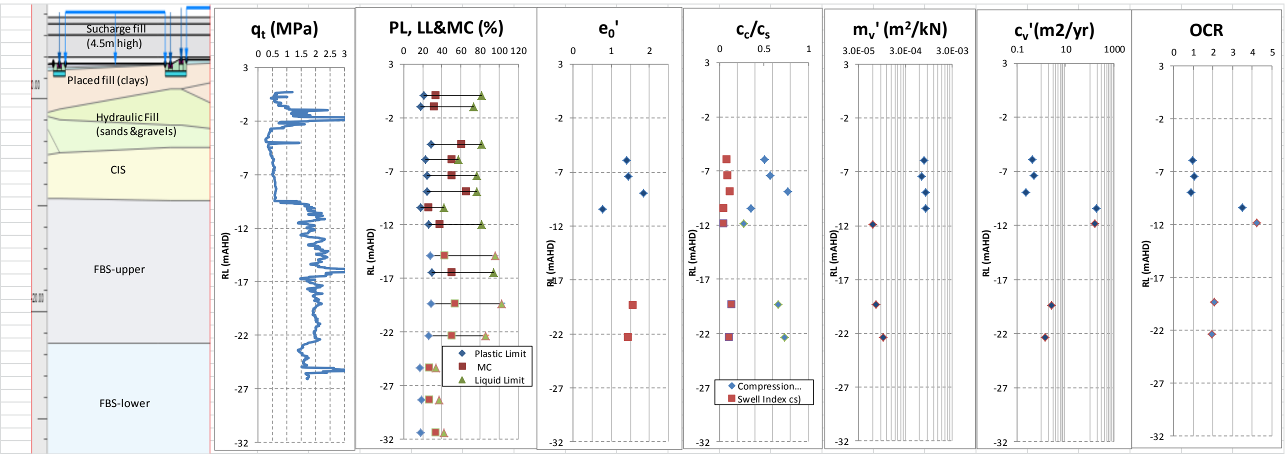


Figure 4: CPT q_t profile and material properties (BH210 & BH224)

Given the significance of settlement for the TA development, a detailed settlement analysis was undertaken as part of the geotechnical design works. The analysis was undertaken using finite element method software (PLAXIS 2D AE 2014). The left most pane of **Error! Reference source not found.** shows a ground profile from the PLAXIS model constructed for this analysis.

A back analysis was firstly undertaken for the existing ground conditions with the known load – settlement history such that the soil stresses modelled reflected the reclamation and previous development history. The material property inputs for PLAXIS were adjusted if necessary so that the calculated deformations match the measured settlements. A PLAXIS analysis was then undertaken using the back analysed properties for the proposed development, with and without preloading ground improvement. Finally, the analysis was undertaken, following completion of the preloading, to simulate the preloading process in real time. A comparison of the calculated and monitored settlements during the preloading was made and is presented in Section 7.

The soil properties adopted and the material models assumed for the PLAXIS deformation analysis are presented in **Figure 4**. Note that the properties presented are either directly adopted or correlated from the Oedometer tests shown in **Error! Reference source not found.** except for the c_α value of CIS, which is based on c_α = 0.03 c_c, (Balasubramaniam et al, 2010). The properties for fills have been omitted from the table as the soils were either clays above the water table or sands, and as such they contribute little to the long term consolidation settlement of the ground. Soil strength parameters were also required for the analyses, however, these have not been presented.

Table 2: Material properties * for consolidation analysis using PLAXIS

Material	Model	γ_w (kN/m^3)	e_o	c_c	c_s	c_α	k_v (m/d)	OCR
CIS	SSC	16	2.14	0.72	0.1	0.02	$6.95\text{x}10^{-5}$	1.0
FBS-U	HS	19	0.82	0.26	0.05		$1.92\text{x}10^{-4}$	4.0
FBS-L)	HS	17	1.6	0.66	0.11		$6.16\text{x}10^{-6}$	2.0

*γ_w – Bulk density of soils, e_o - initial void ratio, c_c - compression index, c_s - recompression index, c_α - coefficient of secondary compression, k_v - vertical permeability OCR - over consolidation ratio.

The Soft Soil Creep (SSC) material model has been adopted for modelling behaviour of the CIS while the Hardening Soil (HS) model was used for all other soils. HS can be used to model the behaviour of compressible soft soils but is not suitable when considering creep, or secondary compression. All soils exhibit some degree of creep, and thus the primary compression is therefore always followed by a certain amount of secondary compression. CIS is not an exception and is understood to exhibit significantly more creep than the FBS. There have been many records of significant creep settlements occurring in the Yarra Delta area (Ervin, 1992, and Srithar, 2010) and consequently it is more appropriate to adopt the SSC model for modelling the behaviour of the CIS.

The modelling of the soft soil behaviour is further discussed below in respect to the e_o and k_v values that were to be adopted for PLAXIS analysis.

As shown in **Error! Reference source not found.**, the void ratio (e_0´) of CIS ranges between 1.40 and 1.85. This is based on the Oedometer tests undertaken for CIS sampled during the current site investigation. A prime sign (´) is used to distinguish it from the initial void ratio of the virgin CIS prior to the land reclamation. The initial void ratios obtained from the Oedometer tests corresponded to a pre-consolidation pressure of 100kPa, as shown in Figure 5. These values, however, are not the true initial void ratios of the virgin material, which was modelled in PLAXIS from the time of pre development. To better model the material, the e – log(p) plots for the samples tested (see Figure 5) were extrapolated back to a pre-consolidation pressure of 10kPa to represent the stress state of the soil prior to development. As can be seen from Figure 5, this gave initial void ratios of between 1.8 and 2.6. An average initial void ratio of 2.14 was adopted in the PLAXIS analysis.

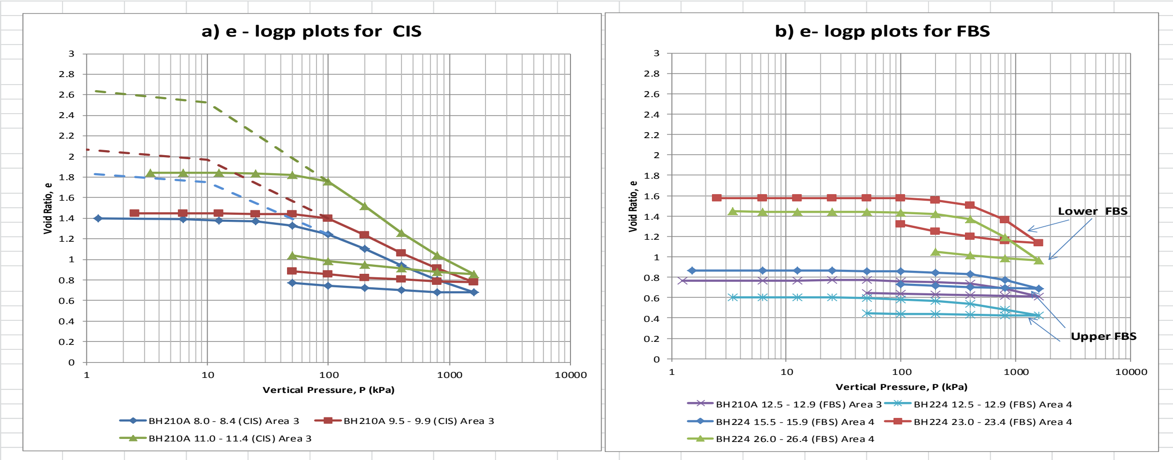


Figure 5: e-log(p) curves for CIS and FBS

Vertical permeability (k_v) values shown in **Figure 4Error! Reference source not found.** were correlated from the Oedometer tests using the equation: $k_v = \gamma_w . c_v . m_v$, refer to the footnote in Table **2** for definition of these variables. Oedometer tests conducted on samples obtained during the investigation yielded c_v results typically between 0.3 and 3 m^2/year. Dissipation tests conducted by the CPTu rig yielded a coefficient of horizontal consolidation (c_h) of between 0.3 and 9 m^2/year. Typically horizontal permeability is greater than vertical permeability as the in-situ dissipation testing captures the presence of sand lenses, which can reduce the drainage path and increase the coefficient of consolidation. At this site the presence of a free draining sand fill layer at the top of the CIS means that the rate of settlement is predominantly a function of the vertical permeability. Furthermore, as values of c_v reduce with increased loading and consolidation, the c_v values from before the original development of the dock are likely to be higher than those obtained from the recent Oedometer tests on the soil which has experienced compression under the reclamation fill.

Settlement Back Analysis

As was previously mentioned, a back analysis was undertaken to model the previous reclamation to allow a comparison between the calculated and monitored settlements under the previous development. The benchmarks for the back analysis were the measured settlements at the location of S14 –a settlement plate that was installed during 1998 (Coffey) and is close to the proposed surcharge area (shown on **Figure 2**) where the PLAXIS analysis was undertaken. At the location the measured settlement was 355mm for 2 years of monitoring, 516 mm for the 10 year period between 2000 and 2010 and 50 mm for the 5 year period between 2010 and 2015. The total measured settlement as at 2015 is 921mm. As noted in Table **1**, the measured settlement does not include the portion of settlement that had occurred prior to the installation of the settlement plates. In order to find this "missing" settlement, an assessment of the sea bed level prior to the reclamation level was compared with the current level that was assessed from CPTu data. As shown in Figure 3, the original sea bed level was RL -3.6 whereas the level indicated from CPTu is about RL - 4.7. The difference between these two levels is 1100 mm. Mud waving of CIS could have affected this assessment but the records indicate that there was no evidence of mud waving formed in the northern part of the reclamation (Area 4 –N). Therefore the figure of 1100 mm is considered to be good estimation of the total settlement at the S14 location.

It was found from the back analysis that the final settlement calculated using the average c_c, and c_s values adopted directly from the Oedometer tests matched very well with the measured final settlement. The settlement vs. time curves for the calculations and measurements, however, best matched when a c_v towards the high end of the Oedometer range of 3 m^2/year was adopted.

The settlement vs. time results of the PLAXIS back analysis, compared with the monitored settlements, are presented in **Figure 6**a. It can be seen that the curve calculated by PLAXIS closely matches the curve produced with the monitored points both during the early stage of the previous development, prior to 2000, and at a couple of milestone points at 2010 and 2015, after development. Some smaller differences at the earlier stage of the previous development may be due to discrepancy between the assumed and actual loading vs time sequence. Based on the PLAXIS analysis the residual settlement after 2015, under the existing fill loads, is estimated to be 56 mm over the future 10 to 20 years including a component of creep.

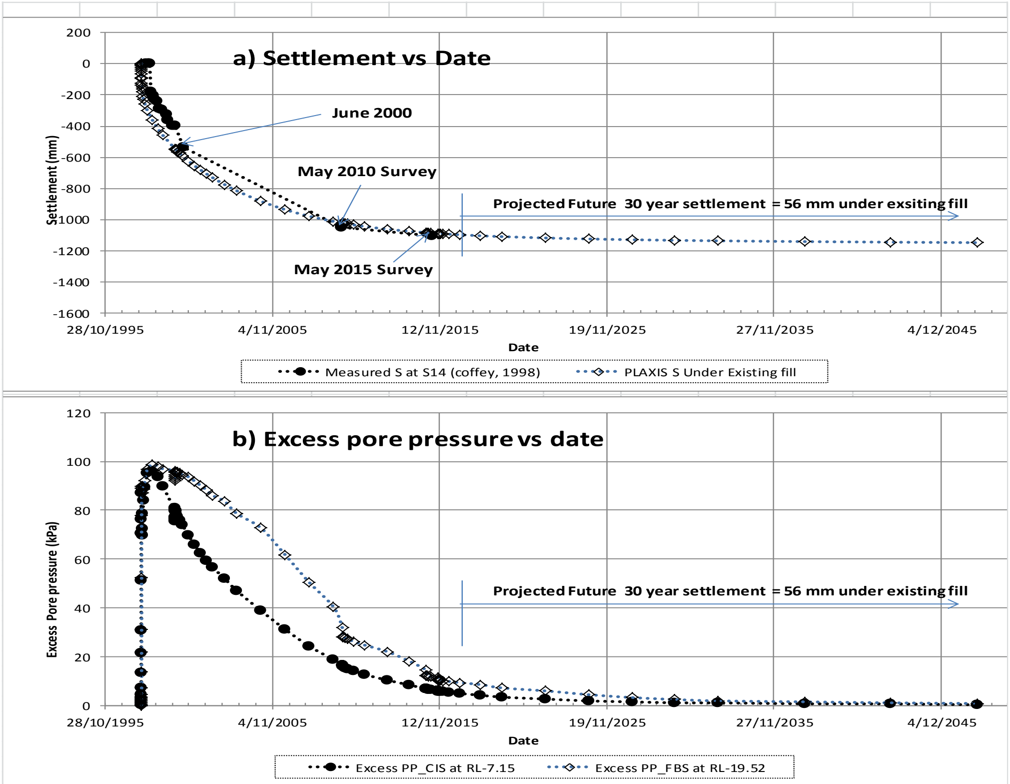


Figure 6: Settlement -Back Analysis for location at S14 (Coffey 1998)

The excess pore water pressure (u_e) vs. time results of the PLAXIS back analysis are presented in **Figure 6**b. However there is no measured pore pressure data to be compared with. The following key points can be observed from the results:

- The maximum excess pore pressure reached about 100 kPa within both the CIS and FBS during the reclamation, corresponding to the loads imposed by 5 to 6 m of reclamation filling and indicating a slow rate of consolidation;
- The excess pore pressure starts to dissipate after the reclamation, dissipating by about 94 kPa in CIS and 88 kPa in FBS to 2015 over a period of 17 years since 1998. The inferred degree of consolidation is 94% in CIS and 88% in FBS; and
- The pore pressure in the CIS dissipates faster than in the FBS, reflecting the drainage path being longer in FBS than in CIS.

Settlement Predictions

The results of the settlement back analysis discussed above appear to provide a reasonable basis for undertaking modelling of settlement of the future development. The modelling covered various loading cases, including the future development with and without preloading ground improvement. The loads adopted for the long term settlement analysis of the container stacks is 30 kPa after the live load deduction. The preloading case considered a 4.5m m high surcharge mound being placed for about 2 months followed by construction of the ASC rails and container plinths and the application of the 30 kPa container stack load. The surcharge load is equivalent to about 80 kPa, which is significantly higher than the container stack load of 30 kPa. The high surcharge was adopted to compensate for the short amount of time available for the preloading.

The settlement predictions for the operation of the development, based on the results of the analysis, are presented in **Figure 7** and summarised in **Table 3**. In addition to the settlement for the two cases with and without preloading, **Figure 7** also shows the predicted settlement under the existing fill alone scenario and the predicted settlement under permanent surcharge load scenario. It is worth noting that the horizontal axis in this figure shares the same date datum as that in Figure 6. Also note that the vertical axis in this figure has the same zero datum as that of the settlement in **Figure 6**. The settlements relative to those at the 17/09/2015, the commencement of preloading, are also shown both in **Figure 7** and in **Table 3**.

It can be seen from **Figure 7** that:

- Settlement under the future container stack load without preloading is predicted to be 201mm.
- Settlement under the future container stack load following the preloading is assessed to be 129mm, which is after 88 mm settlement is removed by preloading between 17/09/2015 and 12/12/2015.
- If the preload was to become a permanent load, the final settlement would be 440mm.

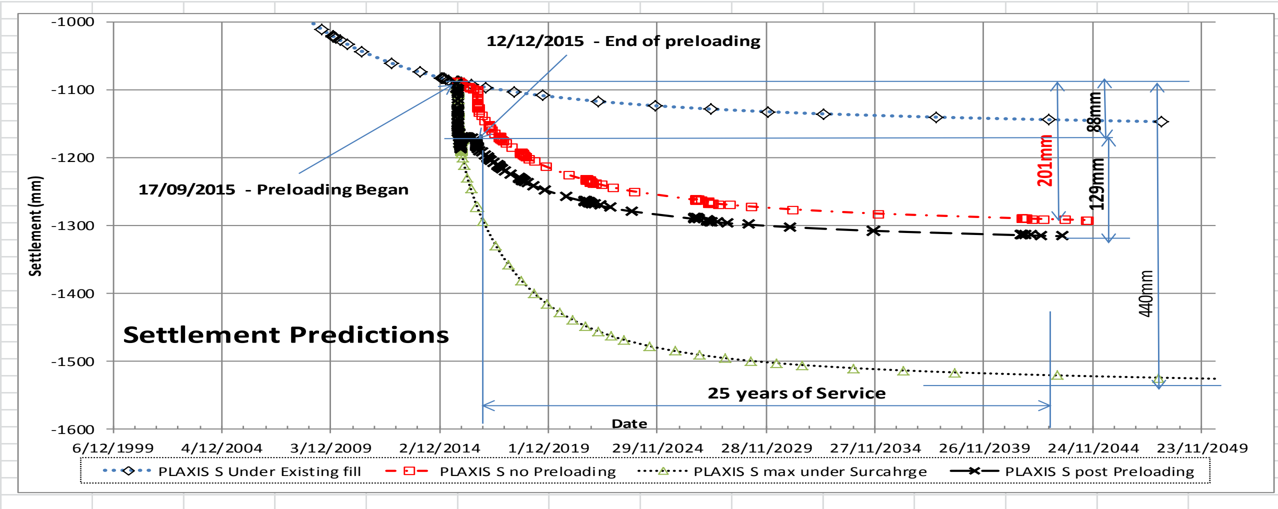


Figure 7: Results of settlement predictions

The settlement removed as a result of preloading is 88 mm, which counts for approximately 20% of the total 440 mm settlement that would have occurred should the preload become a permanent load. In other words, the 2 month preload has removed about 20% of the final consolidation settlement of the surcharge. On the other hand, the 88mm of settlement removed by preloading counts for about 44% of the 201mm settlement predicted to occur without preloading. Therefore preloading with a 4.5 m surcharge for 2 months could remove 44% of the potential settlement under the container load.

The analysis indicated that without preloading the future estimated settlement is 201 mm. Although this would still be within the manageable limits (see Section 3), may result in the need for re-ballasting maintenance more than once. On the other hand the predicted settlement of 129 mm for the preloading case would mean that the future ASC operations may only require re-ballasting maintenance once during their operational lifetime. The cost implication for this difference is understood to be significant to the client and therefore a preloading regime was approved.

Table 3: Summary of predicted settlements in future 25 years

Case	**Settlement (mm)**	
	Total since1998	Since Preload (17/09/2015)
Under existing fill alone	1150	56
Under existing fill + container load	1298	201
Under existing fill + 2 month preload + container load	1320	129 (88)*
Under existing fill + preload becoming permanent	1530	440

*figure in bracket is the settlement removed by preloading

5 PRELOADING DETAILS

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, shortening the drainage paths using prefabricated vertical drains or PVDs (Bergado, 2002) or vacuuming (Mohamedelhassan, 2002), or combinations of the above (Day, 2007).

Based on the analysis presented above, preloading with a surcharge of 3 m to 5 m high compacted earthfill for a period of 2 to 3 months would result in more than 40% reductions in potential settlements that would be experienced under the ASC and container stacking loads (as demonstrated in Section 0). The preloading was undertaken as a rolling operation and was initially undertaken for Module 7 and then gradually implemented at Modules 8, 9 and 10 (see Figure 8). Surcharging and the associated monitoring were undertaken separately for each module with a surcharge overlap ensured between modules to prevent any gaps over the area.

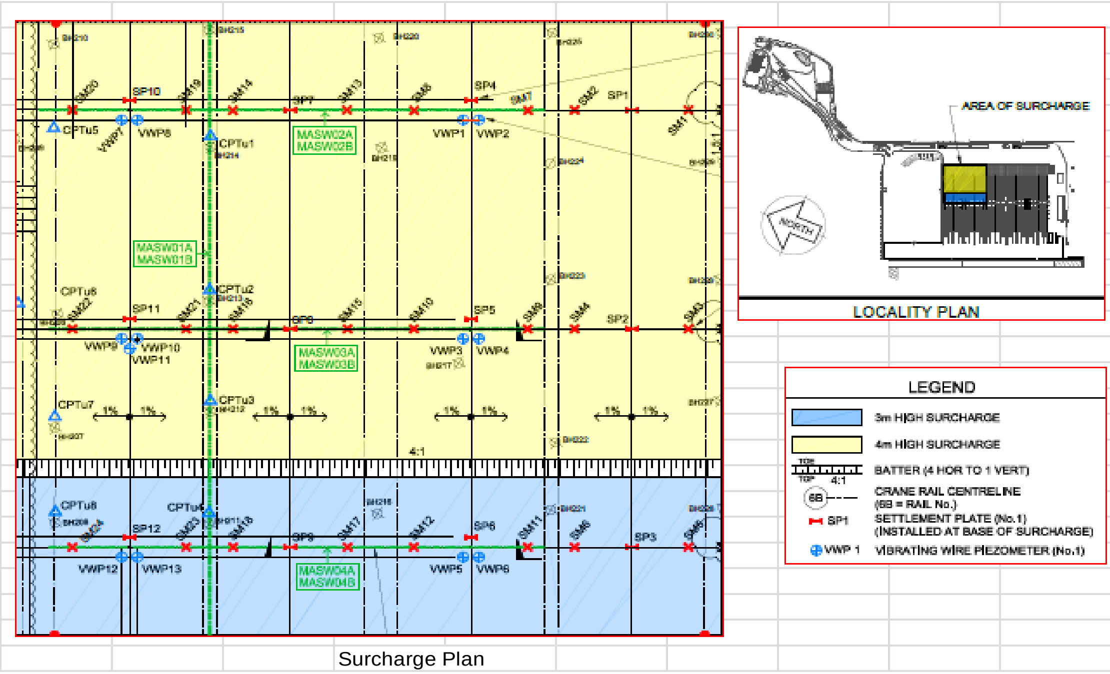


Figure 8: Plan of Preloading Area and Instrumentations

Monitoring instrumentation consisted of Settlement Plates (SPs), Inclinometers and Vibrating Wire Piezometers (VWPs) which allowed the settlement of surcharge, lateral displacement and pore water pressures in the ground beneath the surcharge to be observed. Figure 8 provides a plan of the instrumentation, along with the locations of boreholes drilled as part of the surcharge site investigation. A total of 12 settlement plates (SP1 to SP12) were installed at the existing ground surface prior to the placement of the surcharge so that a full settlement history could be surveyed. VWPs were installed at 15 locations at depths ranging between 4m and 25 m to target the compressible CIS, FBS-L and soft clay fill soils. Figure 3 depicts some of the SP and VWP installations. A total of six inclinometers were installed at the northern and southern ends of the surcharge, however, these will not be discussed in this paper.

6 RESULTS OF MONITORING

Monitoring demonstrated the successful transfer of the surcharge load to the ground below and consequent removal of settlement, the settlement removed by preloading is summarised in Table 4. This section provides a discussion of the monitoring results for Module 8, which is typical of the results achieved in other modules.

Table 4: Removed Settlements at Modules 7, 8, 9 and 10

	Module 7	**Module 8**	**Module 9**	**Module 10**
Range of Effective Settlement Removed (mm)	81 – 115	89 – 118	46 – <67	49 – 56

Figure 9 presents the surcharge loading sequence, measured settlements and measured pore pressures for the three settlement plates (SP4, SP5 and SP6) located within Module 8. The surcharge was built to 4.5m high and left at full height for 33 days at SP4 and SP5 and built to 3.7m high and left at full height for 56 days at SP6. Nearly half of the predicted long term settlements due to container loading was removed at SP4 and SP5, while approximately two thirds was removed at SP6.

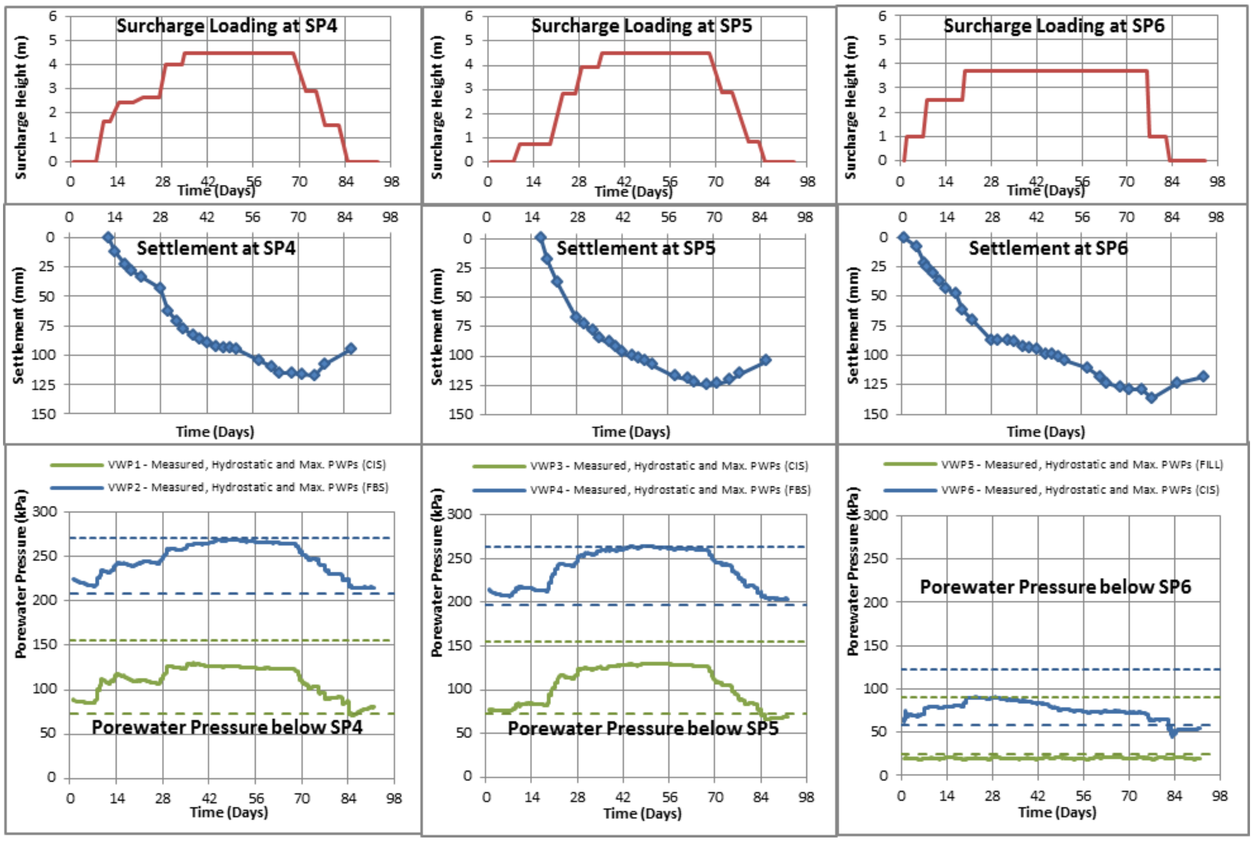


Figure 9: Settlement and pore pressures vs time

The pore pressures monitored in the CIS by VWP1, VWP3 and VWP6, depicted in Figure 9, show the progressive increase in pressure from the hydrostatic pressure during the surcharge loading placement period. The pressure fails to reach the theoretical maximum pressure that would be developed with 100% transfer of surcharge load to the CIS as an excess pore pressure. These small declines in pressure can be seen between the steep jumps associated with the rapid placement of surcharge material and indicate that consolidation of the CIS was occurring from the commencement of loading. Following placement of the surcharge there is generally a gradual decline in pressure before unloading is commenced and pressure drops back to that of hydrostatic. VWP3, however, does not appear to show clear dissipation during the surcharge holding period.

By contrast, the pore pressures monitored in the FBS-L by VWP2 and VWP4 show the pore pressure fully developed to the theoretical maximum pressure that would be developed under 100% load transfer to the soil with only a small reduction in pore pressure over the preload period. This is due to the longer drainage path leading to a slower rate of consolidation in this unit. The pore pressures monitored by VWP5, which is located in fill material show very little response to surcharge loading confirming this unit acts as a free draining material and a drainage path for consolidation of the CIS.

Following conclusion of the preloading, the PLAXIS analysis used to predict settlement was re-run replicating the as-built surcharging process. The results of the analysis at SP5 are presented in **Figure 10Error! Reference source not found.**, together with the measured settlement vs time curve in the middle of Module 8 surcharge (SP5 as shown Figure 8). At this location it can be seen that a very close match between the calculated vertical displacement and monitored settlements at the middle of Module 8 was achieved without any adjustment to the soil parameters.

Similar results were achieved at SP4 and SP6 within Module 8 and in Module 7. Although not presented in this paper, a similar approach was adopted for Modules 9 and 10 (Area 3) which achieved lower than expected settlements. The reclamation at these modules was completed about 24 years earlier than Modules 7 and 8 (Area 4) and may have been used for more heavily loaded pavements in the past. This may have had an influence on both the soil properties and over-consolidation assumptions. Some reduction in c_c would be required to replicate the observed settlements accurately at these locations.

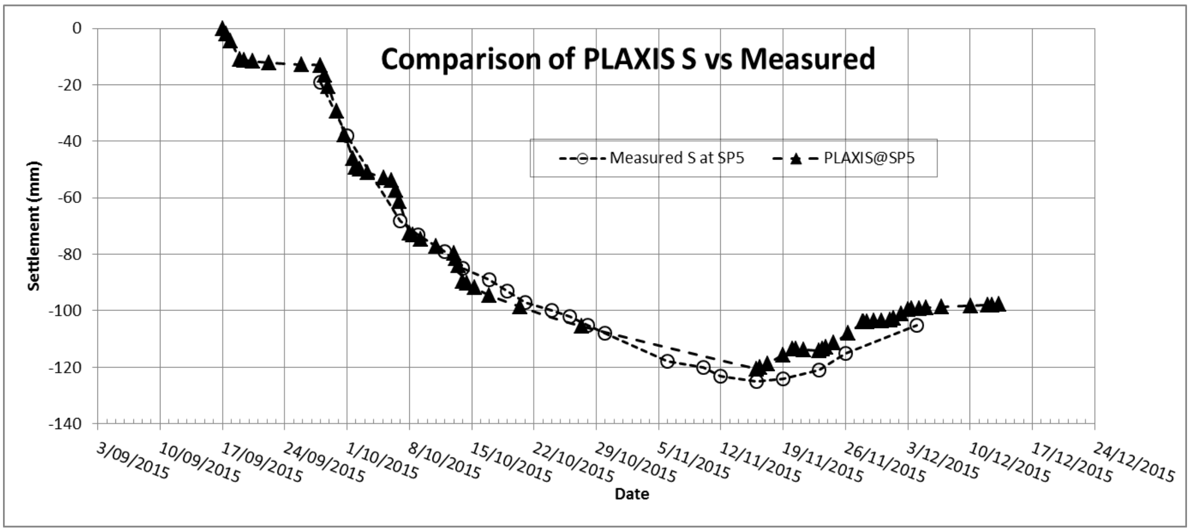


Figure 10: Comparison of measured and predicted settlements

7 CONCLUSIONS

The process adopted for back analysis of known settlements of the reclamation was successfully able to be used to derive consolidation parameters for Coode Island Silt (and to a lesser extent the Fishermans Bend Silt) that could in turn be used to predict additional consolidation settlements under preloading. These predictions demonstrated that around 20% to 30% of the primary consolidation of a relatively thin 3 to 5m thick layer of Coode Island Silt could be achieved from a preloading period as short as 2 to 3 months. Of note, it was found that laboratory derived consolidation parameters, particularly the compression index (c_c) and coefficient of vertical consolidation (c_v) provided a good match to observed behaviour of full scale reclamation and preloading.

Although removal of 20% to 30% of the primary consolidation (equivalent to 40% to 60% of the potential settlement under the design load) would not normally be sufficient to provide suitable long term settlement solution for a container terminal, by combining high preload surcharges (2 to 3 times the long term loads) and adopting a crane rail solution with the built in ability to adjust levels and re-ballast the crane rail over the design life, an acceptable solution could be achieved.

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