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Performance and Prediction of Vacuum Combined Surcharge Consolidation at Port of Brisbane

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ABSTRACT

During the past decade, the application of vacuum pressure for stabilising soft coastal clay and other low-lying estuarine soils has become increasingly popular in Australia, mainly due to the proven cost-effectiveness in view of the significantly reduced time for achieving a high degree of consolidation and enhanced shear strength. Due to an increase in trade activities at the Port of Brisbane, new facilities on Fisherman Islands at the mouth of the Brisbane River will be constructed on the new outer area (235ha) adjacent to the existing port facilities via land reclamation. A scheme of vacuum assisted surcharge load in conjunction with conventional surcharge and prefabricated vertical drains was selected to reduce the required consolidation time through the deeper subsoil layers. The design of the combined vacuum and surcharge fill system and the construction of the embankment are described in this paper. A comparison of the performance of the vacuum combined surcharge loading system with a standard surcharge fill highlights the clear benefits of vacuum pressure application. A comprehensive array of field data is presented to demonstrate how the embankment had performed during construction. A new analytical solution for radial consolidation considering both time-dependent surcharge loading and vacuum pressure is proposed to predict the settlements and associated excess pore pressures of the soft Holocene clay deposits in the Port of Brisbane, for future design of port infrastructure under increasing live loads over the next two decades.

Keywords: Consolidation, Preloading, Reclamation, Surcharge, Vertical Drain, Vacuum Pressure

1 INTRODUCTION

The traditional preloading method of improving the performance of soft clays by a surcharge fill embankment is generally a low-cost solution. However, in thick soft soil sites (more than 10m deep), there can be significant delays due to the very low soil permeability and the lack of effective drainage. When prefabricated vertical drains (PVDs) are installed in the ground, the drainage path is shortened (radial), thereby considerably reducing the time for excess pore pressure dissipation (Holtz et al. 1991; Indraratna & Redana 2000). The installation of PVD, followed by the application of vacuum pressure (suction) as a preload at the soil surface (i.e. prior to construction of the main structure) facilitates rapid dissipation of pore water pressure in the soil (Shang et al., 1998; Bergado et al. 2002; Bo et al. 2003). Suction pressures (negative) propagated along the drain length increases the radial hydraulic gradient towards the drain, preventing the build up of excess pore water pressure in the soil, thus considerably reducing the risk of failure (Indraratna et al. 2004). The vacuum application with PVDs was applied successfully for road embankments and port reclamation projects in Australia.

In this paper, the performance comparison between the vacuum and non-vacuum areas at the Port of Brisbane has been made based on the measured vertical deformations, excess pore pressures and horizontal displacements. The effects of drain spacing, drain type and improvement technique are elaborated based on the observed degree of consolidation. The analytical solutions for radial consolidation considering both time dependent surcharge loading and vacuum pressure are proposed to predict settlement and associated excess pore pressure.

2 SITE DESCRIPTIONS

The Port of Brisbane is Australia's third largest container port located between the mouth of the Fisherman Islands and Brisbane River (Indraratna et al. 2011). An outer area (2,350,000 m²) adjacent to the current port facilities has been reclaimed to maximise the available land, and to provide the additional berths suitable for imported and exported cargo and container handling. Based on the geotechnical site investigation data, the soil profile consists of a highly compressible marine clay layer over 30m in thickness with an undrained shear strength near the surface lower than 15 kPa. The strength of the dredged mud used for reclamation was very low (<5kPa) depending on the time of placement and the duration the capping material (surcharge). Without surcharge preloading, it was shown that the consolidation time would be longer than 50 years with vertical displacements of 2.5-4.0m under the required service loadings. Therefore, vacuum consolidation with prefabricated vertical drains (PVDs) was selected to speed up the consolidation process and to limit horizontal deformation which can affect the Moreton Bay Marine Park located immediately to the site (Austress Menard 2008). In order to compare the performance of the vacuum system and the non-vacuum system (PVD and surcharge load), a trial area (S3A) shown in Fig. 1 was divided into sub areas WD1-WD5 (Non-vacuum areas) and VC1-VC2 (vacuum preloadng). Each area encompasses 0.15 to 1.1 ha. After drying, the mud was capped off with a few meters of dredged sand, which performs as a working platform for PVD installation rigs, while providing a drainage layer for vacuum system.

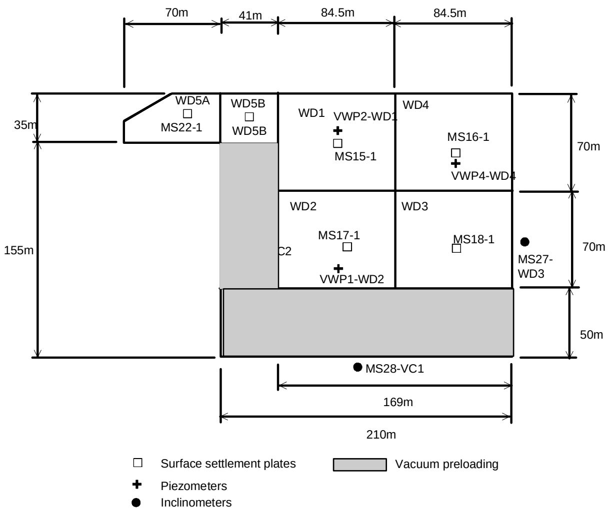

Figure 1. Site layout for S3A with instrumentation plan (Data Source: Indraratna et al. 2011)

The upper Holocene sand underneath the dredged mud was approximately 2m thick, and it overlies the lower Holocene clay layer having a thickness varying from 6m to 25m. The highly compressible Holocene clay layer with low shear strength is referred to as PoB clay (Ameratunga et al. 2010). The Holocene clay overlies the stiff deposit (highly over-consolidated). Site investigations including cone penetration/piezocone tests, dissipation tests, boreholes, field vane shear tests and oedometer tests were performed to evaluate the consolidation and stability parameters. The soil profile and the corresponding soil properties are shown in Fig. 2 where the groundwater level is located at +3.5m RL. The water contents of the soil layers were at or more than their liquid limits. The field vane tests indicated that the undrained shear strength of the dredged mud and the Holocene clays varied from 5 to 60 kPa. The compression index (C_c) varied from 0.1-1.0. The coefficient of consolidation in vertical direction (c_v) was approximately the same as that in horizontal direction (c_h) for the totally remoulded dredged mud layer, while c_v/c_h ratio is about 2 for the Holocene clay.

The vacuum combined preloading approach was applied to curtail lateral displacement and to minimise disturbance of the marine eco-system. Rigorous design specifications were considered including (a) service load of 15-25 kPa, and (b) maximum residual settlement of not more than 250 mm over 20 years after the application of service load. Based on the design criteria, the surcharge embankment heights varied from 3.0m to 9.0m. Table 1 presents the PVD characteristics and treatment types applied to each section. In non-vacuum areas, both circular and band shape PVDs were installed in a square pattern at a spacing varying between 1.1-1.3m. The lengths of drains were from 6m to 28.7m across the site as shown in the Table 1. The variation in drain lengths was owing to the non-uniform clay thickness. At this site, wick drains (band drain Type-A and band drain Type-B) typically had dimensions of 100mm x 4mm, and the circular pipe drains had an internal diameter of 34mm. The Authors have purposely omitted the commercial brand names of all PVDs used. The soil profiles for each area are tabulated in Table 2. The variation of the Holocene clay depth is significant from one to another, with the depth of lower Holocene clay changing from about 6m (VC1) to 23m (WD4). The thickness of the upper Holocene clay was from about 1.5m (WD4) to about 5m (WD2).

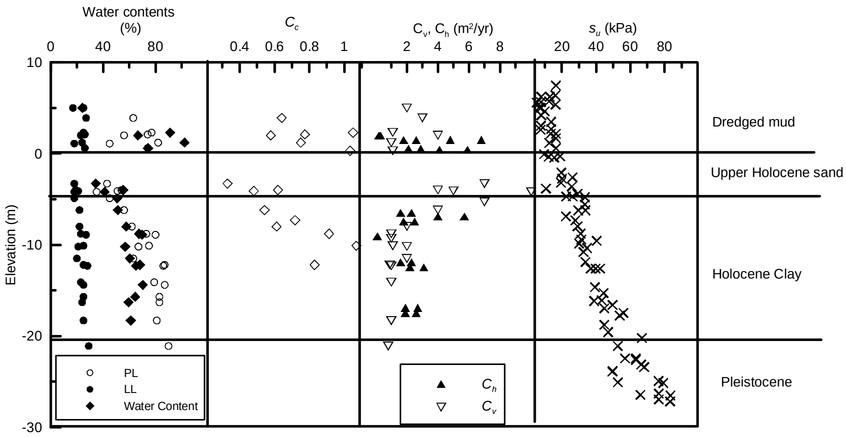

Figure 2. Soil properties and profile at S3A, Port of Brisbane (Data Source: Indraratna et al. 2011)

Table 1: PVD characteristics and improvement scheme (Indraratna et al. 2011)

Section	Drain type	Drain length (m)	Drain spacing (m)	Clay thickness (m)	Total fill height (m)
WD1	Circular drains - 34mm diameter	14.5-18.5	1.1	12.0-15.5	5.2
WD2	Circular drains - 34mm diameter	22.5-27.5	1.3	20.0-23.5	7
WD3	Band drain Type-A (100×4 mm^2)	17.1-23.5	1.1	14.0-17.0	4.3
WD4	Band drains Type-A (100×4 mm^2)	27.0-28.7	1.3	22.5-24.5	6.1
WD5A	Band drains Type-B (100×4 mm^2)	6.0-8.0	1.2	6.0-8.0	3.3
WD5B	Band drains Type-B (100×4 mm^2)	13.5	1.1	9.5	5.5
VC1	Circular drains - 34mm diameter	14.0-26.5	1.2	9.0-21.0	3.2
VC2	Circular drains - 34mm diameter	15.5-22.5	1.2	12.5-18.5	2.8

Table 2: Soil profiles for individual sections (Indraratna et al. 2011)

Area	Layer Thickness (m)			
	Dredged Mud	Upper Holocene Sand	Upper Holocene Clay	Lower Holocene Clay
WD1	2-3	1-2	4-6	10-12
WD2	1-2.5	1-3	2-5	18-20
WD3	2-4	1-3	2-3	8-15
WD4	1.5-2.2	1-2	1.5-3.5	18-23
WD5A	0-1	0-1	2-4	6-8
WD5B	2-4	1-2	2-4	7-8
VC1	2-3	2-3	2-3	5-18
VC2	0.5-2.5	2-3	2-3	9-16

3 ANALYSIS OF FIELD DATA

The embankment responses including vertical displacements and excess pore pressures together with the staged construction of the embankments are presented in Fig. 3. It can be seen that the settlement curves are very similar where the settlement occurs more swiftly at the initial consolidation stage. The magnitude of ultimate settlement relies on the clay thickness and embankment height. The maximum settlement was in the WD4 area having the greatest clay thickness (19-26m), while the minimum settlement was observed to be in WD5A area in which the clay layer is relatively thin (8-12m).

Based on the settlement-based degree of consolidation (DOC) for selected settlement plate locations and three drain types used in S3A area, namely, circular drain, band drain Type-A and band drain Type-B, there is no distinct relationship between the drain types and the target DOC according to the settlement - based analysis. It is important to note that the DOC achieved in VC2 is higher than the surcharge only areas.

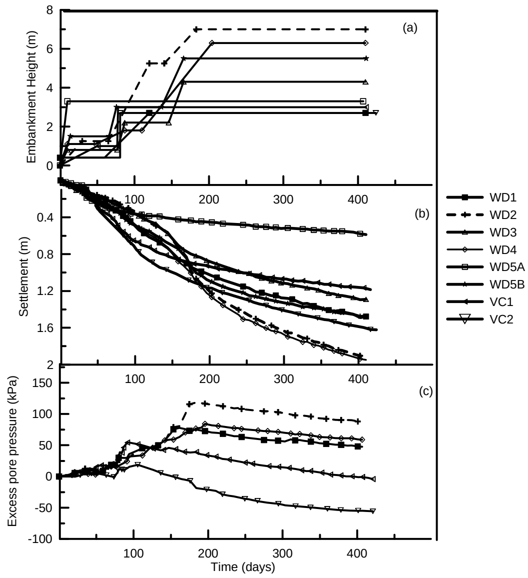

Figure 3. Embankment responses (a) staged construction, (b) settlements and (c) excess pore pressures (Data Source: Indraratna et al. 2011)

In order to compare system performance in terms of pore pressure dissipation, the time dependent excess pore water pressure reduction with time is shown in Fig. 4. VC1 and VC2 with circular drains provide the best option in view of excess pore pressure dissipation rates, compared to the surcharge only areas. This is possibly, because, the circular drains offer less resistance to vacuum pressure propagation compared to thin band drains (i.e. without losing suction head). The measured horizontal displacement normalized to total change in applied stress (vacuum plus surcharge load) for two inclinometer locations (VC1/MS28 and WD3/MS27) is presented in Fig. 5. In VC1 and WD3 areas, the total load on the surface was almost the same. As expected, the lateral displacements were smallest in the Holocene sand layer. These plots clearly indicate that the lateral movements are effectively controlled via isotropic vacuum consolidation.

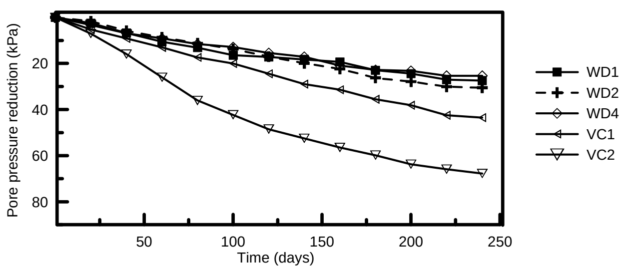

Figure 4. Pore pressure reduction after embankments reached the maximum height at various sections (Data Source: Indraratna et al. 2011)

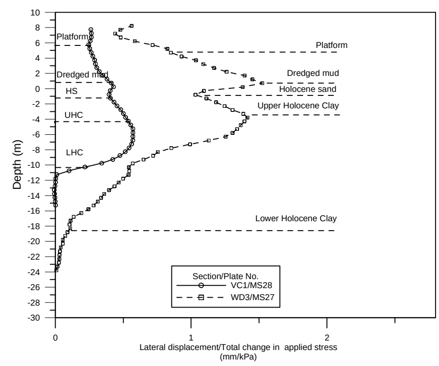

Figure 5. Comparison of lateral displacements at the embankment toe in vacuum and non-vacuum area after 400 days (Data Source: Indraratna et al. 2011)

4 SETTLEMENT AND EXCESS PORE PRESSURE PREDICTIONS USING UNIT CELL THEORY

Following the procedure proposed by Lekha et al. (1998), the excess pore pressure due to radial consolidation considering smear effect under time-dependent surcharge can be expressed by:

$$\overline{u_L} = \frac{\mu d_e^2}{8c_h t_0}\left(1-\exp\left(\frac{-8c_h t}{\mu d_e^2}\right)\right)\sigma_1 \quad \mathit{for} \qquad 0 \le t \le t_0 \tag{1}$$

$$\overline{u_L} = \frac{\mu d_e^2}{8c_h t_0}\left(1-\exp\left(\frac{-8c_h t_0}{\mu d_e^2}\right)\exp\left(\frac{-8c_h\left(t-t_0\right)}{\mu d_e^2}\right)\right)\sigma_1 \quad \mathit{for} \quad t > t_0 \tag{2}$$

Recently, Indraratna et al. (2005) showed that the excess pore pressure under radial consolidation due to vacuum pressure alone could be determined from:

$$u_{vac} = 0, \qquad t < t_{vac} \tag{3}$$

$$u_{vac} = p_0 \exp\left(-\frac{-8c_h\left(t-t_{vac}\right)}{\mu d_e^2}\right) - p_0\,, \qquad t \ge t_{vac} \tag{4}$$

$$\mu = \left[\ln\left(\frac{n}{s}\right) + \frac{k_h}{k_s}\ln(s) - \frac{3}{4} + \pi\frac{2k_h}{3q_w}l^2\right] \tag{5}$$

$$n = d_e / d_w \tag{6}$$

$$s = d_s / d_w \tag{7}$$

where, d_e = the diameter of soil cylinder dewatered by a drain, d_s = the diameter of the smear zone, d_w = the equivalent diameter of the drain, k_s = horizontal soil permeability in the smear zone and q_w = drain discharge capacity.

The excess pore pressure at a given time *t* can determined based on the combination of Equations (2) to (7).

In order to predict excess pore pressures and associated settlements, Equations (1)-(7) are used in conjunction with the soil properties for each layer. The vertical and horizontal coefficients of consolidation were determined using the oedometer and Rowe cell. For the completely remoulded dredged mud that was reclaimed from the seabed and Upper Holocene Sand that ratio k_h/k_s were assumed to be unity. For the upper and lower Holocene clay, the ratios of k_h/k_s and d_s/d_w were assumed to be 2 and 3, respectively, in accordance with the laboratory tests conducted by Indraratna and Redana (2000).

The embankment load was applied according to a staged construction (with compacted unit weight of 20 kN/m^3). Settlement and associated excess pore pressure predictions were carried out at the embankment centreline using the proposed analytical model. The computation of consolidation settlement and excess pore pressure at the centreline (zero lateral displacements) is straightforward and follows the basic 1-D consolidation theory. In vacuum areas, the suction pressure of 65 kPa based on field measurement was used to compute the settlement and excess pore pressure.

Figure 6 presents the predicted settlement and associated excess pore pressure with the measured data in VC2. Overall, the comparisons between prediction and field observation show that the settlement and associated pore water pressure can be predicted very well. The degree of consolidation exceeded 90% at 400 days. This confirms that, at a given time, the vacuum combined preloading would accelerate consolidation faster than the surcharge preloading alone.

5 CONCLUSION

Based on Port of Brisbane experience, it can be confirmed that a system of vertical drains with vacuum preloading would be an effective method for speeding up soft soil consolidation. A vacuum consolidation with membrane type system was utilised with an array of instrumentation including settlements, piezometers and inclinometers. In the surcharge only areas, both band PVDs and circular PVDs yielded a similar performance. However, the circular drains performed better in vacuum areas probably because they could propagate vacuum pressure more effectively to a larger depth than band drains. When the vacuum pressure combined with surcharge fill is used, the overall lateral displacement can be controlled effectively by the isotropic vacuum consolidation.

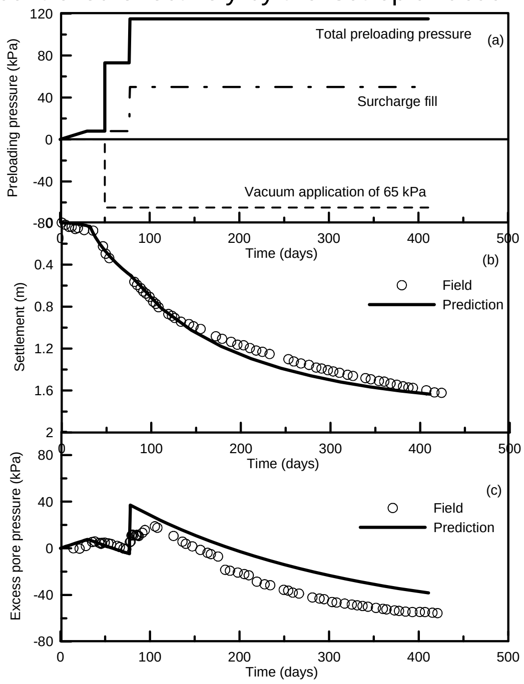

Figure 6. VC2 area: (a) stages of loading, (b) surface settlements at the embankment centreline and (c) excess pore pressures at 14.1m deep (Data Source: Indraratna et al. 2011)

6 ACKNOWLEDGEMENT

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