2012

Performance and prediction of surcharge and vacuum consolidation via prefabricated vertical drains with special reference to highways, railways and ports

Buddhima Indraratna
University of Wollongong, indra@uow.edu.au

Cholachat Rujikiatkamjorn
University of Wollongong, cholacha@uow.edu.au

Xueyu Geng
University of Wollongong, xgeng@uow.edu.au

Publication Details

Indraratna, B., Rujikiatkamjorn, C. Geng, X. (2012). Performance and prediction of surcharge and vacuum consolidation via prefabricated vertical drains with special reference to highways, railways and ports. In N. Denies H. Noel (Eds.), International Symposium on Ground Improvement (pp. II-145-II-168).

Performance and prediction of surcharge and vacuum consolidation via prefabricated vertical drains with special reference to highways, railways and ports

Buddhima Indraratna[1] Cholachat Rujikiatkamjorn[2], Geng Xueyu[3]

[1] Professor, Director, Centre for Geomechanics and Railway Engineering, Faculty of Engineering, University of Wollongong, NSW, Australia, email: indra@uow.edu.au

[2] Senior Lecturer, Centre for Geomechanics and Railway Engineering, Faculty of Engineering, University of Wollongong, NSW, Australia
Buddhima Indraratna[1]
[3] Research Fellow, Centre for Geomechanics and Railway Engineering, Faculty of Engineering, University of Wollongong, NSW, Australia

ABSTRACT

Much of the world's essential infrastructure is built along congested coastal belts that are composed of highly compressible and weak soils up to significant depths. Soft alluvial and marine clay deposits have very low bearing capacity and excessive settlement characteristics, with obvious design and maintenance implications on tall structures and large commercial buildings, as well as port and transport infrastructure. Stabilising these soft soils before commencing construction is essential for both long term and short term stability. A system of vertical drains combined with vacuum pressure and surcharge preloading has become an attractive ground improvement alternative in terms of both cost and effectiveness. This technique accelerates consolidation by promoting rapid radial flow which decreases the excess pore pressure while increasing the effective stress. Over the past 15 years, the Author and his co-workers have developed numerous experimental, analytical, and numerical approaches that simulate the mechanics of prefabricated vertical drains (PVDs) and vacuum preloading, including two-dimensional and three-dimensional analyses, and more comprehensive design methods. These recent techniques have been applied to various real life projects in Australia. The equivalent 2-D plane strain solution is described which includes the effects of smear zone caused by mandrel driven vertical drains. The equivalent (transformed) permeability coefficients are incorporated in finite element codes, employing the modified Cam-clay theory. Numerical analysis is conducted to predict the excess pore pressures, lateral and vertical displacements. Two case histories are discussed and analysed, including Port of Brisbane and Ballina and the predictions are compared with the available field data. These recent advances enable greater accuracy in the prediction of excess pore water pressure, and lateral and vertical displacement of the stabilised ground.

1. INTRODUCTION

The booming population and associated development in coastal and metropolitan areas have necessitated the use of previously undeveloped low lying areas for construction purposes (Indraratna et al., 1992). Most of the Australian coastal belt contains very soft clays up to significant depths, especially in Northern Queensland and New South Wales. The low bearing capacity and high compressibility of these deposits affects the long term stability of buildings, roads, rail tracks, and other forms of major infrastructure (Johnson 1970). Therefore, it is imperative to stabilise these soils before commencing construction to prevent unacceptable differential settlement. However, attempts to improve deep bearing strata may not commensurate with the overall cost of the infrastructure (Bo et al. 2003). In the past, various types of vertical drains such as sand drains, sand compaction piles, PVDs (geo-synthetic), stone columns, and gravel piles have commonly been used. Certain types of granular piles and deep stone columns may indeed significantly enhance the intensity of the soil. However, because of the minimum risk of damage to utilities from lateral ground movement and a significant reduction in the price of flexible PVDs over the years, they have often been used to more conventional the original compacted sand drains, gravel piles stone columns etc. Their installation can significantly reduce the preloading period by decreasing the length of the drainage path, sometimes by a factor of 10 or more. More significantly, PVDs can be installed quicker with minimum environmental implications and quarrying requirements.

Preloading (surcharge embankment) is one of the most successful techniques for improving the shear strength of low-lying areas because it loads the ground surface to induce a greater part of the ultimate settlement that it is expected to bear after construction (Richart 1957; Indraratna and Redana 2000; Indraratna et al. 2005a). In order to control the development of excess pore pressure, a surcharge embankment is usually raised as a multi-stage exercise, with rest periods between the loading stages (Jamiolkowski et al. 1983). Since most compressible low-lying soils have very low permeability and are often thick, a lengthy time period is usually needed to achieve the desired primary degree of consolidation (>95%). In these instances, the height of the surcharge can be excessive from an economic perspective and stability consolidation (Indraratna et al. 1994). One drawback of this surcharge technique is that the preload should be applied for a sufficiently long period, which may at times become impractical due to stringent construction schedules and deadlines. When PVDs combined with surcharge preloading is applied, vertical drains provide a much shorter drainage path in a radial direction which reduces the required preload period significantly. PVDs are cost effective and can be readily installed in moderate to highly compressible soils (up to 40m deep) that are normally consolidated or lightly over-consolidated. PVDs do not offer particular advantages if installed in heavily over-consolidated clays.

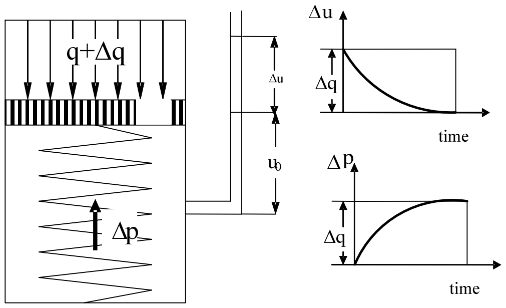
Apart from the method of drainage and PVDs combined with surcharge preloading, vacuum pressure has been used to enhance the efficiency of PVD when a desired degree of consolidation is required over a relatively short time period. Negative pore pressures (suction) distributed along the drains and on the surface of the ground accelerate consolidation, reduce lateral displacement, and increase the effective stress. This allows the height of the surcharge embankment to be reduced to prevent any instability and lateral movement in the soil. Today, PVDs combined with vacuum preloading are being used more and more in practical ground improvement all over the world.

This paper includes selected salient aspects of more than 15 years of active research conducted at the University of Wollongong in the area of soft soil stabilisation using PVDs and vacuum preloading, undoubtedly a vital Australian contribution to the field of soft soil improvement worldwide, plus offering a significant component of higher education training through almost a dozen doctoral studies to date, promoting the advancement of current industry practices in infrastructure development in coastal and low lying areas.

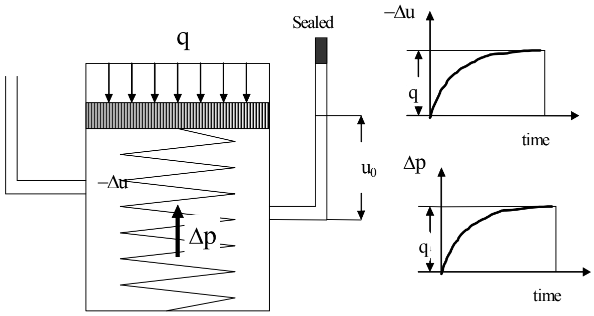
2. PRINCIPLES OF VACUUM CONSOLIDATION VIA PVDS

The vacuum preloading method for vertical drains was arguably first introduced in Sweden by Kjellman (1942). Since then, it has been used extensively to accelerate the consolidation of soft ground worldwide, for instance at the Philadelphia International Airport, USA; Tianjin port, China; North South Expressway, Malaysia; Reclamation world in Singapore and Hong Kong, China; Suvarnabhumi Second Bangkok International Airport, Thailand; Balina Bypass New South Wales and the Port of Brisbane, Queensland in Australia, among many other projects (Holtan 1965; Choa 1990; Jacob et al. 1994; Bergado et al. 2002; Chu et al. 2000; Yan and Chu 2003). When a high surcharge load is needed to achieve the desired undrained shear strength, and this cost becomes substantial due to an excessively high embankment and a long preloading period in order to achieve 95% or more consolidation, the optimum solution is to adapt to a combined vacuum and fill surcharge approach. In very soft clays where a high surcharge embankment cannot be constructed without affecting stability (large lateral movement) or having to work within a tight construction schedule, the application of vacuum pressure is quite often the most appropriate choice.

This PVD coupled system is designed to distribute the vacuum (suction) pressure to deep layers of the subsoil to increase the consolidation rate of reclaimed land and deep estuarine plains (e.g. Indraratna et al. 2005b; Chu et al. 2000). The mechanism for vacuum preloading can be explained by the spring analogy (Fig.1) described by Chu and Yan (2005), where the effective stress increases directly due to the suction (negative) pressure, while the total stress remains the same. This is in the context of conventional case surcharge preloading.



(a) $$u_0 = q$$
$$\Delta p = q + \Delta q - (u_0 + \Delta u) = \Delta q - \Delta u$$



(b) $$u_0 = q$$
$$\Delta p = q - (u_0 - \Delta u) = \Delta u$$

Figure 1: Spring analogy of vacuum consolidation process: (a) under fill surcharge; (b) under vacuum load (adopted from Chu and Yan 2005).

The general characteristics of vacuum preloading compared to conventional preloading are as follows (Qian et al. 1992; Indraratna and Chu 2005):

- The effective stress related to suction pressure increases isotropically, whereby the corresponding lateral movement is compressive. Consequently, the risk of shear failure can be minimised even at a higher rate of embankment construction, although any ‘inward’ movement towards the embankment toe should be carefully monitored to avoid excessively high tensile stresses.

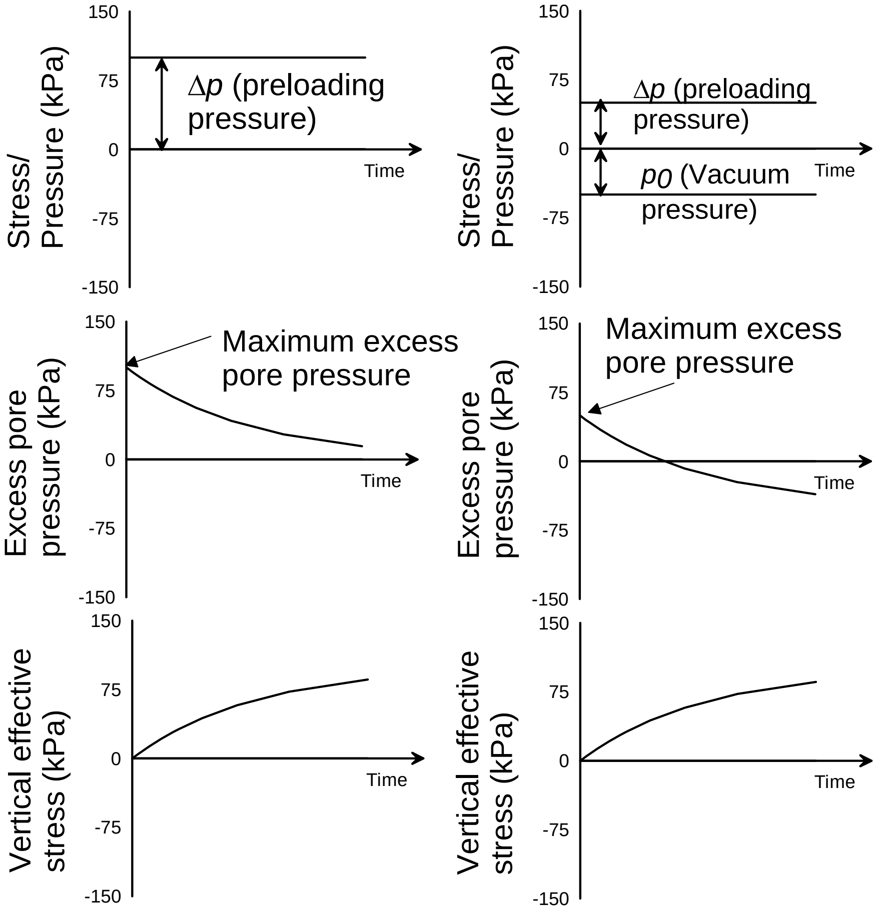
- The vacuum head can propagate to a greater depth of subsoil via the PVD system and the suction can propagate beyond the tips of the drain and the boundary of the PVD.

- Assuming on air leaks and depending on the efficiency of the vacuum system used in the field, the volume of surcharge fill can be decreased to achieve the same degree of consolidation.

- Since the height of the surcharge can be reduced, the maximum excess pore pressure generated by vacuum preloading is less than the conventional surcharge method (Fig. 2).

- With the applied vacuum pressure, the inevitable unsaturated condition at the soil-drain interface may be partially compensated for.

- With field vacuum consolidation, the confining stress applied to a soil element may consist of two parts: (a) vacuum pressure and (b) lateral earth pressure (Chai 2005). Chai et al. (2008) demonstrated the possibility of improving clayey deposits by combining the cap-drain with a vacuum and the surface or subsurface soil as a sealing layer, in lieu of an air tight sheet on the ground surface. However, the efficiency of the method depends on preventing the surface sand from being affected by the pressure from pervious layers of sand and discontinuities in the ground.

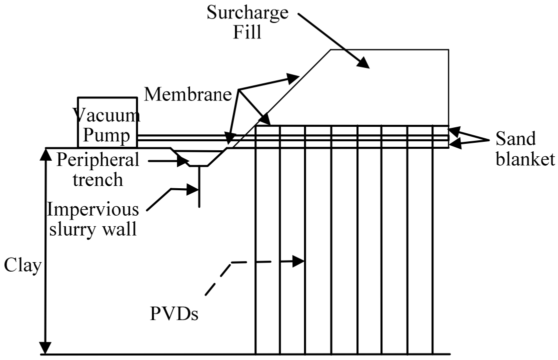


(a) (b)

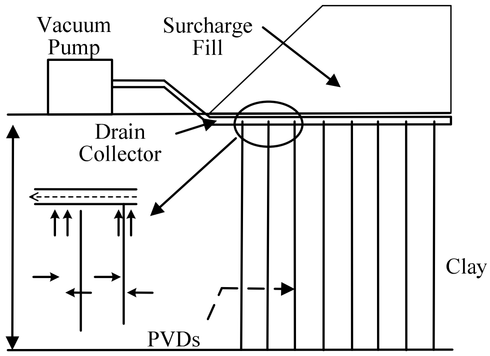
Figure 2: Consolidation process: (a) conventional loading； (b) idealised vacuum preloading (inspired by Indraratna et al. 2005c).

It is essential with vacuum assisted preloading that some horizontal drains be placed in transverse and longitudinal directions after installing the sand blanket, in order to uniformly distribute the surface suction. All vertical and lateral drains can then be connected to the edge of a peripheral Bentonite slurry trench, which is normally sealed by an impervious membrane system (Fig. 3a). The trenches can then be filled with water or Bentonite slurry to improve the intact sealing of the membrane around the boundary of the treated zone. The vacuum pumps are then connected to the prefabricated discharge system extending from the trenches (Fig. 3b). The suction head generated by the vacuum pump helps dissipate the excess pore water pressure via the PVDs.

When a reclaimed area has to be sub-divided into a number of sections to facilitate installation of the membrane (e.g. Port of Brisbane), vacuum preloading can only be effectively carried out in one section at a time. Vacuum preloading can become cumbersome over a very large area because the suction head may not be sustained due to the pressure of sand layers, or because it is too close to a marine boundary, in which case a cut off wall will often be used. An alternative method is to apply the vacuum directly to the individual PVD with flexible tubes without using a membrane. Here, each PVD is connected directly to the collector drain (Fig. 3b). Unlike the membrane system where an air leak can affect the entire PVD system, in this membrane-less system, each drain acts independently. However, installing an extensive tubing system for a large number of PVDs can increase the time and cost of installation (Seah 2006).



(a)



(b)

(c)

Figure 3: Vacuum-assisted preloading system: a) membrane system; (b) membrane-less system; and (c) vacuum at Port of Brisbane (Indraratna et al. 2005c).

Apart from the obvious inherent characteristics of these two vacuum systems, their effectiveness depends entirely on the properties of the soil, the thickness of the clay, the drain spacing, the type and geometry of PVDs, and the mechanical design and capacity of the vacuum pump. The selection and use of these systems are often based on empirical assessments that are invariably influenced by various aspects of the tender and/or the experience of the contractors, rather than detailed numerical studies. In this section, the analytical solutions to vertical drains incorporating the vacuum preloading of both systems (membrane and membrane-less) under time-dependent surcharge loading will be described.

The analytical modelling principles for both systems of the applied vacuum are shown in Fig. 4. The smear zone and well resistance are also considered in both models. The general solutions for excess pore water pressure, settlement, and degree of consolidation were derived through the Laplace transform technique. In addition, a time dependent surcharge preloading can be considered in lieu of an instantaneously applied constant load that can neither simulate the construction history of the embankment nor the variation of the applied vacuum pressure. The results elicited the need for correctly determining the permeability of the sand blanket in the membrane system, and the role of any variation in vacuum in the membrane-less system. The permeability of the sand blanket can significantly affect overall consolidation in the membrane system.

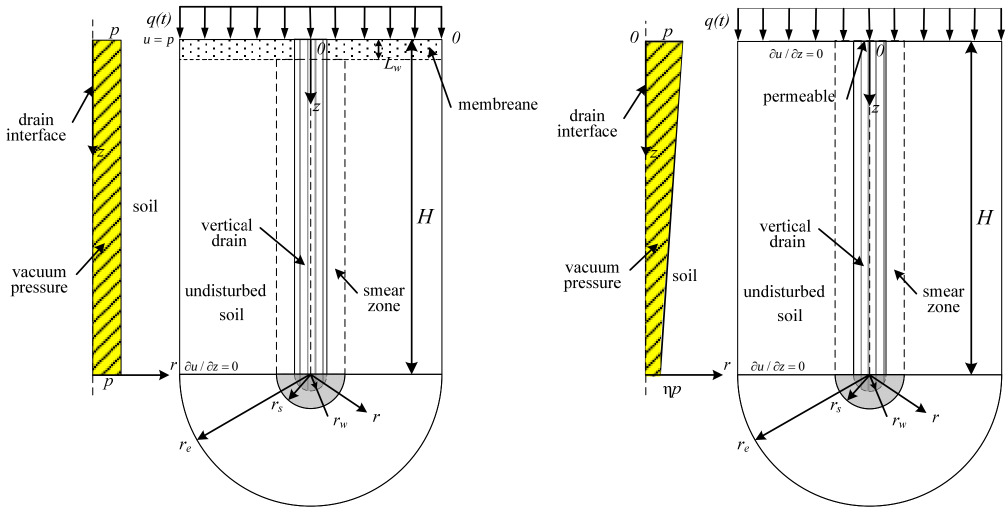


Figure 4: Analysis schemes of unit cell with vertical drain: (a) membrane system; and (b) membrane system.

With the Second Bangkok International Airport, various ground improvement schemes via PVDs with vacuum preloading and conventional surcharge preloading were studied by Indraratna and Redana (2000) and Indraratna et al. (2004). Consolidation with only surcharge preloading would take much longer than using vacuum preloading (Fig. 5). The corresponding settlements, excess pore water pressure, and lateral displacement, are presented in Figs. 5 and 6, respectively. As expected, the use of a vacuum pressure increased the rate of excess pore pressure dissipation. This is a direct result of an increased pore pressure gradient towards the drains due to negative pressure (suction) along the length of the drain. The use of sufficient vacuum pressure with properly sealed surface membranes accelerates the preconstruction settlement faster and better than the conventional surcharge preloading method. Moreover, a combination of embankment load with vacuum pressure can reduce the outward lateral displacement (Chai et al. 2005; Indraratna and Redana 2000).

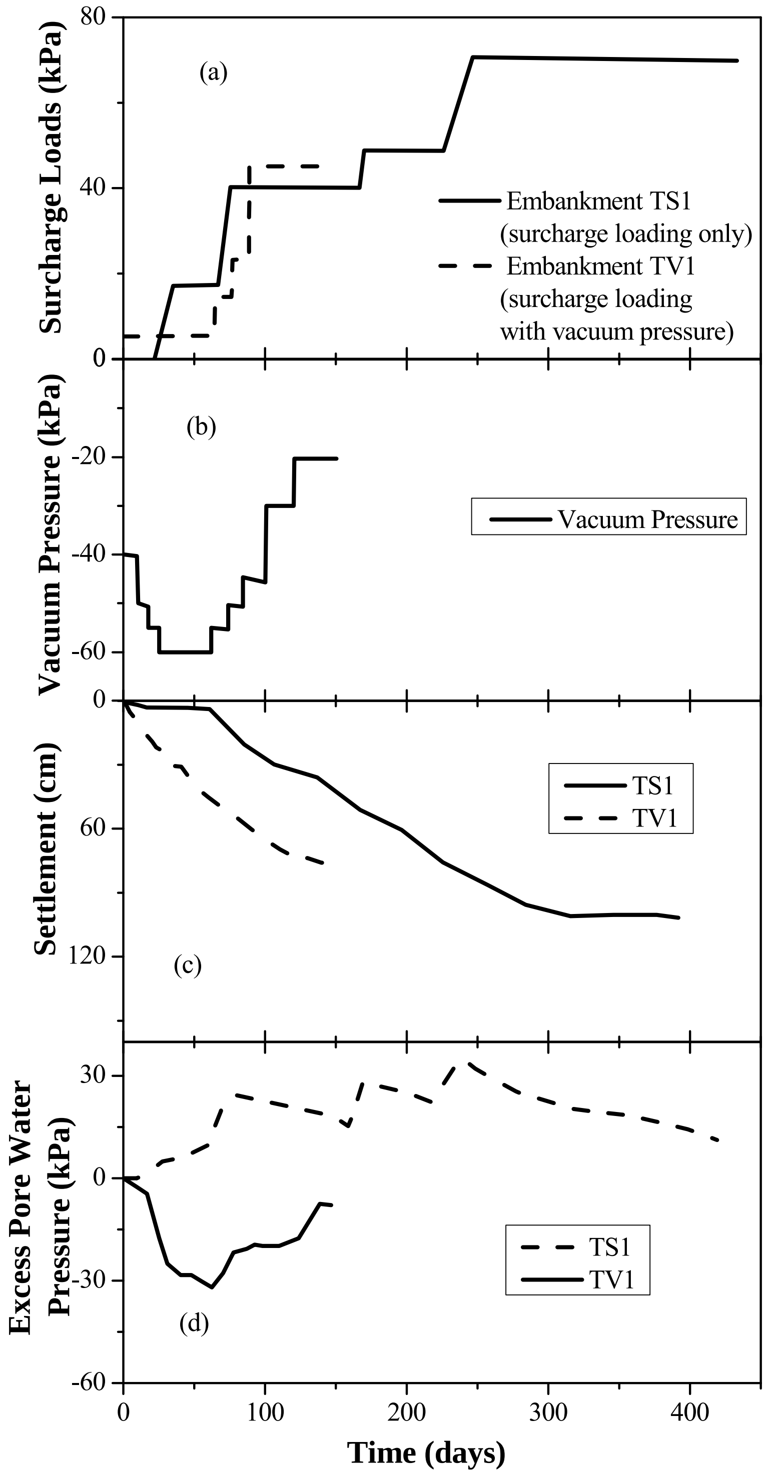


Figure 5: Second Bangkok International Airport (without vacuum pressure):(a) Construction loading history; (b) Surface settlement at the centreline of embankment; (c) Variation of excess pore-water pressure at 8m depth below ground level at the centreline for embankments (for surcharge only); (d) at 3m depth below ground level, 0.5 m away from the centreline for embankments (with surcharge and vacuum preloading together) (Indraratna et al. 2000, 2004).

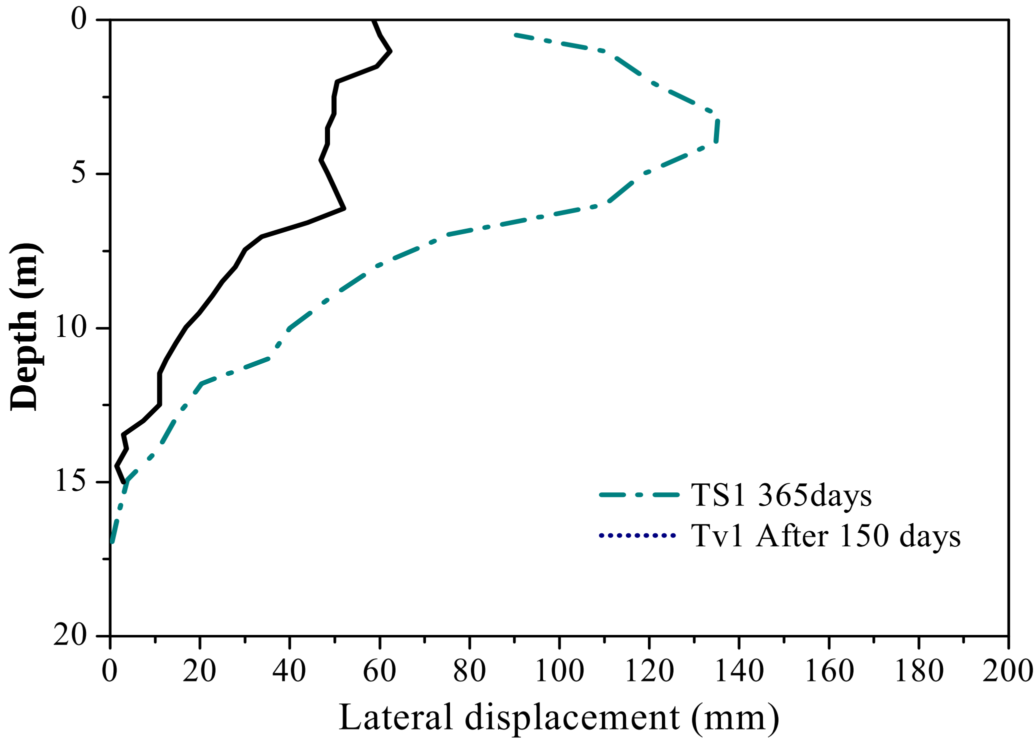


Figure 6: Lateral-displacement profiles 20m from the centreline of embankments (Indraratna et al. 2000, 2004).

3. VACUUM CONSOLIDATION THEORY

Mohamedelhassan and Shang (2002) developed a combined vacuum and surcharge load system and adopted the one-dimensional Terzaghi's consolidation theory (Figure 7). The mechanism for the combined vacuum and surcharge loading (Fig. 7a) may be determined by the law of superposition (Figs. 7b and 7c). The average of degree of consolidation for combined vacuum and surcharge preloading can then be expressed by:

$$U_{vc} = 1 - \sum_{m=0}^{\infty} \frac{2}{M} \exp^{-M^2 T_{vc}} \tag{1}$$

$$T_{vc} = c_{vc} t / H^2 \tag{2}$$

where T_{vc} is a time factor for combined vacuum and surcharge preloading, and c_{vc} is the coefficient of consolidation for combined vacuum and surcharge preloading.

Indraratna et al. (2004) showed that when a vacuum is applied in the field through PVDs, the suction head along the length of the drain may decrease with depth, thereby reducing its efficiency. Laboratory measurements taken at a few points along PVDs installed in a large-scale consolidometer at the University of Wollongong clearly indicated that the vacuum propagates immediately, but a gradual reduction in suction may occur along the length of the drain. The rate at which the suction develops in a PVD depends mainly on the length and type of PVD (core and filter properties). However, some field studies suggest that the suction any develop rapidly even if the PVDs are up to 30 m long (Bo et al. 2003; Indraratna et al. 2005a).

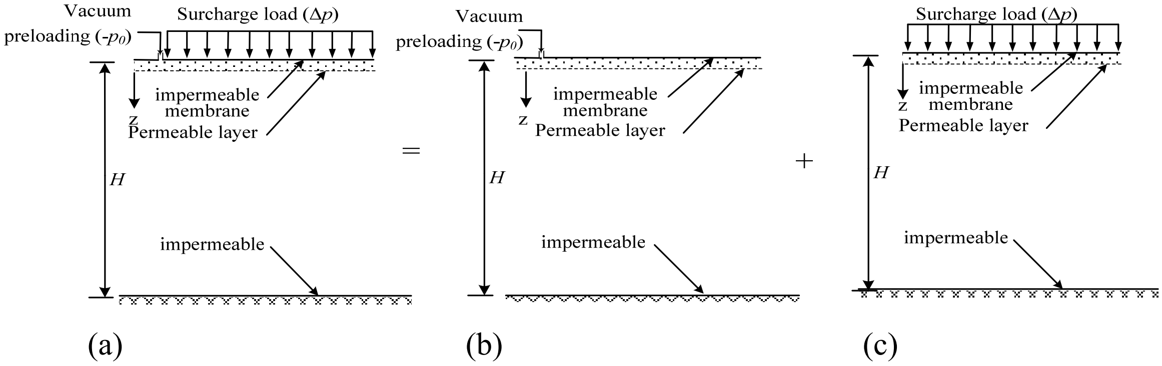


Figure 7: Schematic diagram of vacuum preloading system: (a) vacuum and surcharge combining load; (b) surcharge preloading; and (c) vacuum preloading (after Mohamedelhassan and Shang 2002).

Indraratna et al. (2004, 2005a) proposed a modified radial consolidation theory inspired by laboratory observations to include different distribution patterns of vacuum pressure (Fig. 8). These results indicated that the efficiency of the PVD depended on the magnitude and distribution of the vacuum. In order to quantify the loss of vacuum, a trapezoidal distribution of vacuum pressure along the length of the PVD was assumed.

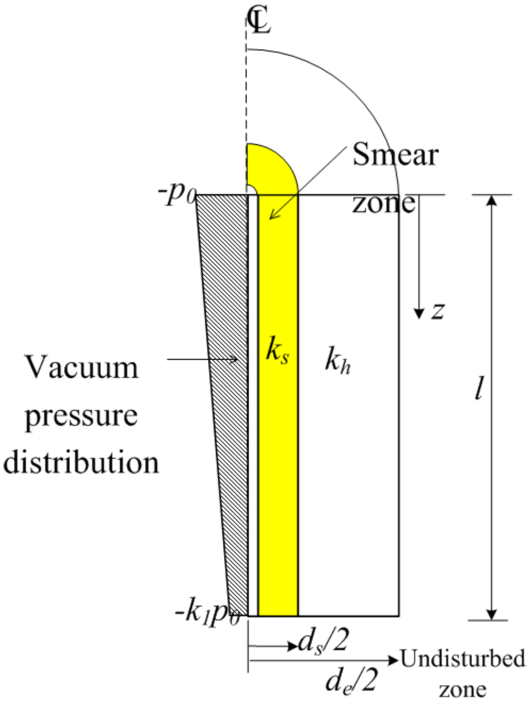


Figure 8: The distribution patterns of vacuum pressure in the vertical directions (after Indraratna et al. 2005a).

Based on these assumptions, the average excess pore pressure ratio $(R_u = \Delta p/\bar{u}_0)$ of a soil cylinder for radial drainage that incorporates vacuum preloading can be given by:

$$R_u = \left(1 + \frac{p_0}{\bar{u}_o}\frac{(1+k_1)}{2}\right)\exp\left(-\frac{8T_h}{\mu}\right) - \frac{p_0}{\bar{u}_0}\frac{(1+k_1)}{2} \tag{3}$$

and

$$\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right)\ln(s) - 0.75 + \pi z(2l - z)\frac{k_h}{q_w}\left\{1 - \frac{k_h/k_s - 1}{(k_h/k_s)(n/s)^2}\right\} \tag{4}$$

where p_0 = the vacuum applied at the top of the drain, k_1 = ratio between the vacuum at the top and bottom of the drain, $\bar{u}_0$ = the initial excess pore water pressure, k_h = the horizontal permeability coefficient of soil in the undisturbed zone, k_s = the horizontal permeability coefficient of soil in the smear zone, T_h = the time factor, n = the ratio d_e/d_w (d_e is the diameter of the equivalent soil cylinder = $2r_e$ and d_w is the diameter of the drain = $2r_w$), s = ratio d_s/d_w (d_s is the diameter of the smear zone = $2r_s$), z = depth, l = the equivalent length of drain, q_w = the well discharge capacity.

4. NUMERICAL ANALYSIS

4.1. Two-dimensional plane strain conversion

For multi-drain simulation, plane strain finite element analysis can be readily adapted to most field situations (Indraratna and Redana 2000; Indraratna et al. 2005a). Nevertheless, realistic field predictions require that the axi-symmetric properties to be converted to an *equivalent* 2D plane strain condition, especially the permeability coefficients and drain geometry. Plane strain analysis can also accommodate vacuum preloading in conjunction with vertical drains. Indraratna et al. (2005b) proposed an equivalent plane strain approach to simulate vacuum pressure for a vertical drain system with modification to the original theory introduced by Indraratna and Redana (1997), as shown in Fig. 9.

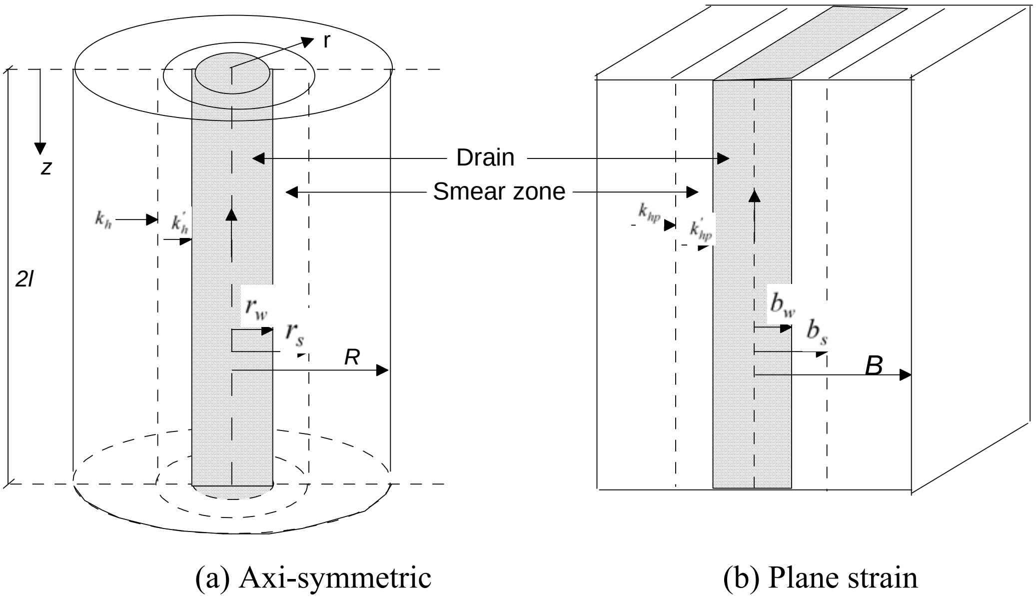


Figure 9: Conversion of an axi-symmetric unit cell into plane strain condition (Indraratna et al. 2005b).

Equivalent plane strain conditions can be fulfilled in three ways:

(1) Geometric approach where the PVD spacing varies, but the soil permeability remains constant;

(2) Permeability approach where the equivalent permeability coefficient is determined while the drain spacing remains unchanged;

(3) Combined permeability and geometric approach where plane strain permeability is calculated based on a convenient space between the drains.

Indraratna et al. (2005b) proposed an average degree of consolidation for plane strain by assuming that the plane strain cell (width of 2*B)*, the half width of the drain b_w and the half width of the smear zone b_s may be kept the same as their axi-symmetric radii r_w and r_s, respectively. This implies that $b_w = r_w$ and $b_s = r_s$ (Fig.9). To excess pore pressure can be determined form:

$$\frac{\bar{u}}{\bar{u}_0} = \left(1 + \frac{p_{0p}}{\bar{u}_o}\frac{(1+k_1)}{2}\right)\exp\left(-\frac{8T_{hp}}{\mu_p}\right) - \frac{p_{0p}}{\bar{u}_0}\frac{(1+k_1)}{2} \tag{5a}$$

and

$$\mu_p = \left[\alpha + (\beta) \frac{k_{hp}}{k'_{hp}} \right] \tag{5b}$$

where $\bar{u}_0$ = the initial excess pore pressure, $\bar{u}$ = the pore pressure at time t (average values) and T_{hp} = the time factor in plane strain, and k_{hp} and k'_{hp} are the equivalent undisturbed horizontal and corresponding smear zone permeability, respectively. The geometric parameters α and β are given by:

$$\alpha = \frac{2}{3} - \frac{2b_s}{B}\left(1 - \frac{b_s}{B} + \frac{b_s^2}{3B^2}\right) \tag{6a}$$

$$\beta = \frac{1}{B^2}(b_s - b_w)^2 + \frac{b_s}{3B^3}\left(3b_w^2 - b_s^2\right) \tag{6b}$$

At a given level of effective stress, and at each time step, the average degree of consolidation for the axi-symmetric ($\bar{U}_p$) and equivalent plane strain ($\bar{U}p, pl$) conditions are made equal.

By making the magnitudes of R and B the same, Indraratna et al. (2005a) presented a relationship between k_{hp} and k'_{hp}. The smear effect can be captured by the ratio between the smear zone permeability and the undisturbed permeability, hence:

$$\frac{k'_{hp}}{k_{hp}} = \frac{\beta}{\frac{k_{hp}}{k_h}\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k'_h}\right)\ln(s) - 0.75\right] - \alpha} \tag{7}$$

Ignoring the effects of smear and well resistance in the above expression would lead to the simplified solution proposed earlier by Hird et al. (1992):

$$\frac{k_{hp}}{k_h} = \frac{0.67}{[\ln(n) - 0.75]} \tag{8}$$

Indraratna et al. (2005b) compared two different distributions of vacuum along a single drain for the equivalent plane strain (2D) and axi-symmetric conditions (3D). Varying the vacuum pressure in PVDs installed in soft clay would be more realistic for long drains, but a constant vacuum with depth is justified for relatively short drains

5. CASE HISTORIES

5.1. Port of Brisbane

The Port of Brisbane is located at the mouth of the Brisbane River at Fisherman Islands. An expansion of the Port includes a 235ha area to be progressively reclaimed and developed over the next 20 years using dredged materials from the Brisbane River and Moreton Bay shipping channels. The site contains compressible clays of over 30 m in thickness. At least 7 m of dredged mud capped with 2 m of sand was used to reclaim the sub-tidal area. Generally, the complete consolidation of the soft deep clay deposits may well take in excess of 50 years, if surcharging was the only treatment, with associated settlements of probably 2.5-4 m likely. To reduce the consolidation period, the method of PVDS and surcharge or PVD combined with vacuum pressure (at sites where stability is of a concern) was chosen to be trialled. Three contractors undertook the trial works being Austress Menard, Van Oord and Cofra/Boskalis. All three trialled prefabricated drains and surcharge with Austress Menard and Cofra/Boskalis also trialling their respective proprietary membrane and membrane-less vacuum systems. Figure 10 shows the final layout of a typical trial area with the PVD design results for each area.

The typical soil properties are summarised in Table 1.

Table 1: Typical Soil Properties (Port of Brisbane Corporation and Austress Menard 2008).

Soil layer	Soil type	γ_t (kN/m^3)	$Cc/(1+e_0)$	C_v (m^2/yr)	C_h (m^2/yr)	$C\alpha/(1+e_0)$
1	Dredged Mud	14	0.3	1	1	0.005
2	Upper Holocene Sand	19	0.01	5	5	0.001
3	Upper Holocene Clay	16	0.18	1	2	0.008
4	Lower Holocene Clay	16	0.235	0.8	1.9	0.0076

To compare two locations with a different loading history, the lateral displacement normalised by the applied effective stress at two inclinometer positions (MS24 and MS34) are plotted in Fig. 11. It was clear that the lateral displacements were largest in the upper Holocene shallow clay depths and were insignificant below 10m. From this limited inclinometer data, the membrane-less BeauDrain system (MS34) had controlled the lateral displacement more effectively than the surcharge only section (MS24). Settlement and excess pore water pressure predictions and field data for a typical settlement plate (TSP3) are shown in Fig. 12. The predicated settlement curve agreed with the field data. The excess pore water pressures were more difficult to predict than settlement, but they did indicate a slower rate of dissipation in the Holocene clays in every section monitored, in spite of the PVDs. From the perspective of stability, the incremental rate of change of the lateral displacement/settlement ratio (μ) with time can be plotted as shown in Fig. 13. This rate of change of μ can be determined for relatively small time increments where a small and decreasing gradient can be considered to be stable with respect to lateral movement, while a continuously increasing gradient of μ reflects potential lateral instability. In Fig. 13, the gradient in the non-vacuum area WD3 increased initially, which could be attributed to the final surcharge loading placed quickly, while the clay was still at early stages of consolidation. However, as the PVDs become fully active and settlement increased at a healthy rate, the gradient of μ decreased, as expected. In general, Figure 13 illustrates that the vacuum pressure provides a relatively unchanging gradient of μ with time.

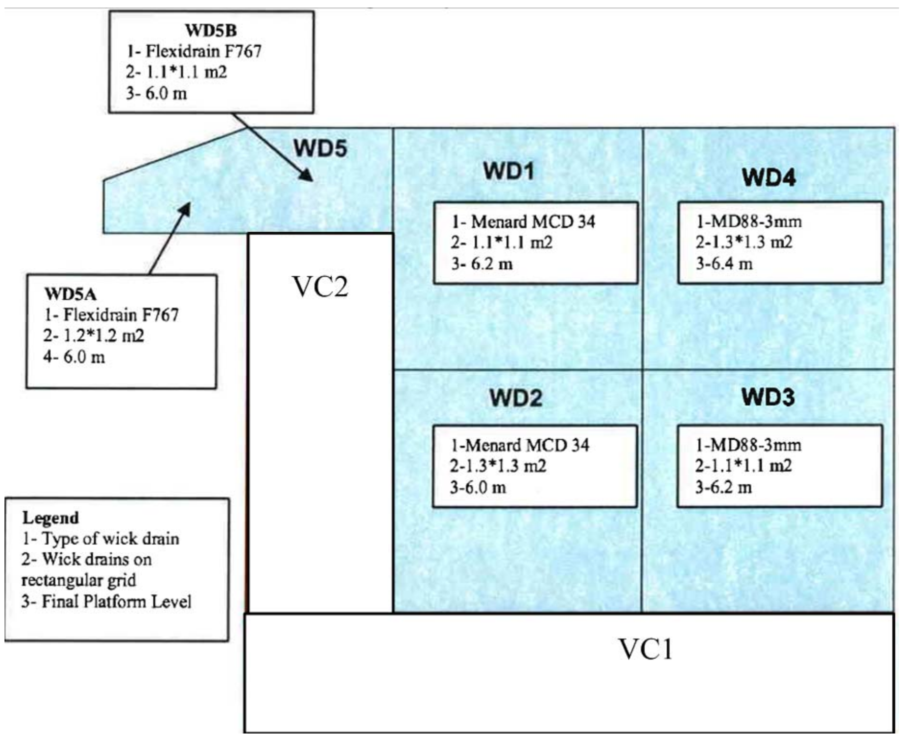


Figure 10: S3A Trial Area – Layout and detail design specifications (Port of Brisbane Corporation and Austress Menard 2008).

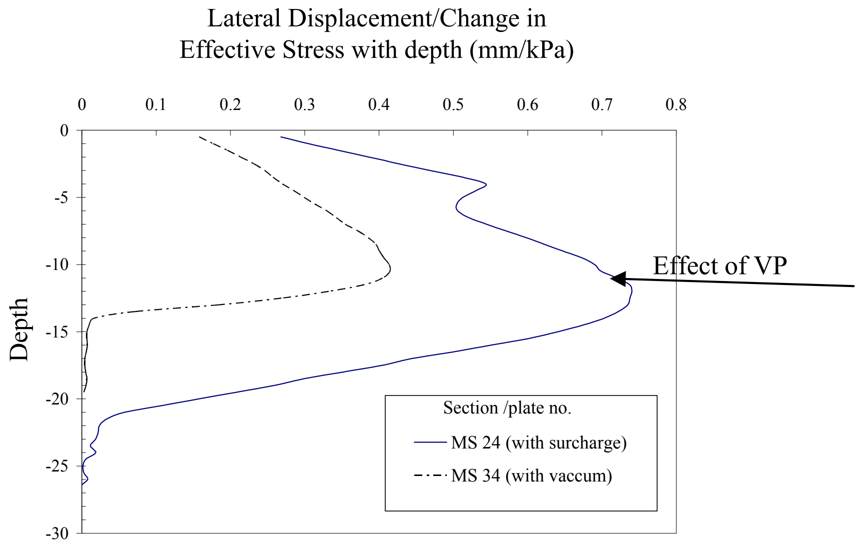


Figure 11: Comparison of lateral displacements in vacuum and non-vacuum areas.

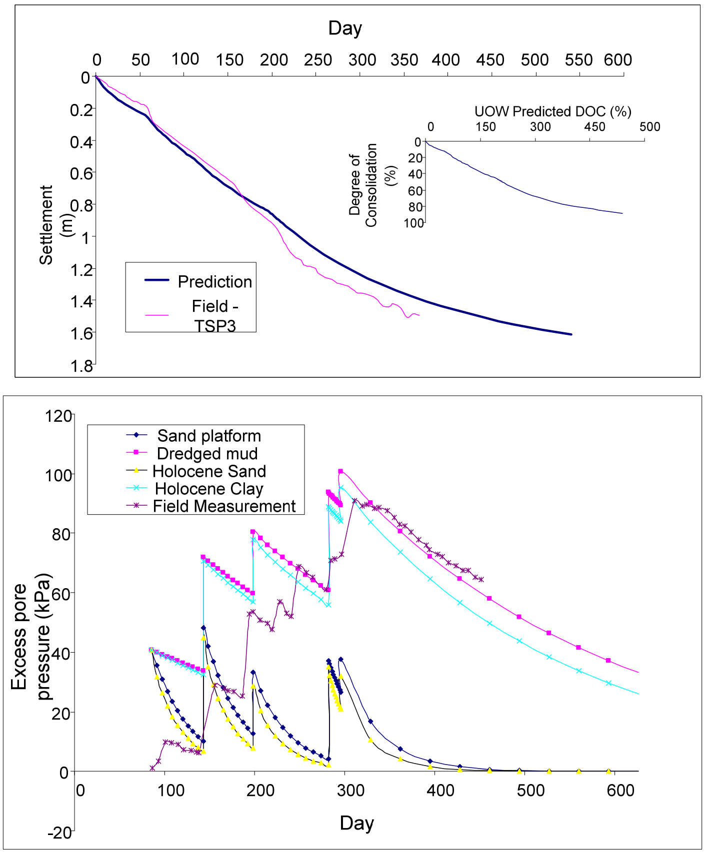


Figure 12: (a) Settlement; and (b) excess pore water pressure predictions and field data for a typical settlement plate location.

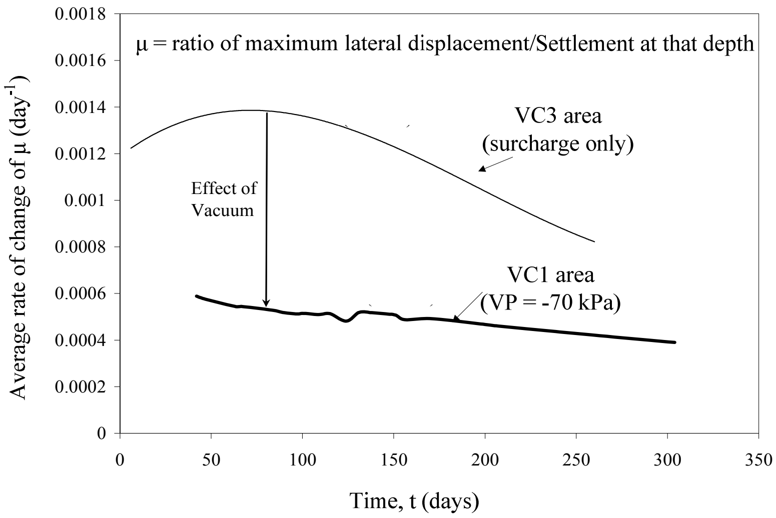


Figure 13: Rate of change of lateral displacement/settlement ratio with time.

Figure 14 provides approximately linear relationships between the long term residual settlement (RS) and the clay thickness of clay for an over-consolidation ratio (OCR) from 1.1 to 1.4, and for a degree of consolidation (DOC) that exceeding 80%. The reduced settlement (RS) was determined based on the theory secondary consolidation employing the secondary compression index C_α (Table 1). More detail on the computation of RS are given by Mesri and Castro (1987), Bjerrum (1972) and Yin and Clark (1994). As expected, when the OCR increased, the RS decreased substantially. In general, as the thickness of Holocene clay increased, the RS also increased. The corresponding regression lines and best–fit equations are also shown in Figure 14. In particular, the vacuum consolidation locations (VC1-2, VC2-2 and VC2-3) show a considerably reduced RS at an OCR approaching 1.4, which is well below the permissible limit of 250mm. At an OCR of approximately 1.3, the residual settlement associated with membrane-less BeauDrain consolidation (TA8) and membrane type (VC1-5) were also small.

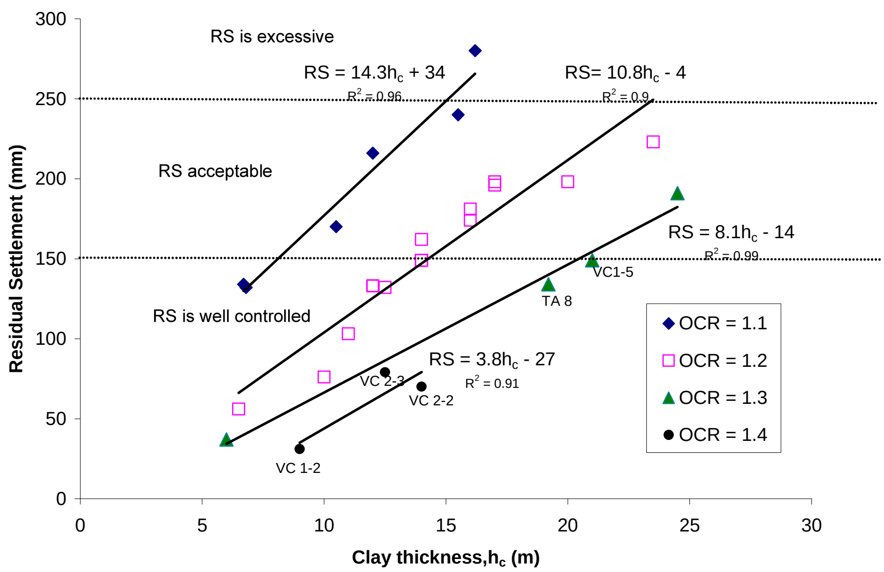


Figure 14: Effect of OCR and clay thickness on residual settlement.

5.2. Ballina Bypass

The Pacific Highway linking Sydney and Brisbane is constructed to reduce the high traffic congestion in Ballina. This bypass route has to cross a floodplain consisting of highly compressible and saturated marine clays up to 40 m thick. A system of vacuum assisted surcharge load in conjunction with PVDs was selected to shorten the consolidation time and stabilise the deeper clay layers. To investigate the effectiveness of this approach, a trial embankment was built north of Ballina, 34mm diameter circular PVD at 1.0m spacing were installed in a square pattern. The vacuum system consisted of PVDs with an air and water tight membrane, horizontal transmission pipes, and a heavy duty vacuum pump. Transmission pipes were laid horizontally beneath the membrane to provide uniform distribution of suction. The boundaries of the membrane were embedded in a peripheral trench filled with soil-bentonite to ensure absolute air tightness. Figure 15 presents the instrument locations, including surface settlement plates, inclinometers and piezometers. The piezometers were placed 1m, 4.5m, and 8m below the ground level, and eight inclinometers were installed at the edges of each embankment. The embankment area was then divided into Section A (no vacuum pressure), and Section B, subject to vacuum pressure and surcharge fill. As the layers of soft clay fluctuated between 7m to 25 m (Table 2), the embankment varied from 4.3m to 9.0m high, to limit the post-construction settlement. A vacuum pressure of 70 kPa was applied at the drain interface and removed after 400 days. The geotechnical parameter of the three subsoil layers obtained from standard odometer tests are given in Table 3.

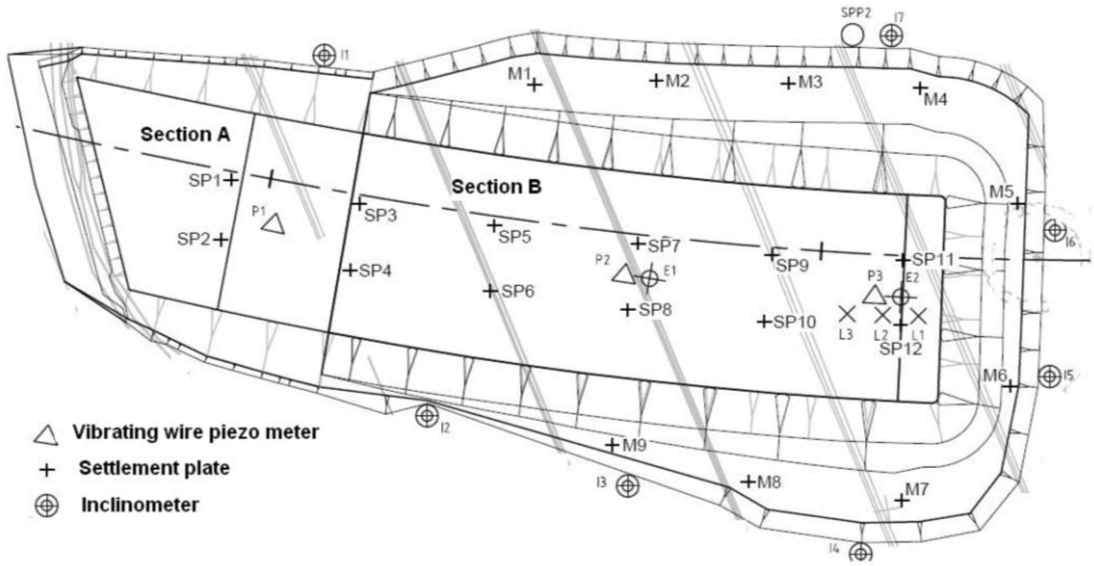


Figure 15: Instrumentation layout for the test embankments at Ballina Bypass (Indraratna et al. 2009).

Table 2: Bottom level of soft clay layer at each settlement plate (Indraratna et al. 2009)

Settlement plate	SP1 SP2	SP3 SP4	SP5 SP6	SP7 SP8	SP9 SP10	SP11 SP12
Bottom level of soft clay layer (m-RL)	2.7-6.7	6.7-9.7	9.7-11.7	11.7-14.7	14.7-17.7	20.7-24.7

Table 3: Soil parameters at SP12 (Indraratna et al. 2009).

Depth (m)	Soil Type	λ	κ	γ kN/m3	e_0	$k_{h,ax}$ 10^{-10} m/s	OCR
0.0-0.5	Clayey silt	0.57	0.06	14.5	2.9	10	2
0.5-15.0	Silty Clay	0.57	0.06	14.5	2.9	10	1.7
15.0-24.0	Stiffer Silty Clay	0.48	0.048	15.0	2.6	3.3	1.1

The soil profile with its relevant properties is shown in Fig. 16. A soft silty layer of clay approximately 10m thick was underlain by moderately stiff and silty layer clay located 10-30m deep, which was in turn underlain by firm clay. The groundwater was almost at the ground surface. The water content of the soft and medium silty clay varied from 80 to 120%, which was generally at or exceeded the liquid limit, ensuring that the soils were fully saturated. The field vane shear tests indicated that the shear strength was from 5-40 kPa. The compression index ($C_c / (1+e_0)$) determined by standard oedometer testing was between 0.30-0.50.

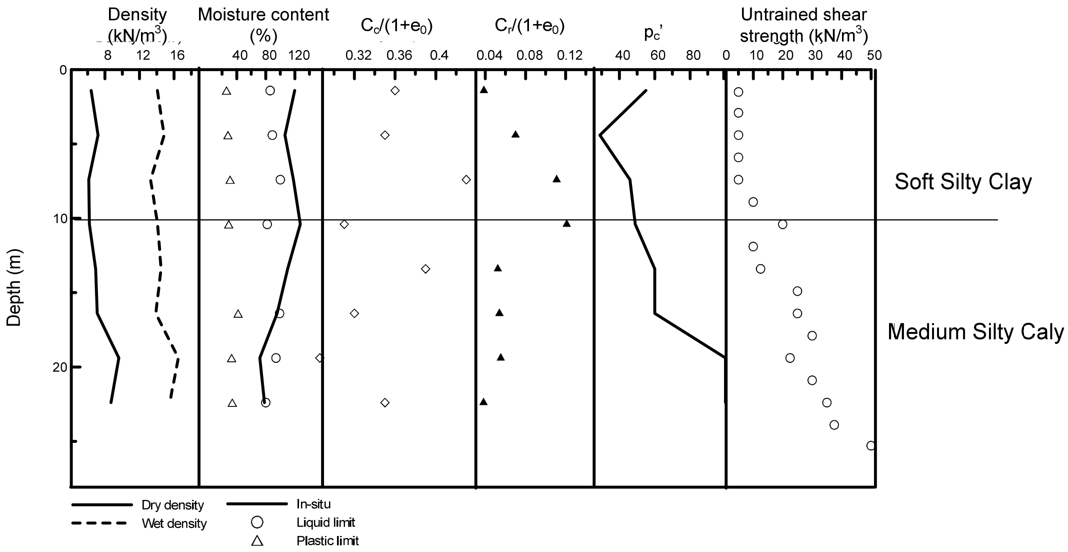


Figure 16: General soil profile and properties at Ballina Bypass (Indraratna et al. 2009).

Settlement and associated pore pressure recorded by the settlement plates and piezometers are shown in Fig. 17, with the embankment construction schedule. The actual suction varied from -70 kPa to -80 kPa, and no air leaks were encountered. Suction was measured by miniature piezometers embedded inside the drains.

Lateral displacement at the border of the embankments needed to be examined carefully, particularly the vacuum area where the surcharge loading was raised faster than that at the non-vacuum area. The soil properties and lateral displacement plots before and after vacuum are shown in Fig. 17. Inclinometer I1 was installed at the border of the vacuum area, whereas inclinometers I2-I4 were located at the edge of the vacuum area. Here the lateral displacement subjected to vacuum was smaller even though the embankments were higher. In Figure 17b the plot of lateral displacement normalised to embankment height showed that the vacuum pressure undoubtedly reduced lateral displacement.

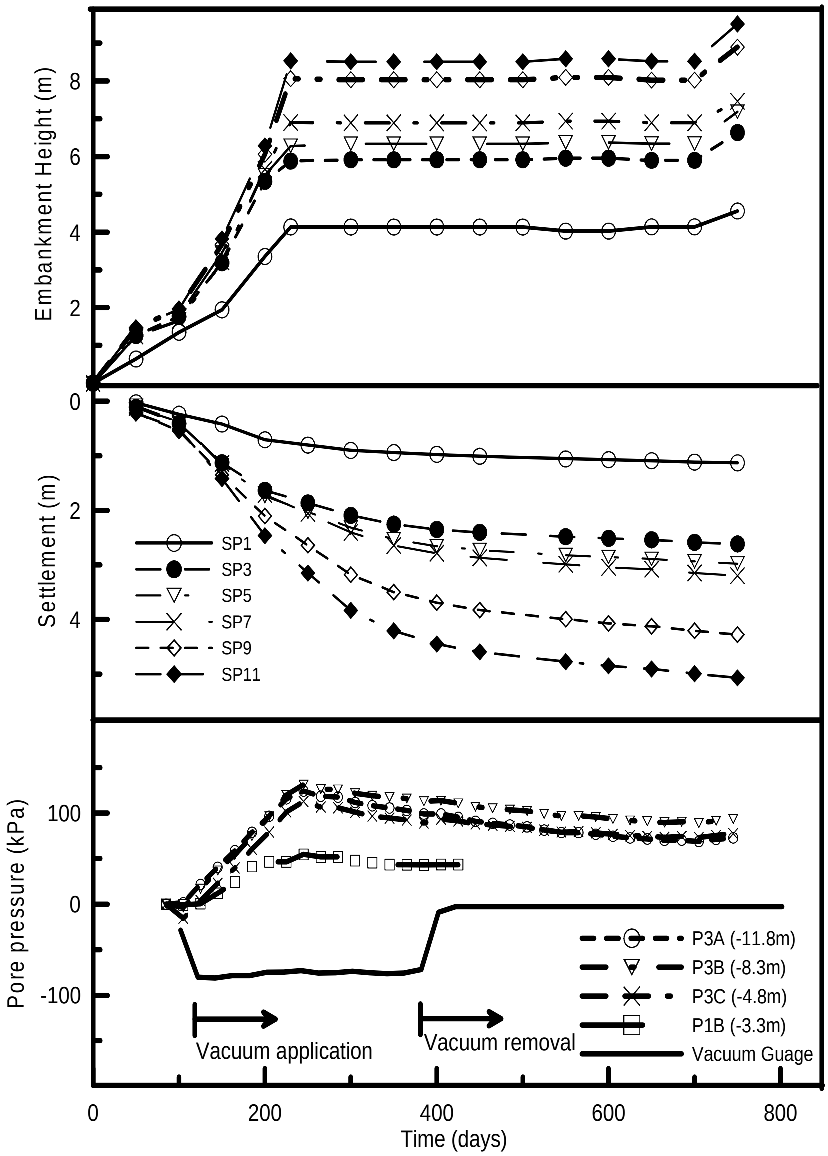
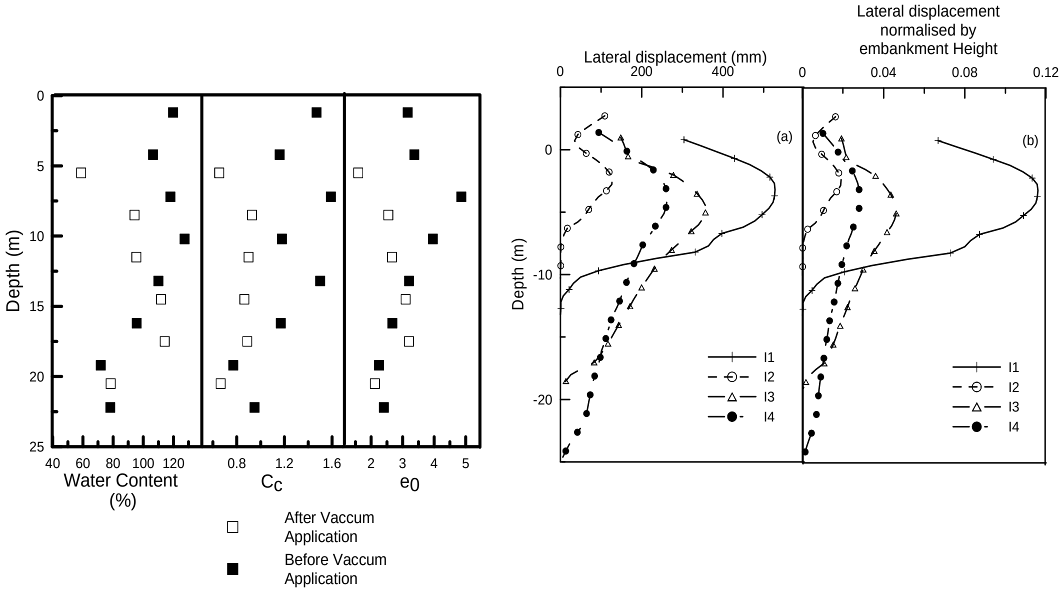


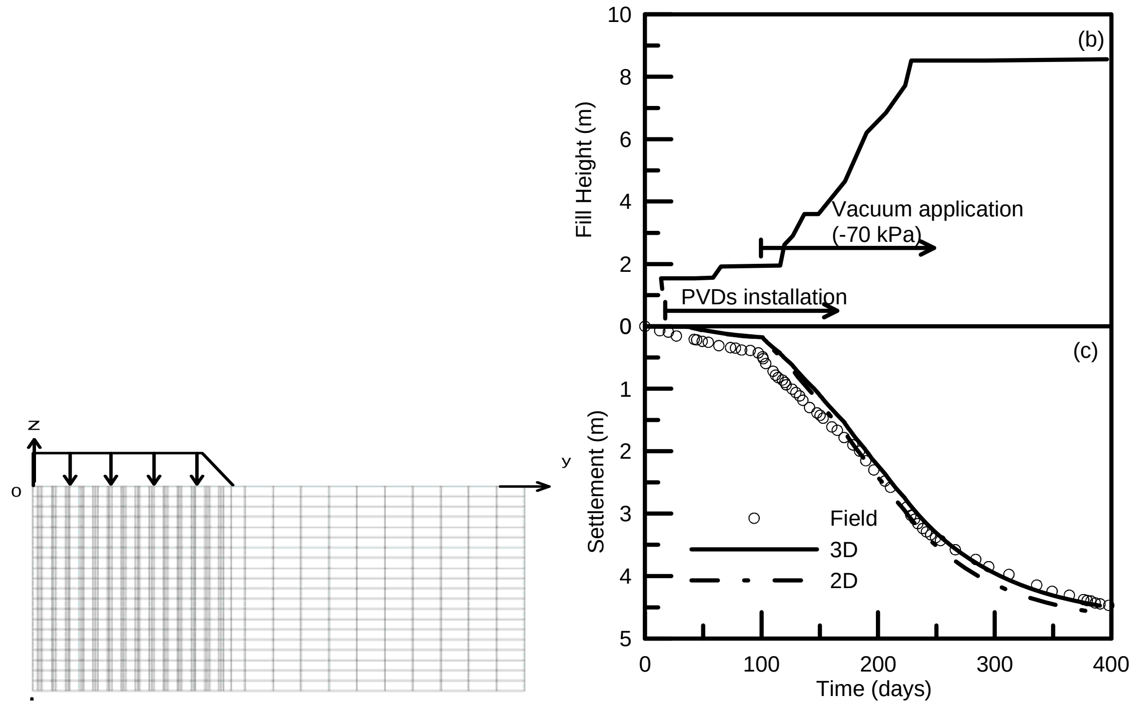
Figure 16: Embankment stage construction with associated settlements and excess pore pressures (Indraratna et al. 2009).

2D and 3D single drain analyses were used to compute the settlement at location SP-12. Typical 2D finite element mesh is shown in Fig. 18a. The construction history and measured settlement at the settlement plate SP-12 are shown in Fig. 18b. Here the clay was assumed to 24m thick, based on the CPT data. The analytical pattern was similar to Indraratna et al. (2005a). The predictions from 2D and 3D analyses agreed with the measured data, where the rate of settlement increased significantly after a vacuum was applied (Fig. 18c).



(a) (b)

Figure 17: (a) Soil properties before and after vacuum application; and (b) Measured lateral displacement and lateral displacement normalised with embankment height (Indraratna et al. 2009).



(a)

Figure 18: (a) 2D Finite element mesh; (b) loading history; and (c) consolidation settlements for settlement plate SP-12 (Indraratna et al. 2009).

5.3. Sunshine Coast

The Sunshine Coast is one of Australia's fastest growing regions and the continued economic and population growth has increased the pressure on the region's main traffic corridor, the Sunshine Motorway. Site investigation at the proposed development route revealed that the subsoil consists of highly compressible, saturated marine clays of high sensitivity. In order to evaluate the effective ground improvement techniques, a fully instrumented trial embankment was constructed in 1992 and monitored by the Queensland Department of Main Roads (QDMR), Brisbane.

The subsoil conditions are relatively uniform throughout the site, consisting of silty or sandy clay about 10-11m thick, overlying a layer of dense sand approximately 6m thick. The value of $\lambda/(1+e_0)$ of the subsoil varies from 0.19 to 0.63, and the value of $\kappa/(1+e_0)$ was found to be about 10 times smaller than the compression index (QDMR, 1991, 1992).

The base area of the trial embankment was approximately 90m × 40m and incorporated 3 separate sections (Fig. 19a), identified as Sections A, B, and C, respectively. Sections A and B (each 35m in long) represented the zones of prefabricated vertical drains (installed at 1m intervals) and 'no drains' respectively. Vertical prefabricated drains were installed at a spacing of 2m in Section C. These prefabricated vertical drains (Nylex Flodrain, 100×4mm^2) in Sections A and C were installed in a triangular pattern.

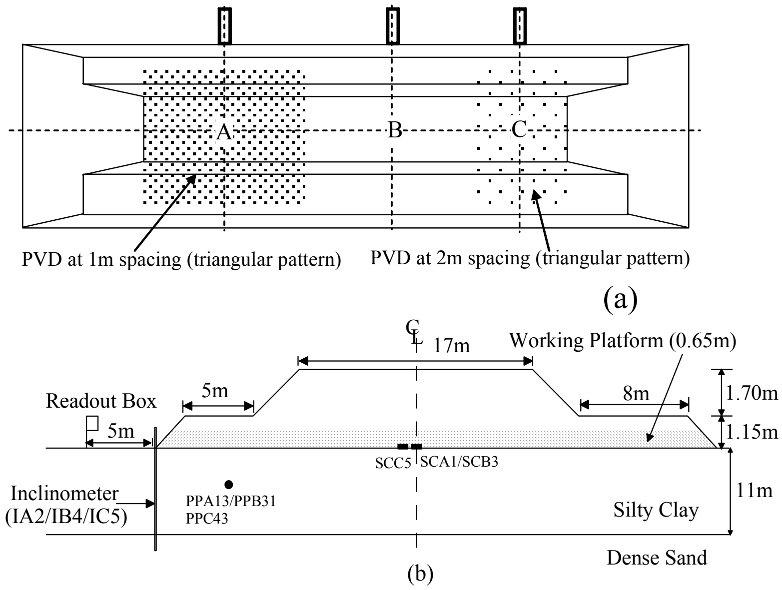


Figure 19: (a) Plan view of trial embankment, and (b) Typical cross-section of embankment with selected instrumentation points (adapted from QDMR, 1992)

A working platform 0.65m thick (500mm thick drainage layer composed of 7mm size gravel, plus 150mm of selected fill) was placed on top for construction traffic access. Prefabricated vertical drains (PVD) were installed from the working platform to a depth of 11m at Sections A and C. The embankment was constructed in stages using a loosely compacted granular material ($\gamma_t \approx 19$ kN/m^3) up to a height of 2.3m. Two berms, 5m in width on the instrumented side and 8m wide on the other side (Fig. 19) were constructed to increase the stability of the embankment. Half of the cross-section was intensively instrumented to capture the foundation response upon loading. A typical cross-section of the embankment with selected instrumentation points is shown in Fig. 19b. In this paper, the deformation and pore water pressure responses below Sections A, B and C were predicted using a plane strain finite element analysis and then compared with the available field data.

The multi-drain plane strain analysis was carried out using the finite element code PLAXIS, where the soil layers were divided into many elements (Sathananthan et al. 2008). In this paper, the soil layer close to the surface was modelled using the modified Cam-clay (MCC) properties determined by QDMR (1991). Given that the soil beneath the surface (up to 2.5m depth) is only lightly overconsolidated based on the results obtained from standard oedometer test (OCR=1.6), the authors have assumed that the application of MCC parameters for this soil layer is valid. Britto and Gunn (1987) and others clarify the validity of MCC for both normally consolidated and lightly overconsolidated clays. It is noted that for situations where the surface soil is heavily overconsolidated (compacted crust), the use of MCC parameters is inappropriate. The soft soil model based on Modified Cam-clay theory was also used to analyse the behaviour of normally consolidated clay layers beneath crust. Based on the oedometer test results on vertical and horizontal samples, the horizontal permeability (k_h) is approximately 2 times the vertical permeability (k_v) for all soil layers. For the embankment material which is silty sand, the Mohr-Coulomb model was used.

The finite element (PLAXIS) mesh contains 15-node triangular elements. The entire width of embankment was modelled because the loading was not symmetrical. The prefabricated vertical drains were modelled with zero thickness drain elements (the excess pore pressure along this element is assumed to be zero). The smear zone was modelled with the same modified Cam-clay properties ($\lambda/(1+e_0)$, $\kappa/(1+e_0)$, M) as the adjacent zone except for the reduced coefficient of lateral permeability. The locations of instruments including settlement gauges, piezometers and inclinometers were conveniently placed in the mesh in such a manner that the measuring points coincided with the mesh nodes. Only 11m depth of the foundation was considered due to the existence of the dense sand layer (below the overlying soft clay layer), which was stiff enough to assume a non-displacement boundary. Both the top (open boundary) and bottom surfaces of the subsoil foundation were assumed to be free draining and the water table coincided with the ground surface.

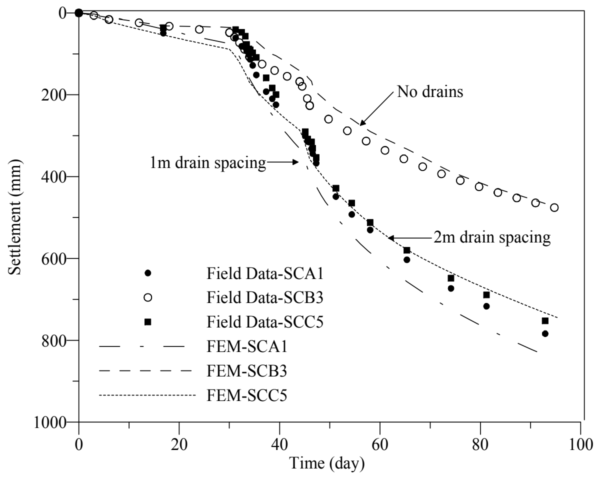


Figure 20: Centreline settlement of Sections A, B and C (Sathananthan et al. 2008).

The settlement gauges under Sections A, B and C, namely, SCA1, SCB3 (both under the centreline) and SCC5 (1m to the left of centreline) were selected for the purpose of comparing the field data with the numerical results. The predicted and measured surface settlements are illustrated in Fig. 20, which shows that the predicted values are in good agreement with the field data for Sections B and C. As expected, the rate of settlement increases due to the PVDs, where the plotted time-settlement at Section B (no drain) is only about 60% of that at Section A (drains at 1m spacing). This proves that the installation of vertical drains significantly decreases the consolidation time for a given settlement. Generally, the settlement rate is expected to be sensitive to drain spacing, but in this study, the difference in settlement-time plots for Sections A and C is small. The installation of closely spaced drains at Section A causes greater smear compared to that at Section C as well as the possible lateral variation of soil properties. In the finite element analysis, the soil properties are assumed constant along any lateral plane. For example, the total width of smear zone at Section A (drains @1m spacing) is about 18.9m compared to 9.7m at Section C (drains@2m spacing). This demonstrates that reducing the drain spacing excessively may only provide a marginal advantage due to increased smear.

Lateral deformation measured by the 3 inclinometers (IA2, IB4 and IC5) installed at the toe of the 5m wide berm at Sections A, B and C are compared with the numerical predictions in Fig. 21. As expected, the vertical drains significantly curtail the lateral deformation. For example, at 1m below the surface, the PVDs installed at 1m spacing (Section A) reduced the lateral displacement by 21% compared to Section B (no drains) and by 6% compared to Section C (drains installed at 2m spacing). Data for Section C indicate that there is not much reduction of lateral displacement below the crust as compared with Section B. These results also indicate that the predicted lateral displacement represents an acceptable match with the field data, but in some plots, a noticeable discrepancy is found approaching the ground surface within the upper most, lightly overconsolidated silty clay (OCR=1.6). For Section A plots (after 100 days) near the surface, the observed field displacements are larger than the predictions, even though the soil is lightly overconsolidated. This may suggest that the closely spaced drains at 1m spacing may have caused excessive smear. This is also supported by the observed excess pore water pressure as discussed below.

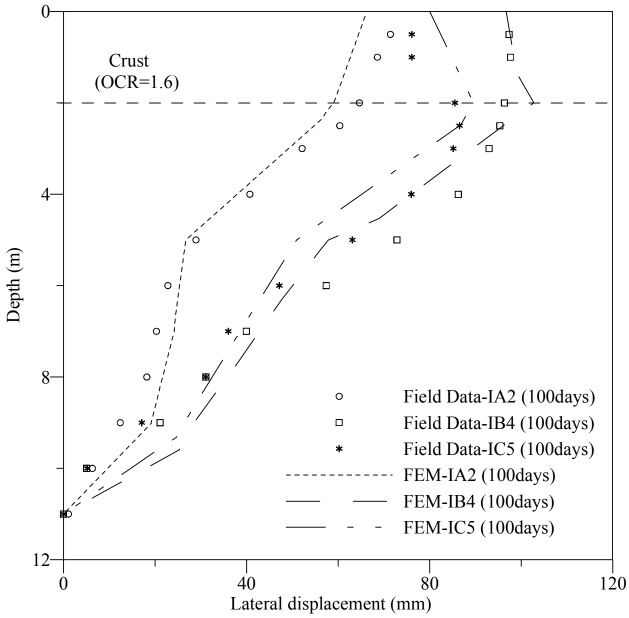


Figure 21: Lateral displacement profiles at the toe 5m berm of the embankment sections (Sathananthan et al. 2008).

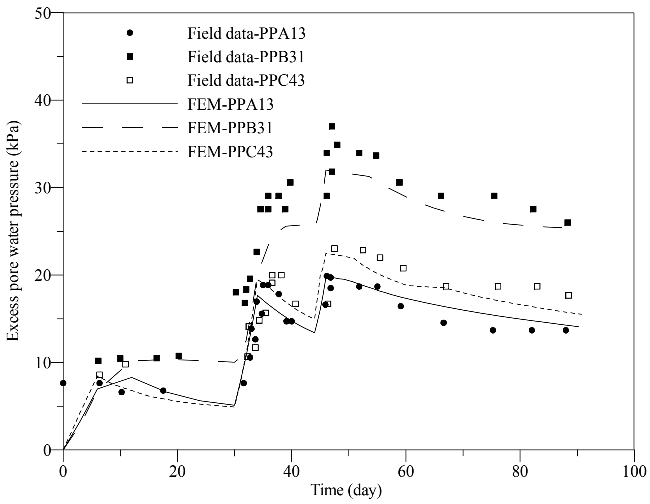


Figure 22: Excess pore pressure variation with time beneath the middle of the berm 5m (Sathananthan et al. 2008).

The predicted and observed variations of excess pore pressure at selected points beneath the middle of the berm are shown in Fig. 22. The selected pneumatic piezometers PPA13, PPB31 and PPC43 were installed at a depth of 5.0m at Sections A, B and C, respectively. Fig. 22 shows that significant excess pore pressures were generated due to embankment loading, and the predictions are in very good agreement with the field data. As expected, the induced excess pore pressure at Section B (no drains) is significantly higher (approximately 30%) than the other sections. Surprisingly, it is observed that at Section A where the drains are closely spaced, the excess pore water pressure is either slightly higher or nearly the same as at Section C before 40 days. After 40 days, rates of excess pore pressure dissipation in Sections A and C are the same. This can be attributed to excessive smear at Section A causing retarded pore pressure dissipation.

6. CONCLUSION

It is clear that the application of PVDs combined with vacuum and surcharge preloading has become common practice, and is now considered to be one of the most effective ground improvement techniques. Analytical and numerical modelling of vacuum preloading is still a developing research area. There has always been a discrepancy between the predictions and observed performance of embankments stabilised with vertical drains and vacuum pressure. This discrepancy can be attributed to numerous factors such as the uncertainty of soil properties, the effect of smear, inaccurate assumptions of soil behaviour and vacuum pressure distribution, and an improper conversion of axi-symmetric condition to plane strain (2D) analysis of multiply drains.

Vacuum assisted consolidation is an innovative method which has recently, and successfully, been used for large scale projects on very soft soils in reclamation areas. The extent of surcharge fill can be decreased to achieve the same amount of settlement and the lateral yield of the soft soil can be controlled by PVDs used in conjunction with vacuum pressure. The effectiveness of this system depends on (a) the air tightness of the membrane, (b) the seal between the edges of the membrane and the ground surface, and (c) the soil conditions and location of the ground water level. The exact role of membrane type and membrane-less systems for vacuum preloading requires detailed evaluation. In the absence of a comprehensive and quantitative analysis, the study of suitable methods to apply vacuum preloading becomes imperative, experimentally, numerically and in the field.

Analytical modelling of vertical drains that include vacuum preloading under axi-symmetric and plane strain conditions that simulate the consolidation of a unit cell surrounding a single vertical drain has been developed. The effects of vacuum propagation along the length of a drain and the occurrence of non-Darcian flow conditions have been incorporated in the proposed solutions to obtain a more realistic prediction. The elliptical cavity expansion theory provided further insight to evaluate the role that smear zone characteristics play on consolidation.

In large construction sites where many PVDs are installed, a 2D plane strain analysis is usually sufficient. The proposed conversion from axi-symmetric to a plane strain condition agreed with the data available from case histories, including Ballina Bypass and Port of Brisbane (Australia). These simplified plane strain methods can be readily incorporated in numerical (FEM) analysis. The conversion procedure from 3D to 2D based on the correct transformation of permeability and vacuum pressure, ensure that the time-settlement curves are the same as the true 3D analysis. Field behaviour and model predictions indicate that the efficiency of vertical drains depends on the magnitude and distribution of vacuum pressure as well as the degree of drain saturation during installation, and the extent of smear caused by the vertically penetrating mandrel.

7. ACKNOWLEDGEMENT

I gratefully appreciate at least a dozen PhD students, whom I have had the pleasure of supervising over the past 15 years, have contributed to the contents of this paper as reflected by the cited references. In particular, I am indebted to Dr Wayan Redana (laboratory evaluation of smear effects), Dr Chamari Bamunawita (numerical modeling and laboratory assessment of vacuum pressure propagation), Dr Iyathurai Sathananthan (non-Darcian flow effects on radial consolidation), Dr Rohan Walker (influence of overlapping smear effects), and Dr Cholachat Rujikiatkamjorn (optimization of PVD and vacuum preloading in design).

A number of research projects on the application of vertical drains and vacuum preloading have been supported in the past and present by the Australian Research Council (ARC). I owe my gratitude to Mr Vasantha Wijeyakulasuriya, Director, Queensland Department of Main Roads (Brisbane) for his continuous support through several such projects over a decade. Indeed, the keen collaborations with industry through numerous projects have facilitated the application of theory to practice. In this respect, I sincerely thank Queensland Department of Main Roads, Port of Brisbane Corporation, Roads and Traffic Authority, Coffey Geotechnics, Polyfabrics, Geofabrics, ARUP, Douglas Partners, Snowy Mountains Engineering Corporation, RailCorp, ARTC, Chemstab, Queensland Rail and Austress Menard.

Much of the contents in this paper are elaborated in numerous issues of the Canadian Geotechnical Journal, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Geotechnique, ASCE J. of Geomechanics, Ground Improvement Case Histories, Elsevier (editors: by Indraratna and Chu) and several Keynote papers at international conferences. Selected contents from some of these articles were reproduced with kind permission.

REFERENCES

Bergado, D. T., Balasubramaniam, A. S., Fannin, R. J. and Holta, R. D. 2002. Prefabricated vertical drains (PVDs) in soft Bangkok clay: a case study of the new Bangkok International Airport Project, Canadian Geotechnical Journal. 39: 304-315.

Bo, M. W., Chu, J., Low, B. K., and Choa, V. 2003. Soil improvement; prefabricated vertical drain techniques, Thomson Learning, Singapore.

Chai, J. C., Cater, J. P., and Hayashi, S. 2005. Ground deformation induced by vacuum consolidation. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 131(12): 1552-1561.

Choa, V. 1990. Soil improvement works at Tianjin East Pier project. Proceedings 10th Southeast Asian Geotechnical Conference, Taipei, 1: 47-52.

Chu, J., and Yan, S.W. 2005. Application of vacuum preloading method in soil improvement project. Case Histories Book, Edited by Indraratna, B. and Chu, J., Elsevier, London. Vol. 3: 91-118.

Chu, J., Yan, S. W., and Yang, H. 2000. Soil improvement by vacuum preloading method for an oil storage station. Geotechnique, 50(6): 625-632.

Holtan, G.W. 1965. Vacuum stabilization of subsoil beneath runway extension at Philadelphia International Airport. In Proc. of 6th ICSMFE, 2.

Indraratna, B., Balasubramaniam, A. S. and Balachandran, S. 1992. Performance of test embankment constructed to failure on soft marine clay. Journal of Geotechnical Engineering, ASCE, 118(1): 12-33.

Indraratna, B., and Redana, I. W. 2000. Numerical modeling of vertical drains with smear and well resistance installed in soft clay. Canadian Geotechnical Journal, 37: 132-145.

Indraratna, B., Bamunawita, C., and Khabbaz, H. 2004. Numerical modeling of vacuum preloading and field applications. Canadian Geotechechnical Journal, 41: 1098-1110.

Indraratna, B., Rujikiatkamjorn C., and Sathananthan, I. 2005a. Analytical and numerical solutions for a single vertical drain including the effects of vacuum preloading. Canadian Geotechnical Journal, 42: 994-1014.

Indraratna, B., Sathananthan, I., Rujikiatkamjorn C. and Balasubramaniam, A. S. 2005b. Analytical and numerical modelling of soft soil stabilized by PVD incorporating vacuum preloading. International Journal of Geomechanics, 5(2): 114-124.

Indraratna, B., Rujikiatkamjorn C., Balasubramaniam. A. S. and Wijeyakulasuriya, V. 2005c. Predictions and observations of soft clay foundations stabilized with geosynthetic drains vacuum surcharge. Ground Improvement – Case Histories Book (volume 3), Edited by Indraratna, B. and Chu, J., Elsevier, London: 199-230.

Indraratna, B., and Rujikiatkamjorn, C., Kelly, R., and Buys, H. 2009. Soft soil foundation improved by vacuum and surcharge preloading at Ballina Bypass, Australia. International Symposium on Ground Improvement Technologies and Case Histories (ISGI09): 95-105.

Kjellman W. 1952. Consolidation of clayey soils by atmospheric pressure. Proceedings of a Conference on Soil Stabilisation, MIT, Boston, pp. 258–263.

Jacob, A., Thevanayagam, S. and Kavazajian, E., 1994. Vacuum-assisted consolidation of a hydraulic landfill. Vertical and Horizontal Deformations of Foundations and Embankments, Settlement'94, College Station. Geotechnical Special Publications No 40, ASCE: 1249-1261.

Jamiolkowski, M., Lancellotta, R., and Wolski, W. 1983. Precompression and speeding up consolidation. Proc. 8th ECSMFE: 1201-1206.

Johnson, S. J., 1970. Precompression for improving foundation soils. Journal of the Soil Mechanics and

Foundations Division, ASCE, 1: 111-114.

Mohamedelhassan E., and Shang, J.Q. 2002. Vacuum and surcharge combined one-dimensional consolidation of clay soils, Canadian Geotechnical Journal, 39: 1126-1138.

Port of Brisbane Corporation and Austress Menard (2008), personal communication internal report and confidential, 79p.

Queensland Department of Main Roads. (1991). Sunshine Motorway Stage 2 – Area 2 Geotechnical Investigation. Materials and Geotechnical Services Branch, June, R1765.

Queensland Department of Main Roads. (1992). Sunshine Motorway Stage 2 – Interim Report on the Performance of the Trial Embankment Area 2A (Ch 28490-28640). Materials and Geotechnical Services Branch, July, R1802.

Richart F.E. 1957. A review of the theories for sand drains. Journal of the Soil Mechanics and Foundations Division, ASCE, 83(3): 1-38.

Sathananthan, I., Indraratna, B., and Rujikiatkamjorn C., (2008). The evaluation of smear zone extent surrounding mandrel driven vertical drains using the cavity expansion theory. International Journal of Geomechanics, ASCE. 8(6), 355-365.

Seah, T.H. 2006. Design and construction of ground improvement works at Suvarnabhumi Airport. Geotechnical Engineering Journal of the Southeast Asian Geotechnical Society, 37: 171-188.

Yan, S.W. and Chu, J. 2003. Soil improvement for a road using a vacuum preloading method. Ground Improvement, 7(4): 165-172.