Current State of the Art in Vacuum Preloading for Stabilising Soft Soil

Cholachat Rujikiatkamjorn[1] and Buddhima Indraratna[2]
Centre for Geomechanics and Railway Engineering, Faculty of Engineering, University of Wollongong, Wollongong City, NSW 2522, Australia; ARC Centre of Excellence in Geotechnical Science and Engineering, Australia
[2] *E-mail*: indra@uow.edu.au

ABSTRACT: In this paper the analytical solutions for radial consolidation that include time dependent surcharge loading and vacuum pressure are proposed, whilst also considering the impact of the parabolic variation of permeability in the smear zone. The use of the spectral method for multilayered soil consolidation is introduced and verified. The Elliptical Cavity Expansion Theory is used to predict the extent of soil disturbance (smear zone) caused by the installation of mandrel driven vertical drains. The predicted smear zone is then compared to the data obtained from large-scale radial consolidation tests. Furthermore, the advantages and limitations of applying a vacuum through vertical drains are discussed using the proposed solutions. The vacuum pressure applied generates a negative pore water pressure that increases the effective stress within the soil, which leads to an accelerated consolidation. Vacuum pressure is modelled as a distributed negative pressure (suction) along the length of the drain and across the surface of the soil. Analytical and numerical analyses that incorporate the Authors' equivalent plane strain solution are conducted to predict the excess pore pressures, lateral and vertical displacement. The application of the theoretical models for selected case histories at the site of the 2nd Bangkok International Airport and the Port of Brisbane, are discussed and analysed. The predictions are compared with the available field data and show that the proposed model can be confidently used to predict the performance with acceptable accuracy through rigorous mathematical modelling and numerical analysis. The research findings verify that the role of the smear zone and vacuum distribution can significantly affect the consolidation of soil, but these aspects need to be modelled appropriately to obtain reliable.

KEYWORDS: Analytical modelling; case history; numerical modelling; vacuum consolidation; vertical drain

1. INTRODUCTION

Soft clay deposits usually possess a low bearing capacity, as well as excessive settlement characteristics, and therefore it is necessary to improve the existing soft soils before construction activities commence, in order to prevent excessive and differential settlement (Richart, 1957). The use of vertical drains with preloading is a popular technique for improving soil. Vertical drains accelerate consolidation by providing short horizontal drainage paths, and are employed worldwide in many soft soil improvement projects (Holtz et al., 1991; Indraratna et al., 1992; Indraratna and Redana, 2000; Chu et al., 2000; Chai et al., 2006; Indraratna et al., 2011). Prefabricated vertical drains (PVDs) have become an economical and viable ground improvement option because of their rapid installation with simple field equipment (Shang et al., 1998; Bo et al., 2003; Chai et al., 2010; Artidteang et al., 2011; Walker et al., 2009). In order to increase the stability of embankments, surcharge placement is usually a multi-stage exercise with rest periods between the loading stages so that consolidation and associated shear strength gain occur before the next lift (Jamiolkowski et al. 1983, Mesri and Khan 2012). This practice may not be possible with tight construction schedules or foundation soil with very low shear strength. The application of a vacuum load in addition to surcharge fill can further accelerate the rate of settlement to obtain the desired settlement without increasing the excess pore pressure (Kjellman, 1952; Qian et al., 1992; Qiu et al., 2007; Saowapakpiboon et al., 2011, Indraratna et al., 2010a). This practice has been used for land reclamation and port projects (Tang and Shang, 2000; Yan and Chu, 2005; Chu and Yan, 2005; Chai et al., 2010a; Saowapakpiboon et al., 2010; Indraratna et al., 2011). The PVDs distribute the vacuum pressure to deep layers of subsoil, thereby reducing the excess pore water pressure due to surcharge loading (Zhu and Miao, 2002; Chai et al., 2009; Indraratna et al., 2010b). The consolidation process of vacuum preloading compared to conventional preloading is shown in Figure 1.

In this paper, a modified radial consolidation theory that considers the effects of time dependent surcharge loading and vacuum pressure is proposed. The smear zone prediction using the Elliptical Cavity Expansion Theory is discussed based on the results of large scale laboratory tests. The equivalent (transformed) plane strain conversion is incorporated into finite element codes using the modified Cam-clay theory. Case histories are discussed and analysed, including the site of the Second Bangkok International Airport (Thailand) and the Port of Brisbane. The predictions are compared with the available field data.

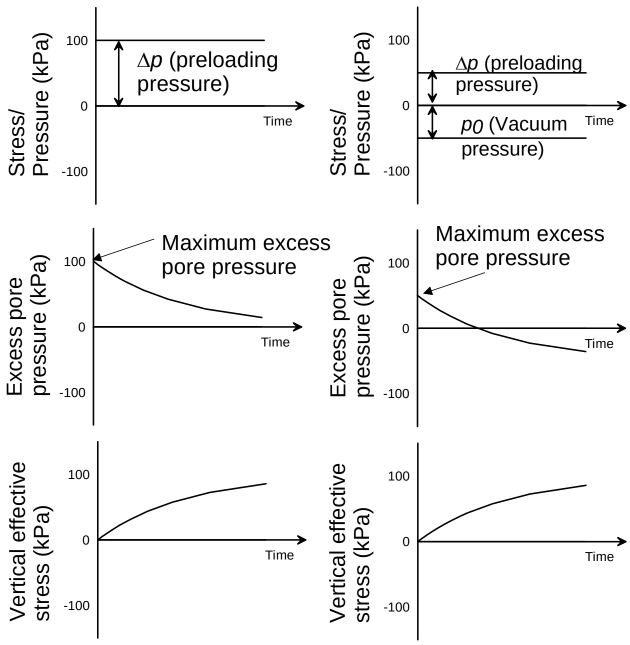


Figure 1 Consolidation process: (a) conventional loading (b) idealised vacuum preloading (modified from Indraratna et al., 2005c)

2. THEORETICAL APPROACH

2.1 Development of Vacuum Consolidation Theories

The mechanism of vacuum-assisted consolidation is comparable to, but not the same as conventional surcharge. In earlier studies, vacuum preloading was often simulated with an equivalent surface load or by modifying the surface boundary condition. However, laboratory observations confirm that the vacuum pressure propagates downwards along the drains in addition to the uniformly applied surface suction. In the absence of vertical drain, Mohamedelhassan and Shang (2002) developed a vacuum and surcharge combined one-dimensional consolidation model based on the Terzaghi's consolidation theory. Indraratna et al (2004; 2005a) Geng et al. (2012) showed that when the vacuum pressure is applied in the field through PVDs, the suction head along the drain length may decrease with depth, thereby reducing the efficiency. A modified radial consolidation theory to include different vacuum pressure distribution patterns has been proposed. The results indicate that the efficiency of vertical drains depends on both the magnitude of vacuum pressure and its distribution. Chai et al. (2006; 2010a) and Robinson et al. (2012) introduced an approximate method for calculating the ground settlement and inward lateral displacement induced by vacuum consolidation. Rujikiatkamjorn and Indraratna (2007; 2009) presented the design charts for vacuum consolidation.

2.2 Solution for Axisymmetric Condition

A radial consolidation theory incorporating the smear effect and well resistance was proposed by Barron (1948) and Hansbo (1981). The application of a vacuum pressure with only a surcharge load along the surface (i.e. no vertical drains), was modelled by Mohamedelhassan and Shang (2002) based on one-dimensional consolidation. The above mathematical models are based on instantaneous loading and a constant coefficient of lateral permeability (k_h). Lekha et al. (1998) further extended the solution by incorporating time dependent surcharge loading. Walker et al. (2009) proposed a spectral method for a vertical and redial consolidation analysis of stratified soils. Indraratna et al. (2005b) introduced the unit cell analysis for vacuum preloading under instantaneous loading. However, while an embankment is being constructed on soft clay, the fill surcharge is usually raised over time to attain the desired height. Therefore, a time dependent loading due to filling would be more appropriate than an instantaneous loading, especially during the initial stages of construction. In this Section the embankment load from filling (σ_t) is assumed to increase linearly up to a maximum value (σ_1) at time t_0 and kept constant thereafter (Figure 2a). The vacuum is applied at $t=t_{vac}$. Figure 2b illustrates the unit cell adopted for analytical solutions with boundary conditions (Figure 2c).

Indraratna et al. (2011) proposed that the average excess pore pressure due to radial consolidation while considering the smear effect under time dependent surcharge ($\overline{u_L}$) can be expressed by:

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

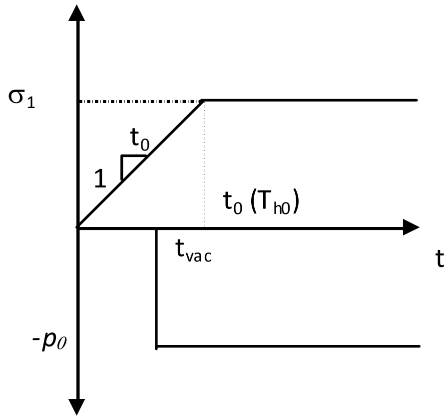
$$\overline{u_L} = \frac{\mu d_e^2}{8c_h t_0}\left(1-\exp\left(\frac{-8c_h t_0}{\mu d_e^2}\right)\exp\left(\frac{-8c_h (t-t_0)}{\mu d_e^2}\right)\right)\sigma_1 \quad for \quad t > t_0 \tag{2}$$

where, d_e is the influence zone diameter, c_h is the coefficient of consolidation for horizontal drainage, σ_1 = applied surcharge pressure, t = time,

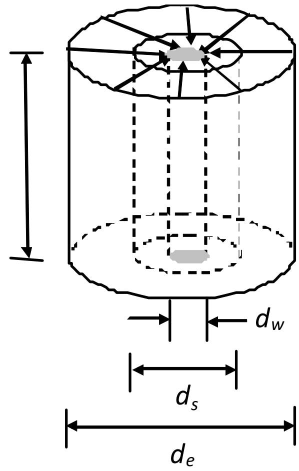
Recently, Indraratna et al. (2005b) showed that the average excess pore pressure under radial consolidation due to vacuum pressure (u_{vac}) alone could be determined from:

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

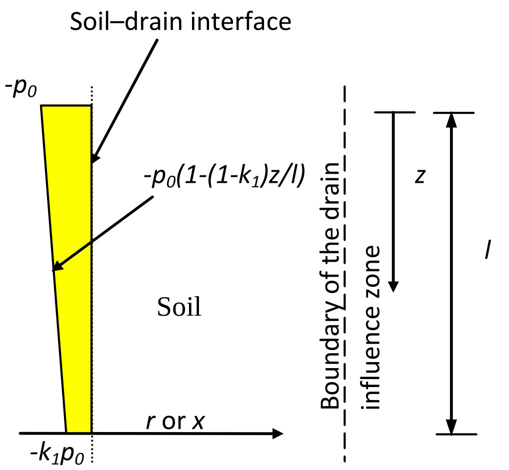
$$u_{vac} = p_0 \exp\left(-\frac{-8c_h (t-t_{vac})}{\mu d_e^2}\right) - p_0, \quad t \ge t_{vac} \tag{4}$$



(a)



(b)



(c)

Figure 2 (a) time-dependent loading, (b) unit cell, and (c) boundary conditions

where d_e = the diameter of the 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. μ = a group of parameters representing the geometry of the vertical drain system and the smear effect. Hansbo (1981) assumed the smear zone to have a reduced horizontal permeability that is constant throughout this zone. The μ parameter is given by:

$$\mu = \ln n/s + k_h/k'_h \ln s - 0.75 \tag{5a}$$

where, $n = \dfrac{d_e}{d_w}$, $s = \dfrac{d_s}{d_w}$, d_s =diameter of smear zone, d_w = equivalent diameter of drain, k_h = permeability in the undisturbed zone and k'_h = permeability in the smear zone.

However, laboratory testing conducted by Onoue et al. (1991), Indraratna and Redana (1998) and Sharma and Xiao (2000), using a large scale consolidometer, suggests that the disturbance in the 'smear zone' increases towards the drain (Figure 3). To obtain more accurate predictions, Walker and Indraratna (2006) employed a parabolic decay in horizontal permeability towards the drain to represent the actual variation of permeability in the smear zone. The μ parameter can be given by:

$$\mu_p = \begin{bmatrix} \ln\left(\dfrac{n}{s}\right) - \dfrac{3}{4} + \dfrac{\kappa(s-1)^2}{\left(s^2 - 2\kappa s + \kappa\right)} \ln\left(\dfrac{s}{\sqrt{\kappa}}\right) \\ - \dfrac{s(s-1)\sqrt{\kappa(\kappa-1)}}{2\left(s^2 - 2\kappa s + \kappa\right)} \ln\left(\dfrac{\sqrt{\kappa}+\sqrt{\kappa-1}}{\sqrt{\kappa}-\sqrt{\kappa-1}}\right) \end{bmatrix} \tag{5b}$$

In the above expression $\kappa = k_h/k_0$ and k_0 = minimum permeability in the smear zone.

The excess pore pressure at a given time t can determined based on the combination of Equations (1) to (5). For normally consolidated clay, the vertical settlement (ρ) can now be evaluated by the following equations:

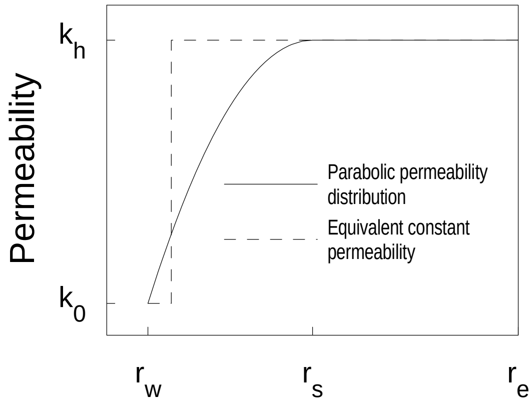
$$\rho = \frac{HC_c}{1+e_0} \log\left(\frac{\sigma'}{\sigma'_i}\right) \tag{6}$$

where ρ = settlement at a given time, C_c = compression index, σ' = effective at a given time, σ'_i = initial effective stress, and H = thickness of compressible soil.

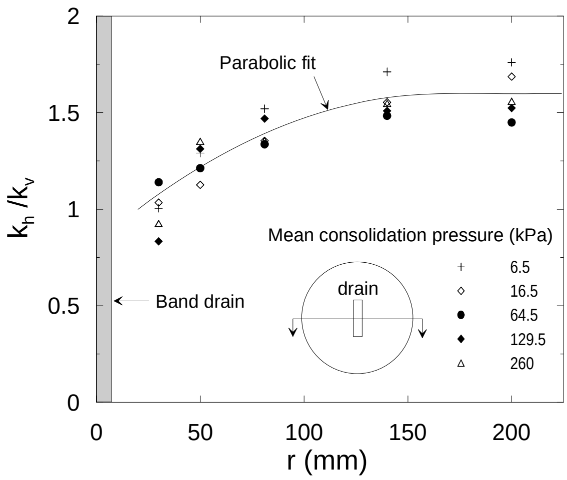
In order to predict excess pore pressures and associated settlements, Equations (1)-(6) can be used in conjunction with the soil properties of each layer and the thickness of the soil for each section.

2.3 Conversion Procedure for Equivalent Plane Strain Analysis

Indraratna and Redana (2000) and Indraratna et al. (2005a) showed that, based on the appropriate conversion procedure, and by considering the degree of consolidation at a given time step, plane stain multi-drain analysis can be used to predict the behaviour of soft soil improved by vertical drains and vacuum preloading. Using the geometric transformation in Figure 4, the corresponding ratio of the permeability of the smear zone to the undisturbed zone is obtained by (Indraratna et al., 2005a):



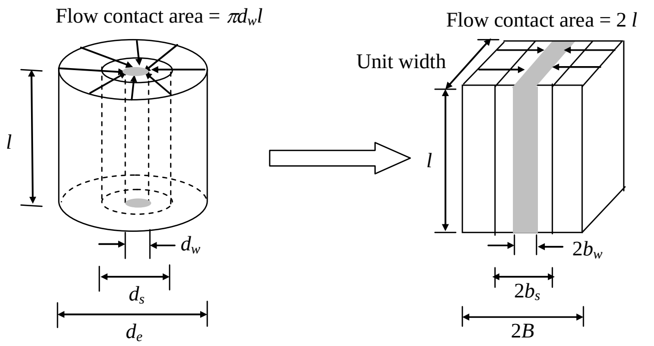
(a)



(b)

Figure 3 (a) Permeability distribution and (b) ratio of horizontal to vertical permeability (k_h/k_v) along radial distance from drain in large scale consolidometer (Walker and Indraratna, 2006)
(r_e=d_e/2, r_s=d_s/2 and r_w = d_w/2)

$$\frac{k'_{h,ps}}{k_{h,ps}} = \beta \Bigg/ \left(\frac{k_{h,ps}}{k_{h,ax}} \begin{bmatrix} \ln\left(\dfrac{n}{s}\right) + \dfrac{k_{h,ax}}{k'_{h,ax}} \ln(s) \\ - \dfrac{3}{4} \end{bmatrix} - \alpha \right) \tag{7}$$



(a) **(b)**

Figure 4 Unit cell analysis: (a) axisymmetric condition,
(b) equivalent plane strain condition (after Indraratna et al., 2005a)
(B, b_s, b_w= half width of unit cell, smear zone and drain, respectively under plane strain condition)

where $\alpha = 0.67 \times (n-s)^3 / n^2(n-1)$,

$\beta = \dfrac{2(s-1)}{n^2(n-1)}\left[n(n-s-1) + \dfrac{1}{3}(s^2+s+1) \right]$ and subscripts ps and ax represent plane strain and axisymmetric condition, respectively.

By ignoring the effects of both smear and well resistance, a simplified ratio of equivalent plane strain to axisymmetric permeability in the undisturbed zone can be attained based on the geometric equivalence (i.e. $d_w=2b_w$, $d_s=2b_s$, $d_e=2B$, in Figure 4, Indraratna et al., 2005) hence;

$$k_{h,ps} / k_{h,ax} = \frac{2}{3}\frac{(n-1)^2}{n^2} / \left[\ln(n) - 0.75\right] \tag{7a}$$

2.4 Consolidation Theory for Multi-layered Soil

Assuming time independent soil properties that vary spatially with depth, the governing equation for consolidation with combined vertical and radial drainage under instantaneous loading and equal strain conditions in a cylindrical unit cell can be derived as (Walker, 2006):

$$\frac{m_v}{\overline{m}_v}\frac{\partial \overline{u}}{\partial t} = -\left[dT_h \frac{\eta}{\overline{\eta}}\overline{u} - dT_v \frac{\partial}{\partial Z}\left(\frac{k_v}{\overline{k}_v}\frac{\partial \overline{u}}{\partial Z} \right) \right] \tag{8}$$

where $Z = \dfrac{z}{H}$, $dT_v = \dfrac{\overline{c}_v}{H^2}$, $dT_h = \dfrac{2\overline{\eta}}{\gamma_w \overline{m}_v}$, $\overline{c}_v = \dfrac{\overline{k}_v}{\gamma_w \overline{m}_v}$, $\eta = \dfrac{k_h}{r_e^2 \mu}$. In the preceding, $\overline{u}$ =average excess pore pressure for a given depth, t =time, z =depth, H =depth of soil, γ_w = unit weight of water, m_v =volume compressibility and k_v =vertical permeability. Equation (8) has been normalised with respect to convenient reference values of each property indicated by the over-bar notation. Vertical flow to the surface is based on the average hydraulic gradient. Walker (2006) presented solutions to Eq. (8) for multiple layers (see Figure 5 based on the spectral method. The three parameters k_v, m_v and η in the l^{th} layer, are described using the unit step (Heaviside) function (Walker 2006):

$$\alpha(Z) = \alpha_l UnitStep(Z - Z_{l-1}) UnitStep(Z_l - Z) \tag{9}$$

The spectral method assumes a truncated series solution of N terms:

$$\overline{u}(Z,t) \approx \mathbf{\Phi A} \approx \begin{pmatrix} [\phi_1(Z) \quad \phi_2(Z) \quad \ldots \quad \phi_N(Z)] \times \\ [A_1(t) \quad A_2(t) \quad \ldots \quad A_N(t)]^T \end{pmatrix} \tag{10}$$

In the preceding, ϕ_j is a set of linearly independent basis-functions, and $A_j(t)$ are unknown coefficients. The basic functions were chosen to satisfy the boundary conditions. In the current analysis, for a pervious top and bottom (PTPB) $\overline{u}(0,t)=0$ and $\overline{u}(H,t)=0$, and for a pervious top and bottom (PTIB) $\overline{u}(0,t)=0$ and $\partial\overline{u}(H,t)/\partial z = 0$. Suitable basis functions are thus:

$$\phi_j(Z) = \sin(M_j Z) \tag{10a}$$

where $M_j = \begin{cases} j\pi & \text{for PTPB} \\ \dfrac{\pi}{2}(2j-1) & \text{for PTIB} \end{cases}$

The Galerkin procedure requires that the error in Eq. (10) is orthogonal to each basis function, hence:

$$\int_0^1 \phi_i L(\mathbf{\Phi A})\, dZ = 0 \tag{11}$$

where L describes the differential operations in Eq. (11). Combining Eqs. (9), (10), (11) yields a set of coupled ordinary differential equations for A_j, which in matrix form reads:

$$\mathbf{\Gamma A'} = -\mathbf{\Psi A} \tag{12}$$

where

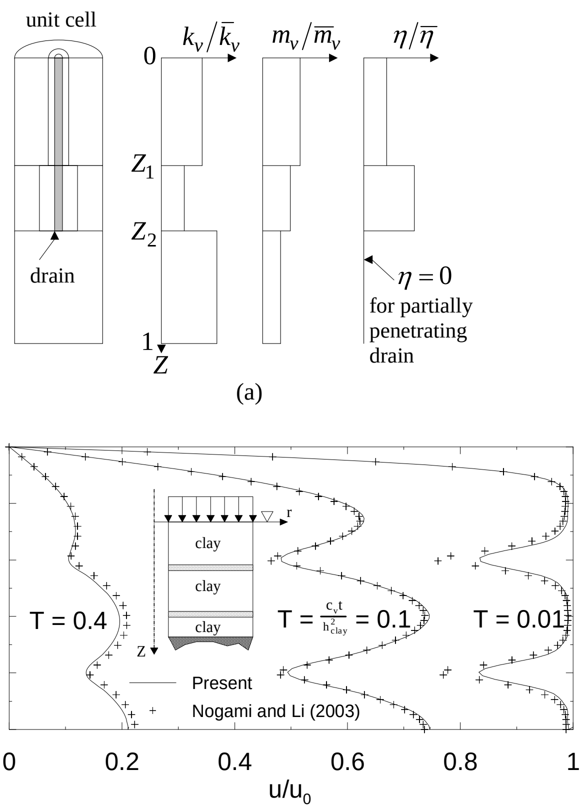
$$\mathbf{\Gamma}_{ij} = \Lambda^-_{ij} m_{vl} / \overline{m}_v$$

$$\mathbf{\Psi}_{ij} = dT_v M_j M_i \Lambda^+_{ij} k_{vl} / \overline{k}_v + dT_h \Lambda^-_{ij} \eta_l / \overline{\eta} \tag{12a}$$

$$\Lambda^{\pm}_{ij} = \begin{cases} SN(M_j - M_i) \pm SN(M_j + M_i) & i \neq j \\ (Z_l - Z_{l-1}) \pm SN(M_j + M_i) & i = j \end{cases},$$

$$SN[\beta] = (\sin(\beta Z_l) - \sin(\beta Z_{l-1}))/\beta \tag{12b}$$

Based on the eigen problem of Eq. (12), under instantaneous loading the solution to Eq. (11) is:



(b)

Figure 5 (a) Multi-layered consolidation properties and (b) model verification: multi-layer equal-strain vs free-strain (Walker et al., 2009)

The diagonal matrix $\mathbf{E}$ (square matrix with non-diagonal terms equal to zero) $\mathbf{E}$ is:

$$\overline{u}(Z,t) \approx \mathbf{\Phi v E}(\mathbf{\Gamma v})^{-1}[\theta_1 \quad \theta_2 \quad \ldots \quad \theta_N]^T \tag{13}$$

$$\mathbf{E}(t) = \mathrm{diag}\left[\exp(-\lambda_1 t) \quad \exp(-\lambda_2 t) \quad \cdots \quad \exp(-\lambda_N t)\right] \tag{13a}$$

where λ is an eigenvalue value of matrix $\mathbf{\Gamma}^{-1}\mathbf{\Psi}$. The eigenvector associated with each eigenvalue makes up the columns matrix $\mathbf{v}$ (i.e. v_{i1} is the eigenvector associated with λ_1). $\mathbf{\theta}$ is a column vector defined by:

$$\theta_i = 2\left(1 - \cos(M_i)\right)/M_i \tag{13b}$$

To find the average pore pressure between depth Z_1 and Z_2 the $\phi_j(Z)$ terms in $\mathbf{\Phi}$ are replaced with:

$$\bar{\phi}_j(Z_1, Z_2) = \begin{pmatrix}\cos(M_j Z_1) - \\ \cos(M_j Z_2)\end{pmatrix} \Big/ M_j (Z_2 - Z_1) \tag{13c}$$

The adoption of the current method via Equation (13c) allows one to apply a straightforward way of determining the average pore pressure values within a soil layer, across some layers, or across all layers. Nogami and Li (2003) developed a free strain approach for calculating the distribution of excess pore pressure for multi-layered soil having vertical and radial drainage. An example problem is presented with a soil system consisting of two identical layers of thin sand (height h_s) separating three identical layers of clay (height, h_c). The properties of the soil are described by the ratios: $k_{\text{sand}} h_s h_c / r_e^2 k_v$ = 5, n = 20, $c_h h_c^2 / c_v r_e^2$ = 1. The average excess pore water pressure calculated with the present approach, and that of Nogami and Li (2003), is compared in Figure 5b. Both methods are in close agreement except for slight deviations in the thin layers of sand at a low degree of consolidation. The close agreement shows that, as for homogenous ground (Hansbo, 1981; Barron, 1948), there is little difference between free strain and equal strain formulations.

3. DETERMINATION OF THE SMEAR ZONE AND LARGE SCALE LABORATORY TESTING

The term 'smear zone' is generally referred to as the disturbance that occurs when installing a vertical drain. This causes a substantial reduction in soil permeability around the drain, which in turn retards the rate of consolidation. In this section, the Cavity Expansion Theory is used to estimate the extent of the smear zone. The prediction is then compared with the laboratory results based on permeability and variations in the water content.

The extent of the "smear zone" caused by mandrel installation can be estimated using the elliptical cavity expansion theory incorporating the modified Cam-clay model (MCC) (Ghandeharioon et al. 2010). The detailed theoretical developments are explained elsewhere by Ghandeharioon et al. (2010), so only a brief summary is given below. The yielding criterion for soil obeying the MCC model is:

$$\eta = \mathrm{M}\sqrt{\left(p_c'/p'\right) - 1} \tag{14}$$

Where, p_c': the stress representing the reference size of yield locus, p' = mean effective stress, M = slope of the critical state line and η = stress ratio. The relationship between the radial distance and its associated deviator stress can be determined by:

$$\ln\left(1 - \frac{r_1^2 - r_0^2}{r^2}\right) = \left[\begin{array}{l} \dfrac{q}{\sqrt{3}G} - \\ 2\sqrt{3}\dfrac{\kappa\Lambda}{\upsilon M}\left[\begin{array}{l}\zeta - \tan^{-1}\left(\dfrac{\eta}{M}\right) \\ + \tan^{-1}\sqrt{n_p - 1}\end{array}\right]\end{array}\right] \tag{15}$$

$$\zeta = \frac{1}{2}\ln\frac{(\eta + M)\left(\sqrt{n_p - 1} - 1\right)}{(\eta - M)\left(\sqrt{n_p - 1} + 1\right)} \tag{15a}$$

$$q = \eta\, p' \tag{15b}$$

$$\sigma_r = \sigma_{r_p} - \frac{2}{\sqrt{3}}\int_{r_p}^{r} q\frac{dr}{r} \tag{16}$$

$$\Delta u = \Delta p - \Delta p' \tag{17}$$

In the above expression, r = radius of the cavity, r_0 = initial radius of the cavity, G = Shear modulus, ν = Poisson's ratio, κ = slope of the over consolidation line, υ = specific volume, σ_{r_p} = total radial stress at the elastic-plastic boundary, Δu = excess pore water pressure and $\Lambda = 1 - \kappa/\lambda$ (λ is the slope of the normal consolidation line).

Based on Equations (15), q and p' can be calculated at any soil element inside the plastic region. Equation (16) is then used to derive the total stress state at that particular position, while noting that $p = \sigma_r - \dfrac{q}{\sqrt{3}}$. Finally, by using Equation (17), the value of excess pore pressure can be determined at the location being considered.

The extent of the smear zone can be defined either by the variation of permeability (Indraratna and Redana, 1998) or by the variation of the water content (Sathananthan and Indraratna, 2006) along the radial distance from the central drain. The permeability variation can be obtained from specimens recovered vertically and horizontally from the large-scale consolidation apparatus. In the field, the measurement of moisture content variation is more convenient. Consolidation tests were conducted to obtain the horizontal and vertical permeabilities under different pressures. Figure 6 shows the variation of the permeability ratio (k_h/k_v) and water content at different consolidation pressures along the radial distance, obtained from large scale laboratory consolidation. Here the radius of the smear zone is about 100mm or 2.5 times the radius of the mandrel, which is in agreement with the prediction using the cavity expansion theory.

Figure 7 presents the analytical predictions of excess pore water pressure using the conventional cylindrical cavity expansion theory (CET) (Cao et al., 2001) and elliptical CET of the Authors (Ghandeharioon et al., 2010), compared with the results of the large scale laboratory tests. It is clear that the elliptical CET estimates the pore pressure during mandrel installation more accurately than the conventional cylindrical CET. By incorporating the laboratory test results on soil permeability, Ghandeharioon et al. (2010) proposed that the plastic shear strain normalised by the rigidity index, $I_r = \sqrt{3}\dfrac{G}{q_f}$, can be adopted to characterise the disturbed soil surrounding the mandrel driven prefabricated vertical drains (Figure 8).

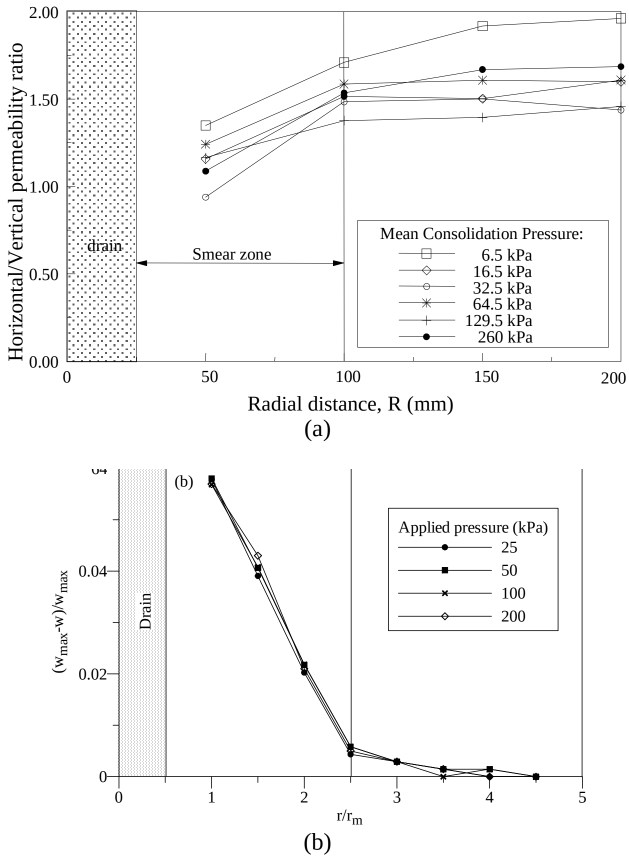


Figure 6 Smear zone determination (a) permeability approach (Indraratna and Redana, 1998); (b) water content approach (Sathananthan and Indraratna, 2006) (w = water content)

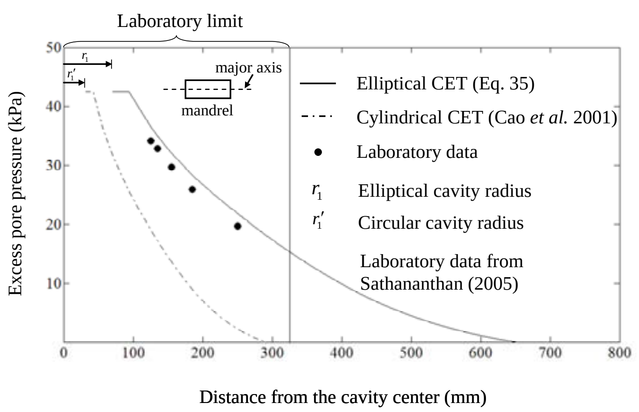


Figure 7 The patterns of distribution s predicted for excess pore pressure with the radial distance using elliptical CET and cylindrical CET along the major axis of the mandrel 0.5m below the soil surface, and measured when the tip of the mandrel's shoe passed the horizontal plane under consideration, with a preconsolidation pressure = 30 kPa (Ghandeharioon et al. 2010)

4. COMPARISON OF MEMBRANE AND MEMBRANELESS VACUUM PRELOADING SYSTEMS

Numerical and analytical modelling of vacuum preloading while considering membrane-type and membraneless systems has been described earlier by Geng et al. (2012), where both vertical and horizontal drainage were captured to reflect in-situ conditions. The placing of the surface sand blanket and installation of a completely air tight membrane is imperative for the membrane type vacuum system, in order to create and sustain a desired uniform vacuum pressure on the surface of the soil, and thereby ensure a speedy propagation of this vacuum head down the PVDs to consolidate the clay layer. While a surface sand blanket has no real advantage except for trafficability, for the membraneless system where the vacuum is applied directly onto the PVDs through a network of tubing, the absence of a membrane eliminates construction time and associated costs. The permeability of the layer of sand plays an important role in this process as it governs how effectively the vacuum pressure propagates from the boundary of the upper soil to the PVD's to consolidate the layer of clay. The roles of the permeability of the sand blanket in a membrane-type vacuum system and adverse effect of the loss of vacuum with depth in a membraneless type system have been analysed by Geng et al. (2012). Figure 9 illustrates the effect of the sand blanket permeability in a membrane system.

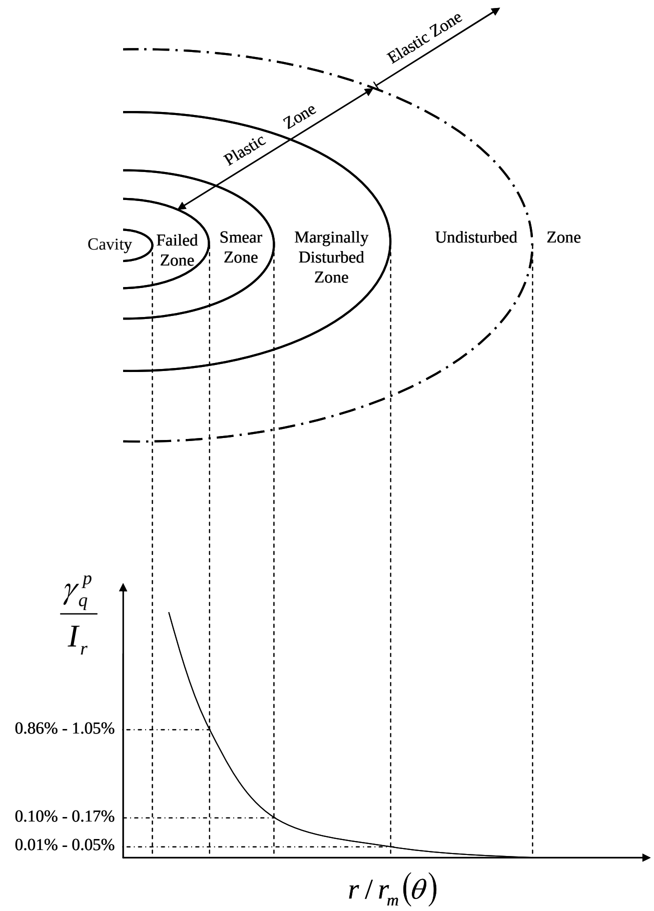
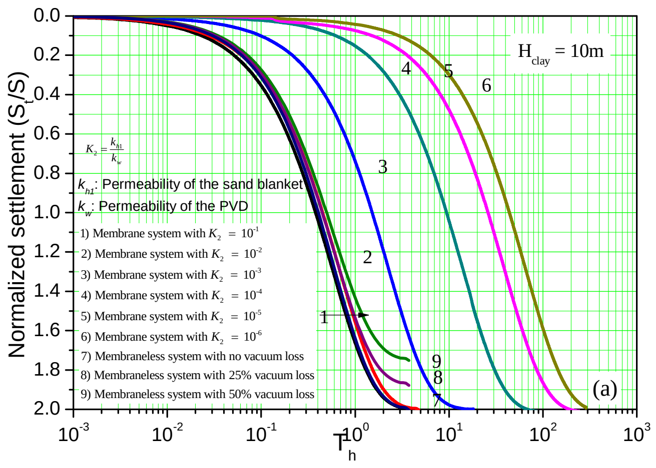
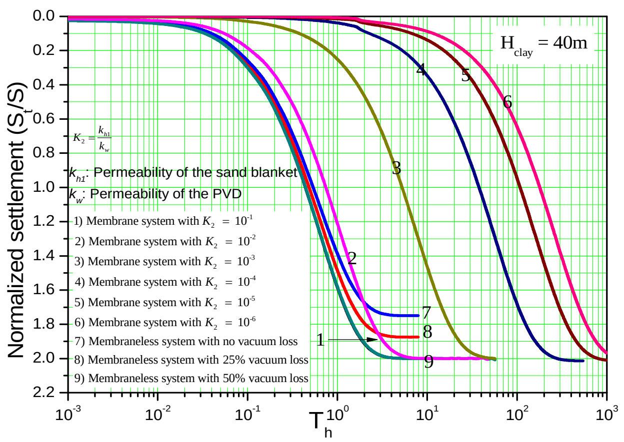


Figure 8 The distribution pattern for the ratio of the plastic shear strain to the rigidity index in relation to the radial distance normalised by the equivalent elliptical radius of the mandrel characterising the disturbed soil surrounding a PVD (Ghandeharioon et al., 2010)

As expected, when permeability decreases, the time for consolidation increases. For relatively short PVDs (less than 10 m), Fig.9a shows that the permeability of the sand blanket should not be less than 0.01 times the permeability of the PVD, and at least 10^4 times the permeability of the clay to maintain an acceptable consolidation time for a degree of consolidation (DOC) of 90%. With longer drains (Figure 9b), the permeability ratio of the sand blanket to PVD should be greater than 0.1, and the permeability ratio of the sand blanket to the clay layer should be at least 10^5. For a membraneless system, the possible reduction in vacuum along the length of long PVDs increases the consolidation time for a given DOC. Where there is no loss of vacuum with depth, the membraneless system is as efficient as the membrane-type (Figure 9), for relatively shallow (10 m) and very thick (40 m) layers of clay.



(a)



(b)

Figure 9 Normalised settlement-time factor curves for varying the permeability of the sand blanket (for membrane system) and the loss of vacuum (for membraneless system): (a) the thickness of the clay is 10 m; (b) the thickness of the clay is 40 m (after Geng et al., 2012)

5. APPLICATION TO CASE HISTORIES

5.1 Second Bangkok International Airport

Indraratna et al. (2004) analysed the performance of test embankments constructed at the Second Bangkok International Airport (SBIA), Thailand. At this site, the use of vacuum preloading *in lieu* of high surcharge embankment as an alternative preloading technique was also studied. Table 1 summarises the typical modified Cam-clay parameters and equivalent plane strain permeability (using Eqs. 7-8) for the FEM analysis. A cross section of the embankment and typical finite element mesh used in the multi-drain analysis are given in Figure 10. The test embankment was raised and stabilised with PVDs installed in a triangular pattern with 1m spacing to a depth of 15 m. The 100mm x 3mm PVDs (Mebra) were used.

The embankment loading was simulated by the sequential construction history (Figure 11a). The following 4 models were considered and numerically evaluated under plane strain multi-drain analysis (Indraratna et al., 2004):

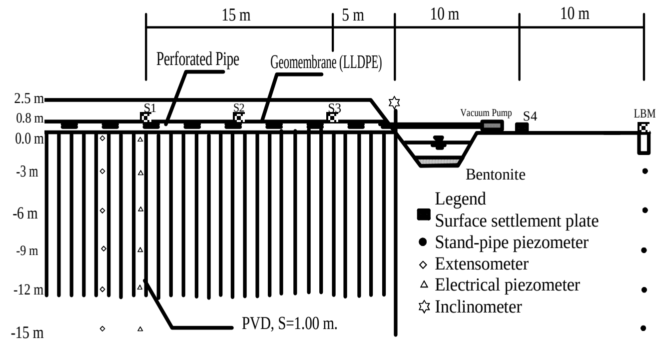
Model 1 –With the application of suction pressure (60 kPa) along the surface of the top soil, and along the length of the drain, a thin layer of unsaturated elements of predetermined constant half widths (30mm) was activated at the boundary of the drain.

Model 2– Similar to Model 1, with a constant 60 kPa suction along the surface of the top soil, but a linearly varying vacuum pressure (60 kPa at top and zero at bottom) applied along the depth of the drain.

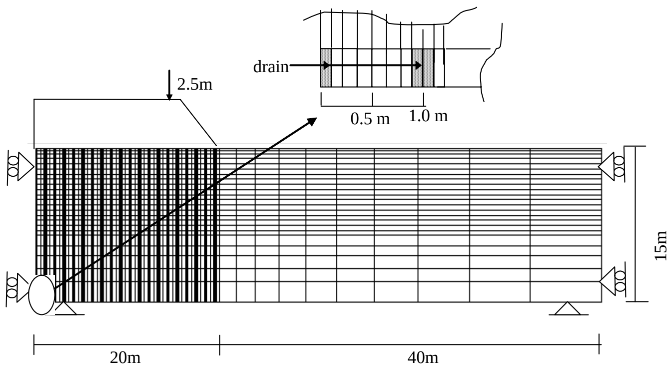
Model 3 – Similar to Model 2, but the vacuum pressure was varied linearly with depth to zero at the bottom of the drain and varied with time (Figure 11b), as occurs in the field. The vacuum distribution with depth and time was measured at the drain interface. However, the predictions were made at the middle between 2 drains

Model 4 – Conventional surcharge alone with no vacuum pressure.

Figure 11c shows the predicted settlement together with the measured settlement. The Model 3 predictions agreed well with the field data. The assumed time dependent variation of vacuum pressure based on surface measurements improves the accuracy of settlement predictions.



(a)



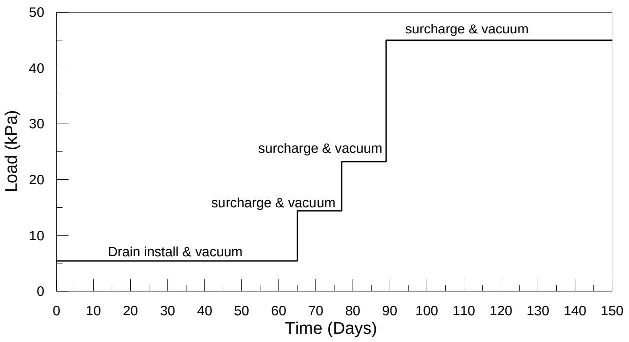
(b)

Figure 10 (a) Cross-section of an embankment with profile of the subsoil and (b) finite element discretisation of the foundation of the embankment (Indraratna et al., 2004)

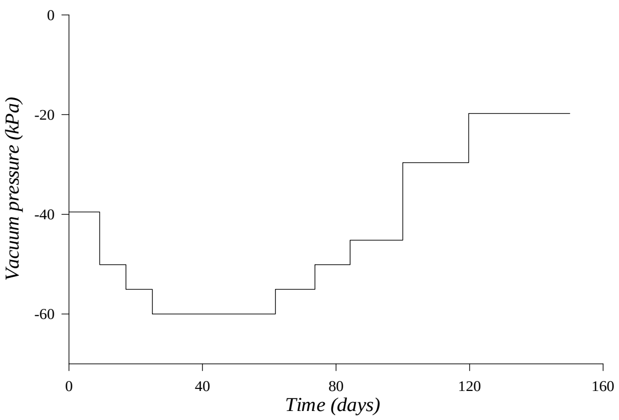
The measured and predicted excess pore pressures along the embankment centreline 3 m below the ground surface are compared in Figure 12a. Model 3 shows that the time dependent variation in vacuum agrees with the field measurements. All the other models that do not consider the time dependent variation in vacuum pressure are unable to predict the field behaviour to an acceptable accuracy. Measured and predicted lateral deformation for the inclinometer installed away from the centreline of the embankment (after 150 days) is shown in Figure 12b. All 3 models incorporating vacuum pressure have caused 'inward' (radial) movement. The effect of the compacted crust is not clearly reflected by the field data, which suggests that the depth of the crust is no more than 1m in the field, whereas the numerical analysis assumed a 2 m thick crust. The loss of the vacuum head increases the lateral movements more in line with Model 4.

5.2 Port of Brisbane

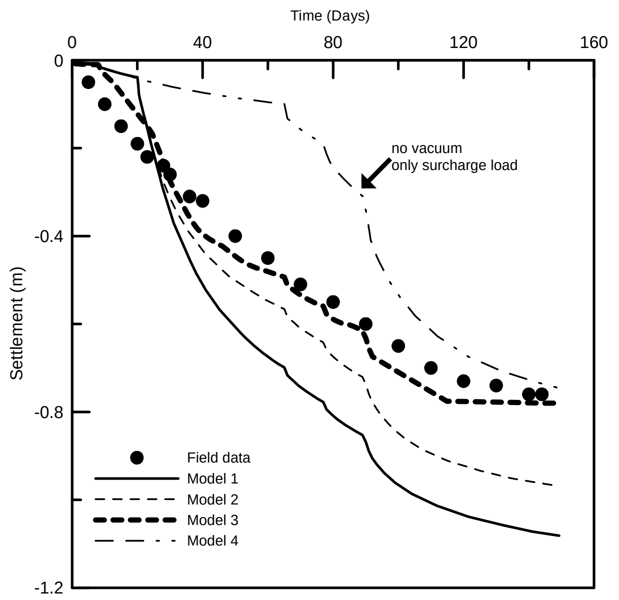
The Port of Brisbane is one of the largest container ports in Australia located at the mouth of the Brisbane River. With an increased demand in commercial activities, a new outer area (235000 m^2) close to the current port facilities is being reclaimed to maximise the land area, and provide an additional number of berths suitable for bulk cargo and container handling. In this area the soil profile primarily consists of high compressible clay over 30 m deep, with an undrained shear strength that is lower than 15 kPa at shallow depths. The dredged mud used for reclamation has a much lower strength, depending on the time of placement and duration that the capping material has been in place. In the absence of surcharge preloading, it is estimated that the consolidation time is in excess of 50 years with vertical settlements of 2.5-4.0m. Therefore, vacuum consolidation with prefabricated vertical drains (PVDs) was recommended to accelerate the consolidation process and minimise lateral deformation adjacent to the Moreton Bay Marine Park (Indraratna et al., 2011).



(a)



(b)



(c)

Figure 11 (a) Stage loading (b) variation of vacuum with time for Model 3 and (c) settlement predictions (Indraratna et al. 2004)

Table 1 Critical state soil parameters used in the analysis (Indraratna et al., 2004)

Depth (m)	**0-2.0**	**2.0-8.5**	**8.5-10.5**	**10.5-13.0**	**13-18**
e_o	1.8	2.8	2.1	1.8	1.2
λ	0.3	0.73	0.5	0.3	0.1
κ	0.03	0.08	0.05	0.03	0.01
ν	0.3	0.3	0.25	0.25	0.25
M	1.2	1.0	1.2	1.4	1.4
$k_{h,ax}$ (m/s)	30 $\times 10^{-8}$	1.3 $\times 10^{-8}$	6.0 $\times 10^{-9}$	2.6 $\times 10^{-9}$	6.0×10^{-10}
$k'_{h,ax}$ (m/s)	3.01 $\times 10^{-9}$	1.27 $\times 10^{-9}$	6.02 $\times 10^{-10}$	2.55 $\times 10^{-10}$	6.02×10^{-11}
$k_{h,ps}$ (m/s)	8.98 $\times 10^{-9}$	3.80 $\times 10^{-9}$	1.80 $\times 10^{-9}$	7.60 $\times 10^{-10}$	4.15×10^{-11}
$k'_{h,ps}$ (m/s)	5.86 $\times 10^{-10}$	2.48 $\times 10^{-10}$	1.17 $\times 10^{-10}$	4.96 $\times 10^{-11}$	2.71×10^{-12}
γ (kN/m^3)	16.0	14.5	15.0	16.0	18.0

Note: e_o= initial void ratio
λ = slope of compression curve in semi-log scale
κ = slope of re-compression curve in semi-log scale
ν = Poisson's ratio
M= Slope of critical state line
k_h = permeability in undisturbed zone
k'_h = permeability in smear zone
γ = Unit weight of soil
ax and ps denote axisymmetric and plane strain condition, respectively

To assess the performance of the vacuum system with a conventional system (PVD and surcharge load), a trial area (S3A) was sub-divided into WD1-WD5 (Non-vacuum areas) and VC1-VC2 (Vacuum areas) (Figure 13). The treatment area of the sub-divisions ranged from 1.5 to 11 ha. To observe the ground behaviour, several instruments were installed including settlement plates, vibrating wire piezometers, magnetic extensometers, and inclinometers and their locations (Figure 2). The inclinometers were critical because excessive lateral deformation adjacent to the Moreton bay Marine Park needed to be controlled. After drying, the mud is capped off with a 2-3 m thick layer of dredged sand, which acts as a working platform for PVD installation rigs, whilst providing a drainage layer for the wick drains to discharge to. Table 2 summarises the PVD characteristics and types of treatment applied to each section. In non-vacuum areas, both circular and band shape drains were installed in a square pattern at a spacing of 1.1-1.3 m. The length of drains varied from 6m to 27.5 m across the site, as shown in Table 2.

It can be observed that the trends are very similar where settlement occurs more rapidly at the initial stage of consolidation. The magnitude of ultimate settlement depends on the thickness of the clay and height of the embankment. The highest settlement is observed in the WD4 area where the clay is thickest (19-26m), whereas the lowest settlement belongs to WD5A area where the clay is relatively thin (8-12m). The measured lateral displacement normalised to total change in applied stress (vacuum plus surcharge load) for two inclinometer locations (VC1/MS28 and WD3/MS27) are shown in Figure 14. In VC1 and the WD3 area, the total load on the surface is similar. At the WD3 area, the total height of surcharge was 4-5m (90 kPa), whereas for the VC1 area the reduced surcharge pressure of 40 kPa (2m surcharge height) was supplemented with a vacuum pressure of 65 kPa. These plots indicate that the lateral movements are well controlled via isotropic consolidation by vacuum pressure.

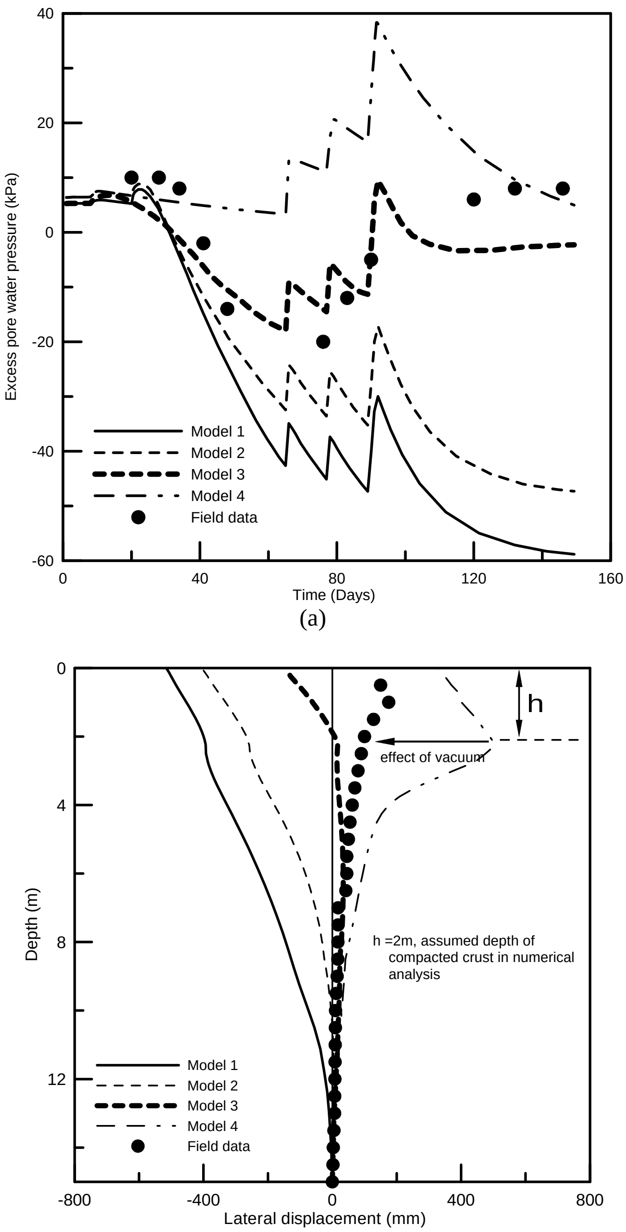


Figure 12 (a) Excess pore pressure predictions and (b) lateral displacement predictions (Indraratna et al., 2004)

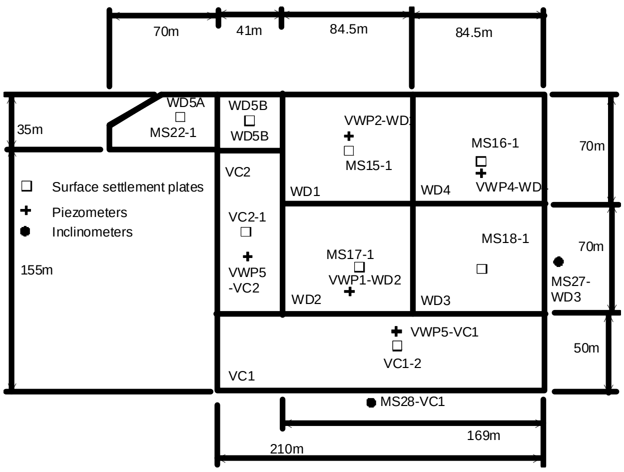


Figure 13 Site layout for S3A with instrumentation plan (Indraratna et al., 2011)

Table 2a PVD characteristics (Indraratna et al., 2011)

ection	Drain type	Drain length (m)	Drain spacing in square pattern (m)
WD1	Circular drains with 34mm diameter	14.5-18.5	1.1
WD2	Circular drains with 34mm diameter	22.5-27.5	1.3
WD3	Band drain Type -A (100×4 mm^2)	17.1-23.5	1.1
WD4	Band drains Type -A (100×4 mm^2)	27.0-28.7	1.3
WD5A	Band drains Type -B (100×4 mm^2)	6.0-8.0	1.2
WD5B	Band drains Type -B (100×4 mm^2)	13.5	1.1
VC1	Circular drains with 34mm diameter	14.0-26.5	1.2
VC2	Circular drains with 34mm diameter	15.5-22.5	1.2

Table 2b Clay thickness and improvement scheme (Indraratna et al., 2011)

Section	Clay thickness (m)	Total fill height (m)	Treatment scheme
WD1	12.0-15.5	5.2	Surcharge
WD2	20.0-23.5	7-7.2	Surcharge
WD3	14.0-17.0	4.3-4.6	Surcharge
WD4	22.5-24.5	6.1	Surcharge
WD5A	6.0-8.0	3.3	Surcharge
WD5B	9.5	5.5	Surcharge
VC1	9.0-21.0	3.2	Surcharge+ 70kPa vacuum
VC2	12.5-18.5	2.8	Surcharge+ 70kPa vacuum

Figure 15 presents the predicted settlement and associated excess pore pressure with the measured data in Areas VC2. Overall, the comparisons between prediction and field observation show that the settlement and associated pore water pressure can be predicted very well.

6. CONCLUSIONS

A system of vertical drains combined with vacuum preloading is an effective method of accelerating soil consolidation by promoting radial flow. The spectral method was proposed to predict the consolidation of layered soil. The variation of horizontal permeability coefficient (k_h) with the stress level was also included. The parabolic decay of horizontal permeability in the smear zone associated with the installation of the drains is considered to represent the actual variation. The elliptical cavity expansion theory was used to predict the extent of the smear zone, which was found to be in agreement with the laboratory data, based on the permeability and water content approaches. The application of a vacuum pressure increases the rate of pore pressure dissipation due to the increased hydraulic gradient towards the drain.

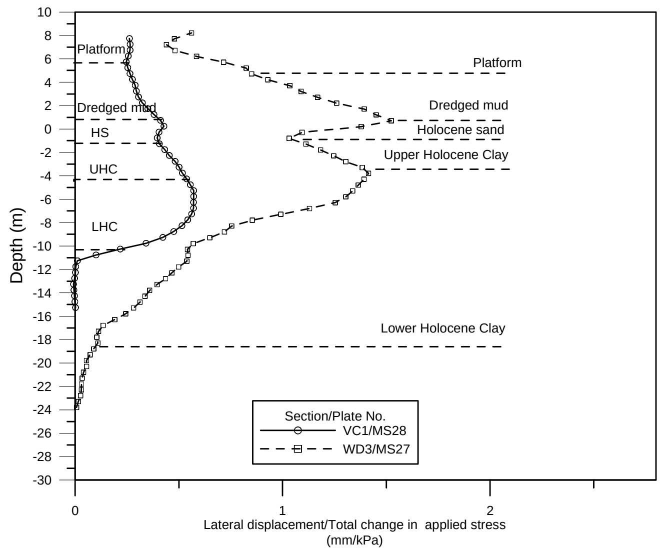


Figure 14 Comparison of lateral displacements at the toe of the embankment in vacuum and non-vacuum area after 400 days (Indraratna et al., 2011)

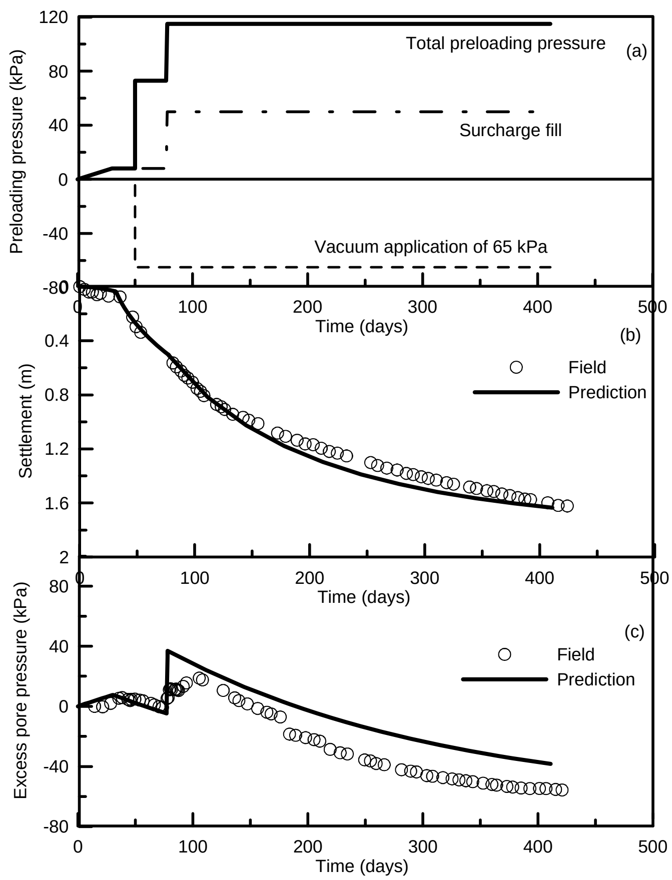


Figure 15 VC2 area: (a) stages of loading, (b) surface settlements under the centreline of the embankment and (c) excess pore pressures (Indraratna et al., 2011)

There are two types of vacuum preloading systems; (a) a membrane system with an airtight membrane over the drainage layer and, (b) a membraneless system (a vacuum system is connected to individual PVD). Their effectiveness varies depending on the types of soil treated and characteristics of the vacuum and drain. The analytical solutions of both systems under time dependent surcharge loading were presented in this paper. It can be seen that the proposed solution also included a loss of vacuum along the length of the drain. The general solutions of pore water pressure, settlement, and the degree of consolidation are derived by applying the powerful spectral technique. There is no doubt that a system of vacuum assisted consolidation via PVDs is a useful and practical approach for accelerating radial consolidation.

Generally, the length of PVD can be reduced to 80% of the layer thickness without significantly affecting the time for settlement. With surcharge preloading combined with vacuum pressure, the length of PVD can only be reduced by 0.1 of the entire thickness of soft clay. It can be seen that vacuum preloading alone may not be effective when there is a permeable layer at the bottom of the clay. The applications of the proposed solutions were validated through various case studies.

7. ACKNOWLEDGEMENTS

The authors wish to thank the Australian Research Council for its support. A number of current and past PhD students, namely, Dr. I. Redana, Dr. C. Bamunawita, Dr. R, Walker, Dr. A. Ghandaharioon, and Dr. I. Sathananthan have also contributed to the contents of this paper. A substantial component of the technical details presented in this paper can be found in earlier publications by the second author and his co-workers (albeit different interpretations) in the Géotechnique, ICE, ASCE and Canadian Geotechnical Journals, since mid 1990’s.

8. REFERENCES

Artidteang, S., Bergado, D.T., Saowapakpiboon, J., Teerachaikulpanich, N., and Kumar, A. (2011) “Enhancement of efficiency of prefabricated vertical drains using surcharge, vacuum and heat preloading”. Geosynthetics International,, 118, issue 1, pp35-47.

Barron, R.A. (1948) “Consolidation of fine-grained soils by drain wells”, Transactions ASCE, 113, pp718-754.

Bo, M.W., Chu, J., Low, B.K. and Choa, V. (2003) “Soil improvement; prefabricated vertical drain techniques”, Thomson Learning, Singapore, 341 p.

Cao, L. F., Teh, C. I., and Chang, M. F. (2001) “Undrained Cavity Expansion in Modified Cam Clay I: Theoretical Analysis”, Géotechnique, 51, Issue 4, pp323-334.

Chai, J.C., Carter, J.P., and Hayashi, S. (2005) “Ground Deformation Induced by Vacuum Consolidation” Journal of Geotechnical and Geoenvironmental Engineering, ASCE,131, Issue 12, pp187–192.

Chai, J.C., Miura, N., Kirekawa, T. and Hino, T. (2009) “Optimum PVD installation depth for two-way drainage deposit”, Geomechanics and Engineering, 1, Issue 3, pp179-192.

Chai, J.C., Carter, J.P. and Hayashi, S. (2006) “Vacuum consolidation and its combination with embankment loading”, Canadian Geotechnical Journal, 43, Issue 10, 985-996.

Chai, J., Hong, Z., and Shen, S. (2010) “Vacuum-drain consolidation induced pressure distribution and ground deformation”, Geotextiles and Geomembranes, 28, Issue 6, pp525-535.

Chu, J., and Yan, S.W. (2005) “Application of vacuum preloading method in soil improvement”, Ground Improvement-Case Histories, Indraratna, B. & Chu, J. (Eds.), Elsevier, pp91-118.

Chu, J., Yan, S.W. and Yang, H. (2000) “Soil improvement by the vacuum preloading method for an oil storage station”, 50, Issue 6, pp625-632.

Ghandeharioon, A., Indraratna, B., and Rujikiatkamjorn, C. (2010) “Analysis of soil disturbance associated with mandrel-driven prefabricated vertical drains using an elliptical cavity expansion theory”, International Journal of Geomechanics, ASCE, 10, Issue 2, pp53-64.

Geng, X. Y., Indraratna, B. and Rujikiatkamjorn, C. (2012) “Analytical solutions for a single vertical drain with vacuum and time-dependent surcharge preloading in membrane and membraneless systems", International Journal of Geomechanics, ASCE, 12, Issue 1, pp27-42.

Hansbo, S. (1981). "Consolidation of fine-grained soils by prefabricated drains and lime column installation", Proceedings of 10th International Conference on Soil Mechanics and Foundation Engineering, Balkema (Rotterdam) 3, pp677-682.

Holtz, R.D., Jamiolkowski, M.B., Lancellotta, R., and Pedroni, R. (1991) "Prefabricated Vertical Drains: Design and Performance", CIRIA: London; 1-131.

Indraratna, B. and Redana, I. W. (1998) "Laboratory determination of smear zone due to vertical drain installation." J. Geotech. and Geoenviron. Engng, 124, Issue 2, pp180-185.

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, pp133-145.

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, Issue 1, pp12-33.

Indraratna, B., Bamunawita, C., and Khabbaz, H. (2004) "Numerical modeling of vacuum preloading and field applications", Canadian Geotechnical Journal, 41, pp1098-1110.

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, pp994-1014.

Indraratna, B., Rujikiatkamjorn C., and Sathananthan, I. (2005b) "Radial consolidation of clay using compressibility indices and varying horizontal permeability", Canadian Geotechnical Journal, 42, pp1330-1341.

Indraratna, B., Rujikiatkamjorn C., Balasubramaniam, A. S. and Wijeyakulasuriya, V. (2005c) "Predictions and observations of soft clay foundations stabilized with geosynthetic drains and vacuum surcharge". Ground Improvement – Case Histories Book (Volume 3), Edited by Indraratna, B. and Chu, J., Elsevier, London, pp199-230.

Indraratna, B., Rujikiatkamjorn, C., Ameratunga, J., and Boyle, P. (2011) "Performance and Prediction of Vacuum Combined Surcharge Consolidation at Port of Brisbane", J. of Geotechnical & Geoenvironmental Engineering, ASCE, 137, Issue 11, pp1009-1018.

Indraratna, B., Geng, X., Rujikiatkamjorn, C. (2010a) "Review of methods of analysis for the use of vacuum preloading and vertical drains for soft clay improvement", Geomechanics and Geoengineering, 5, Issue 4, pp223-236.

Indraratna, B., Rujikiatkamjorn, C., Kelly, R. and Buys, H. (2010b) "Sustainable soil improvement via vacuum preloading", Ground Improvement, 163, Issue 1, pp31-42.

Jamiolkowski, M., Lancellotta, R., and Wolski, W. (1983) "Precompression and speeding up consolidation", Proc. 8th ECSMFE, pp1201-1206.

Kjellman, W. (1952) "Consolidation of clayey soils by atmospheric pressure", Proceedings of a conference on soil stabilization, Massachusetts Institute of Technology, Boston, pp258-263.

Lekha, K.R., Krishnaswamy, N.R. and Basak, P. (1998) "Consolidation of clay by sand drain under time-dependent loading", J. Geotech. Engng., ASCE, 124, Issue 1, pp91-94.

Mesri, G. and Khan, A.Q. (2012) "Ground improvement using vacuum loading together with vertical drains", ASCE. Journal of Geotechnical and Geoenvironmental Engineering, 138, Issue 6, pp680-689.

Mohamedelhassan, E., and Shang, J.Q. (2002) "Vacuum and surcharge combined one-dimensional consolidation of clay soils", *Can. Geotech. J*. 39, pp1126-1138.

Nogami, T. and M. Li (2003) "Consolidation of clay with a system of vertical and horizontal drains", J. of Geotechnical and Geoenvironmental Engineering, ASCE, 129, Issue 9, pp838-848.

Onoue, A., Ting, N.-H., Germaine, J.T. and Whitman, R.V. (1991) "Permeability of disturbed zone around vertical drains", Proc. ASCE Geotech. Engng. Congress. 2, pp879-890.

Qian, J.H., Zhao, W.B., Cheung, Y.K. and Lee, P.K.K. (1992) "The theory and practice of vacuum preloading", Computers and Geotechnics, 13, pp103-118.

Qiu, Q.C., Mo, H.H., and Dong, Z.L. (2007) "Vacuum pressure distribution and pore pressure variation in ground improved by vacuum preloading", Canadian Geotechnical Journal, 44, Issue 12, pp1433-1445.

Richart, F.E. (1957) "A review of the theories for sand drains", Journal of the Soil Mechanics and Foundations Division, ASCE, 83, Issue 3, pp1-38.

Robinson, R.G., Indraratna, B. and Rujikiatkamjorn, C. (2012) "Final state of soils under vacuum preloading", Canadian Geotechnical Journal, 49, Issue 6, 729-739.

Rujikiatkamjorn C. and Indraratna, B. (2007) "Analytical solutions and design curves for vacuum-assisted consolidation with both vertical and horizontal drainage", Canadian Geotechnical Journal, 44, pp188-200.

Rujikiatkamjorn, C. and Indraratna, B. (2009) "Design procedure for vertical drains considering a linear variation of lateral permeability within the smear zone", Canadian Geotechnical Journal, 46, Issue 3, pp270-280.

Saowapakpiboon, J., Bergado, D. T., Youwai, S., Chai, J.C., Wanthong, P. and Voottipruex, P (2010) "Measured and predicted performance of prefabricated vertical drains (PVDs) with and without vacuum preloading", Geotextiles and Geomembranes, 28, Issue 1, pp1-11.

Saowapakpiboon, J., Bergado, D. T., Voottipruex, P., Lam, L. G. and Nakakuma, K. (2011) "PVD improvement combined with surcharge and vacuum preloading including simulations", Geotextiles and Geomembranes, 29, Issue 1, pp74-82.

Sathananthan, I. (2005). "Modelling of Vertical Drains with Smear Installed in Soft Clay", PhD Thesis, University of Wollongong, 264p.

Sathananthan, I. and Indraratna, B. (2006) "Laboratory Evaluation of Smear Zone and Correlation between Permeability and Moisture Content", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 132, Issue 7, pp942-945.

Sharma, J. S. and Xiao, D. (2000) "Characterization of a smear zone around vertical drains by large-scale laboratory tests", Can. Geotech. J., 37, Issue 6, 1265-1271.

Shang, J.Q., Tang, M., and Miao, Z. (1998) "Vacuum preloading consolidation of reclaimed land: a case study", Canadian Geotechnical Journal, 35, pp740-749.

Tang, M., and Shang, J. Q. (2000) "Vacuum preloading consolidation of Yaoqiang Airport runway", Géotechnique. 50, Issue 6, pp613-623.

Walker, R. (2006). "Analytical solutions for modeling soft soil consolidation by vertical drains", PhD Thesis, University of Wollongong, 242p.

Walker, R. and Indraratna, B. (2006) "Vertical drain consolidation with parabolic distribution of permeability in smear zone", J. of Geotechnical & Geoenvironmental Engineering, ASCE, 132, Issue 7, pp937-941.

Walker, R., Indraratna, B., and Sivakugan, N. (2009) "Vertical and radial consolidation analysis of multi layered soil using the spectral method", J. of Geotechnical & Geoenvironmental Engineering, ASCE, 135, Issue 5, pp657-663.

Yan, S. W. and Chu, J. (2005) "Soil improvement for a storage yard using the combined vacuum and fill preloading method", Canadian Geotechnical Journal. 42, Issue 4, pp1094-1104.

Zhu, S. L. and Miao, Z. H. (2002) "Recent development and improvement of vacuum preloading method for improving soft soil", Ground Improvement, 6, Issue 2, pp79-83.