

Faculty of Engineering
Faculty of Engineering - Papers

University of Wollongong

Year 2007

Radial consolidation theories and
numerical analysis of soft soil
stabilisation via prefabricated vertical
drains

B. Indraratna* C. Rujikiatkamjorn†
R. Walker‡

*University of Wollongong, indra@uow.edu.au

†University of Wollongong, cholacha@uow.edu.au

‡University of Wollongong

This paper was originally published as: Indraratna, B, Rujikiatkamjorn, C & Walker, R, Radial consolidation theories and numerical analysis of soft soil stabilisation via prefabricated vertical drains, in Yin, JH, Li, XS, Yeung, AT & Desai, CS (eds.) International Workshop on Constitutive Modelling, Hong Kong, January 2007, 155-167.

This paper is posted at Research Online.

<http://ro.uow.edu.au/engpapers/328>

RADIAL CONSOLIDATION THEORIES AND NUMERICAL ANALYSIS OF SOFT SOIL STABILISATION VIA PREFABRICATED VERTICAL DRAINS

Buddhima Indraratna¹, Cholachat Rujikiatkamjorn² and Rohan Walker³

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

INTRODUCTION

Soft clay deposits usually have a low bearing capacity as well as excessive settlement characteristics. Therefore, it is necessary to improve the existing soft soils before commencing construction activities in order to prevent excessive and differential settlement (Richart, 1957). The application of vertical drains and preloading is a popular soil improvement technique. Vertical drains accelerate soil consolidation by providing short horizontal drainage paths for pore water flow, and are used worldwide in many soft soil improvement projects (Holtz et al., 1991; Indraratna et al., 1992; Indraratna and Redana, 2000). The utilisation of geosynthetic prefabricated vertical drains (PVDs) has become an economical and viable option because of their rapid installation with simple field equipment (Shang et al. 1998). In order to control the risk of embankment failure, surcharge embankments are usually raised as a multi-stage exercise with rest periods provided between the loading stages (Jamiolkowski et al. 1983). This may not be possible with busy construction schedules. Application of vacuum load in addition to surcharge fill can further accelerate the rate of settlement to obtain the desired settlement without increasing the excess

¹ Professor of Civil Engineering, Faculty of Engineering, University of Wollongong, Wollongong City, NSW 2522, Australia, E-mail: indra@uow.edu.au

² Research Fellow, Faculty of Engineering, University of Wollongong, Wollongong City, NSW 2522, Australia

³ Research Associate, Faculty of Engineering, University of Wollongong, Wollongong City, NSW 2522, Australia

pore pressure (Kjellman, 1952; Qian et al., 1992). This practice has been employed for land reclamation and port projects (Chu and Yan, 2005). The PVD system has also been employed to distribute vacuum pressure to deep subsoil layers, thereby increasing the consolidation rate. The consolidation process of the vacuum preloading in comparison with the conventional preloading is shown in Fig. (1).

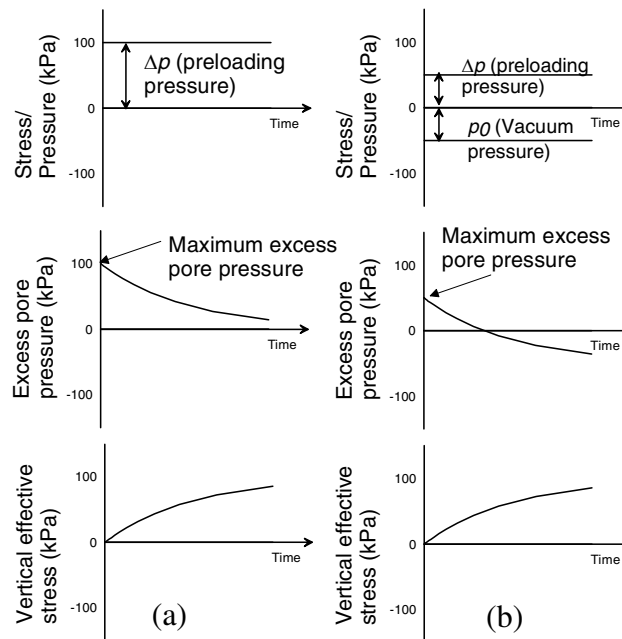


Fig. 1 Consolidation process: (a) conventional loading (b) idealised vacuum preloading (Indraratna et al. 2005c)

In this paper, modified radial consolidation theory considering the effects of the compressibility indices, the variation of soil permeability and the magnitude of preloading are proposed. The smear zone prediction using the Cavity Expansion Theory is discussed based on large scale laboratory results. The equivalent (transformed) plane strain conversion is incorporated in finite element codes, employing the modified Cam-clay theory. A case history is discussed and analysed, including the site of the Second Bangkok International Airport (Thailand) and the predictions are compared with the available field data.

THEORETICAL APPROACH

Solution for Axisymmetric Condition

A rigorous radial consolidation theory incorporating both the smear effect and well resistance was proposed by Barron (1948) and Hansbo (1981). Application of 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 small strain theory over a given stress range with constant volume compressibility (m_v) and a constant coefficient of lateral permeability (k_h). However, the value of m_v varies along the consolidation curve over a wide range of applied pressure (Δp). In the same manner, k_h also changes with the void ratio (e). In this section, the use of compressibility indices (C_c and C_r), which define the slopes of the e - $\log \sigma'$ relationship, and the variation of horizontal permeability coefficient (k_h) with void ratio (e) are considered. The effects of vacuum pressure distribution and parabolic permeability variation in the smear zone are included in the solutions.

The main assumptions of the analysis are given below (Indraratna et al. 2005b):

- (1) According to laboratory measurements, at a few points along the drain in the

large-scale consolidometer (Fig. 2a), the vacuum pressure clearly decreases down the drain length (Indraratna et al., 2004). In the analysis, vacuum pressure is assumed to vary linearly from p_0 at the top of the drain to $k_1 p_0$ at the bottom of the drain.

(2) Homogenous soil is fully saturated and the Darcy's law is adopted. At the external periphery of the unit cell, flow is not allowed to occur (Fig. 2b), hence, only radial (horizontal) flow is permitted.

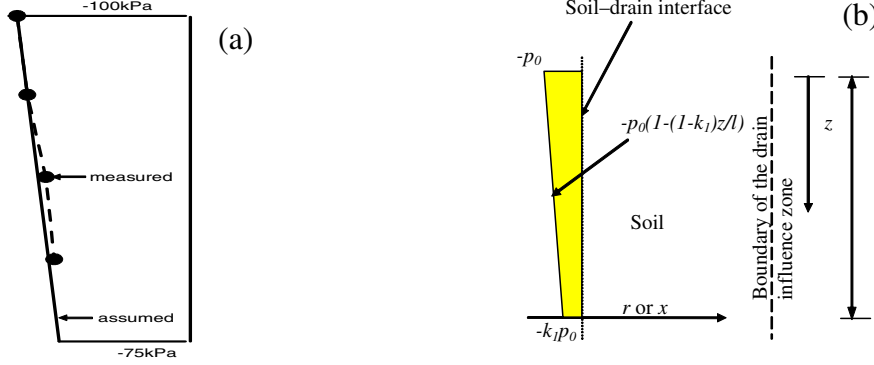


Fig. 2 (a) Distributions of measured negative pore pressure along drain boundary in laboratory testing, (b) distributions of vacuum pressure in model (Indraratna et al. 2005a).

(3) The relationship between the average void ratio and the logarithm of average effective stress in the normally consolidated range (Fig. 3a) can be expressed by: $\bar{e} = e_0 - C_c \log(\sigma' / \sigma'_i)$. If the current vertical effective stress (σ') is less than p'_c , then for this overconsolidated range the recompression index (C_r) is used rather than C_c .

(4) For radial drainage, the horizontal permeability of soil decreases with the average void ratio (Fig. 3b). The relationship between these two parameters is given by Tavenas et al. (1983) by: $\bar{e} = e_0 + C_k \log(k_h / k_{hi})$. The permeability index (C_k) is generally considered to be independent of stress history (p'_c) as explained by Nagaraj et al. (1994).

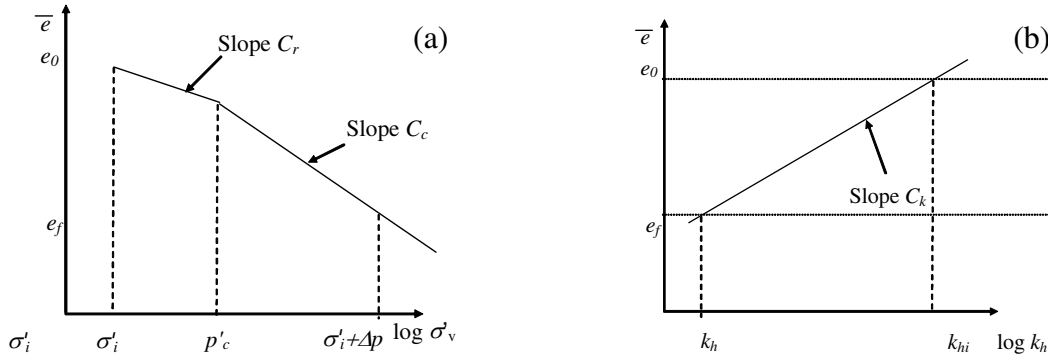


Fig. 3 (a) Soil compression curve and (b) semi-log permeability-void ratio (after Indraratna et al., 2005b)

The dissipation rate of average excess pore pressure ratio ($R_u = \bar{u}_t / \Delta p$) at any time factor (T_h) can be expressed as:

$$R_u = (1 + p_0(1 + k_1) / 2\Delta p) \exp(-8T_h^* / \mu) - p_0(1 + k_1) / 2\Delta p \quad (1)$$

In the above expression,

$$T_h^* = P_{av} T_h \quad (2)$$

$$P_{av} = 0.5 \left[1 + \left(1 + \Delta p / \sigma'_i + p_0(1 + k_1) / 2\sigma'_i \right)^{1 - C_c / C_k} \right] \quad (3)$$

$$T_h = c_h t / d_e^2 \quad (4)$$

where, $n = d_e/d_w$, $s = d_s/d_w$, $d_e =$ equivalent diameter of cylinder of soil around drain, $d_s =$ diameter of smear zone and $d_w =$ diameter of drain well, $k_h =$ average horizontal permeability in the undisturbed zone (m/s), and $k'_h =$ average horizontal permeability in the smear zone (m/s). $\Delta p =$ preloading pressure, T_h is the dimensionless time factor for consolidation due to radial drainage, and $\mu =$ a group of parameters representing the geometry of the vertical drain system and smear effect. Hansbo (1981) assumed the smear zone to have a reduced horizontal permeability that is constant throughout this zone. The μ parameter can be given by:

$$\mu = \ln n / s + k_h / k'_h \ln s - 0.75 \quad (5a)$$

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

$$\mu_p = \ln\left(\frac{n}{s}\right) - \frac{3}{4} + \frac{\kappa(s-1)^2}{(s^2 - 2\kappa s + \kappa)} \ln\left(\frac{s}{\sqrt{\kappa}}\right) - \frac{s(s-1)\sqrt{\kappa(\kappa-1)}}{2(s^2 - 2\kappa s + \kappa)} \ln\left(\frac{\sqrt{\kappa} + \sqrt{\kappa-1}}{\sqrt{\kappa} - \sqrt{\kappa-1}}\right) \quad (5b)$$

In the above expression, $\kappa = k_h/k_0$.

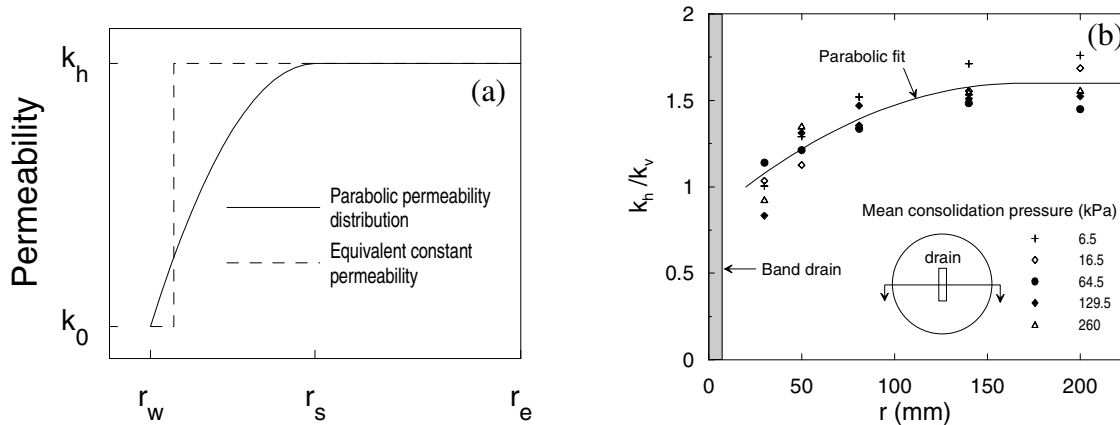


Fig. 4 (a) Permeability distribution and (b) ratio of horizontal to vertical permeability along radial distance from drain in large scale consolidometer (Walker and Indraratna, 2006)

Since the relationship between effective stress and strain is non-linear, the average degree of consolidation can be described based on either excess pore pressure (U_p) or strain (U_s). U_p indicates the rate of dissipation of excess pore pressure whereas U_s shows the rate of development of the surface settlement. Normally, $U_p \neq U_s$ except when the effective stress and strain is a linear relationship, which is in accordance with Terzaghi's one-dimensional theory. Therefore, the average degree of consolidation based on excess pore pressure can be obtained as follows:

$$U_p = 1 - R_u \quad (11)$$

The average degree of consolidation based on settlement (strain) is defined by:

$$U_s = \rho / \rho_\infty \quad (12)$$

The associated settlements (ρ) are then evaluated by the following equations:

$$\rho = HC_r / (1 + e_0) \log(\sigma' / \sigma'_i), \quad \sigma'_i \leq \sigma' \leq p'_c \quad (13a)$$

$$\rho = HC_c / (1 + e_0) [C_r \log(p'_c / \sigma'_i) + C_c \log(\sigma' / p'_c)] \quad p'_c \leq \sigma' \leq \sigma'_i + \Delta p \quad (13b)$$

$$\rho = HC_c / (1 + e_0) \log(\sigma' / \sigma'_i) \quad \text{for normally consolidated clay} \quad (13c)$$

It is noted that ρ_∞ can be obtained by substituting $\sigma' = \sigma'_i + \Delta p$ into the above equations, where: ρ = settlement at a given time, ρ_c = total primary consolidation settlement, σ'_i = effective in-situ stress, σ' = effective stress, C_c = compression index, C_r = recompression index and H = compressible soil thickness.

When the value of C_c/C_k approaches unity and p_0 becomes zero, the authors' solution converges to the conventional solution proposed by Hansbo (1981):

$$R_u = \exp(-8T_h/\mu) \quad (14)$$

Conversion Procedure for Equivalent Plane Strain Analysis

Indraratna and Redana (2000) and Indraratna et al. (2005) showed that, based on the appropriate conversion procedure by matching the degree of consolidation at a given time step, plane strain multi-drain analysis can be employed to predict soft soil behavior improved by vertical drain and vacuum preloading.

Using the geometric transformation in Fig. 5, the corresponding ratio of the smear zone permeability to the undisturbed zone permeability is obtained by (Indraratna et al., 2005a):

$$\frac{k_{s,ps}}{k_{h,ps}} = \beta / \left(\frac{k_{h,ps}}{k_{h,ax}} \left[\ln\left(\frac{n}{s}\right) + \frac{k_{h,ax}}{k_{s,ax}} \ln(s) - \frac{3}{4} \right] - \alpha \right) \quad (15)$$

where, $\alpha = 0.67 \times (n-s)^3 / n^2(n-1)$ and $\beta = \frac{2(s-1)}{n^2(n-1)} \left[n(n-s-1) + \frac{1}{3}(s^2 + s + 1) \right]$

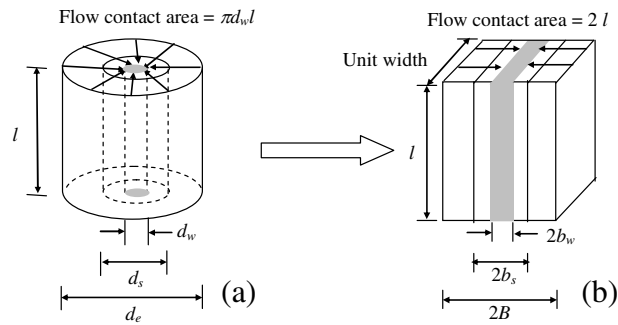


Fig. 5 Unit cell analysis: (a) axisymmetric condition, (b) equivalent plane strain condition (after Indraratna et al., 2005a)

Ignoring both smear and well resistance effects the simplified ratio of equivalent plane strain to axisymmetric permeability in the undisturbed zone can be attained, hence,

$$k_{h,ps} / k_{h,ax} = \frac{2}{3} \frac{(n-1)^2}{n^2} / [\ln(n) - 0.75] \quad (16)$$

The equivalent vacuum pressure can now be expressed by:

$$p_{0,ps} = p_{0,ax} \quad (17)$$

Multilayered Consolidation Theory

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):

$$\frac{m_v}{\bar{m}_v} \frac{\partial \bar{u}}{\partial t} = - \left[dT_h \frac{\eta}{\bar{\eta}} \bar{u} - dT_v \frac{\partial}{\partial Z} \left(\frac{k_v}{\bar{k}_v} \frac{\partial \bar{u}}{\partial Z} \right) \right] \quad (18)$$

$$\text{where, } Z = \frac{z}{H}, \quad dT_v = \frac{\bar{c}_v}{H^2}, \quad dT_h = \frac{2\bar{\eta}}{\gamma_w \bar{m}_v}, \quad \bar{c}_v = \frac{\bar{k}_v}{\gamma_w \bar{m}_v}, \quad \eta = \frac{k_h}{r_e^2 \mu}$$

In the preceding, \bar{u} =average excess pore pressure, t =time, z =depth, H =depth of soil, γ_w =unit weight of water, m_v =volume compressibility and k_v =vertical permeability. Equation (18) has been normalized with respect to convenient reference values of each property indicated by the over-bar notation. Vertical flow is based on the average hydraulic gradient. Walker (2006) presented solutions to Eq. (18) for multiple layers (see

Fig.6) 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 function:

$$\alpha(Z) = \alpha_l \text{UnitStep}(Z - Z_{l-1}) \text{UnitStep}(Z_l - Z) \quad (18a)$$

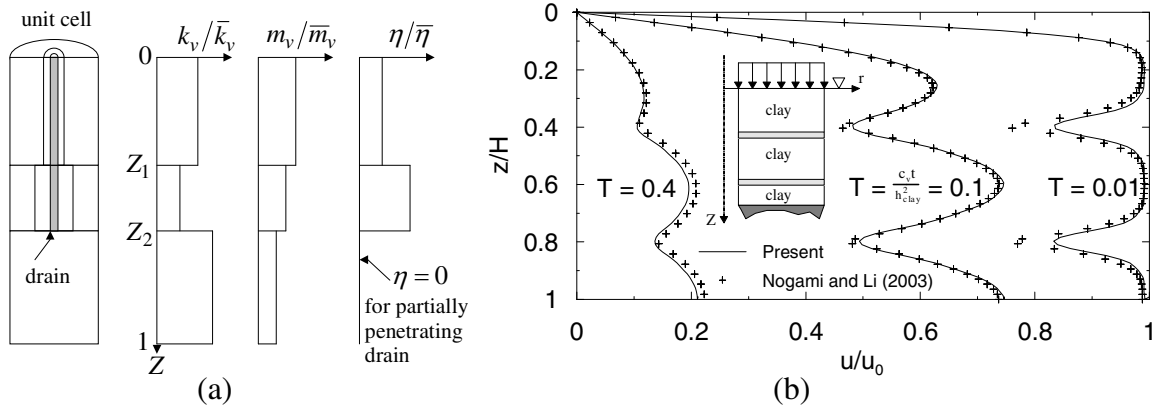


Fig.6 (a) Multi-layered consolidation properties and (b) model verification: multi-layer equal-strain vs free-strain

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

$$\bar{u}(Z,t) \approx \Phi \mathbf{A} \approx [\phi_1(Z) \ \phi_2(Z) \ \dots \ \phi_N(Z)] [A_1(t) \ A_2(t) \ \dots \ A_N(t)]^T \quad (19)$$

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

$$\phi_j(Z) = \sin(M_j Z) \quad \text{where, } M_j = \begin{cases} j\pi & \text{for PTPB} \\ \frac{\pi}{2}(2j-1) & \text{for PTIB} \end{cases} \quad (19a)$$

The Galerkin procedure requires that the error in Eq. (19) is orthogonal to each basis function, hence:

$$\int_0^1 \phi_i L(\Phi \mathbf{A}) dZ = 0 \quad (20)$$

where, L describes the differential operations in Eq. (18). Combining Eqs. (18), (18a), (19) and (20) yields a set of coupled ordinary differential equations for A_j , which in matrix form reads:

$$\mathbf{\Gamma A}' = -\mathbf{\Psi A} \quad (21)$$

where,

$$\mathbf{\Gamma}_{ij} = \Lambda_{ij}^- m_{vl} / \bar{m}_v, \quad \mathbf{\Psi}_{ij} = dT_v M_j M_i \Lambda_{ij}^+ k_{vl} / \bar{k}_v + dT_h \Lambda_{ij}^- \eta_l / \bar{\eta} \quad (21a)$$

$$\Lambda_{ij}^\pm = \begin{cases} SN(M_j - M_i) \pm SN(M_j + M_i) & i \neq j \\ (Z_l - Z_{l-1}) \pm SN(M_j + M_i) & i = j \end{cases}, \quad SN[\beta] = (\sin(\beta Z_l) - \sin(\beta Z_{l-1})) / \beta \quad (21b)$$

Based on the eigen problem of Eq. (21), under instantaneous loading the solution to Eq. (19) is:

$$\bar{u}(Z, t) \approx \mathbf{\Phi v E}(\mathbf{\Gamma v})^{-1} [\theta_1 \quad \theta_2 \quad \dots \quad \theta_N]^T \quad (22)$$

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

$$\mathbf{E}(t) = \text{diag}[\exp(-\lambda_1 t) \quad \exp(-\lambda_2 t) \quad \dots \quad \exp(-\lambda_N t)] \quad (22a)$$

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:

$$\theta_i = 2(1 - \cos(M_i)) / M_i \quad (22b)$$

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

$$\bar{\phi}_j(Z_1, Z_2) = (\cos(M_j Z_1) - \cos(M_j Z_2)) / M_j (Z_2 - Z_1) \quad (22c)$$

Nogami and Li (2003) developed a free-strain approach for calculating the excess pore pressure distribution for multi-layered soil with both vertical and radial drainage. An example problem is presented with a soil system consisting of two identical thin sand layers (height h_s) separating three identical clay layers (height h_c). Soil properties 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 Fig. 6b. Both methods are in close agreement except for slight deviations in the thin sand layers 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.

SMEAR ZONE DETERMINATION 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 also employed to estimate the smear zone extent. The prediction is then compared with the laboratory results based on permeability and water content variations.

Cavity expansion analysis has attracted the attention of numerous researchers because it has exciting numerous geomechanics applications. The extent of “smear zone” caused by mandrel installation can be estimated using the Cylindrical Cavity Expansion theory incorporating the modified Cam-clay model (MCC) (Sathananthan, 2005). The detailed theoretical developments are explained elsewhere by Collins & Yu (1996) and Cao et al. (2001), so only a brief summary is given below. The yielding criterion for soil obeying the MCC model is:

$$\eta = M \sqrt{\left(\frac{p'_c}{p'} \right) - 1} \quad (23)$$

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. Stress ratio at any point can be determined as follows:

$$\ln\left(1 - \frac{(a^2 - a_0^2)}{r^2}\right) = -\frac{2(1+\nu)}{3\sqrt{3}(1-2\nu)} \frac{\kappa}{\nu} \eta - 2\sqrt{3} \frac{\kappa\Lambda}{\nu M} f(M, \eta, OCR) \quad (24)$$

where,

$$f(M, \eta, OCR) = \frac{1}{2} \ln\left[\frac{(M + \eta)(1 - \sqrt{OCR - 1})}{(M - \eta)(1 + \sqrt{OCR - 1})}\right] - \tan^{-1}\left(\frac{\eta}{M}\right) + \tan^{-1}(\sqrt{OCR - 1}) \quad (25)$$

In the above expression, a = radius of the cavity, a_0 = initial radius of the cavity, ν = Poisson's ratio, κ = slope of the overconsolidation line, ν = specific volume, OCR = overconsolidation ratio and $\Lambda = 1 - \kappa/\lambda$ (λ is the slope of the normal consolidation line). Finally, the corresponding mean effective stress, in terms of deviatoric stress, total stress and excess pore pressure, can be expressed by the following expressions:

$$q = \eta p' \quad (26)$$

$$p' = p'_0 \left[OCR / \left(1 + (\eta/M)^2\right)\right]^\Lambda \quad (27)$$

$$p = \sigma_{rp} - q / \sqrt{3} + 2 / \sqrt{3} \int_r^{r_p} q / r dr \quad (28)$$

Employing Equations (26)-(28), the excess pore pressure due to mandrel installation (Δu) can be determined by:

$$\Delta u = (p - p_0) - (p' - p'_0) \quad (29)$$

In the above, p_0 = initial total mean stress. The extent of the smear zone can be suggested as the region in which the excess pore pressure exceeds the initial overburden pressure (σ_{v0}). This is because, in the region surrounding the drains ($r < r_p$), the soil properties (permeability and soil anisotropy) are altered severely at a radial distance where $\Delta u = \sigma_{v0}$.

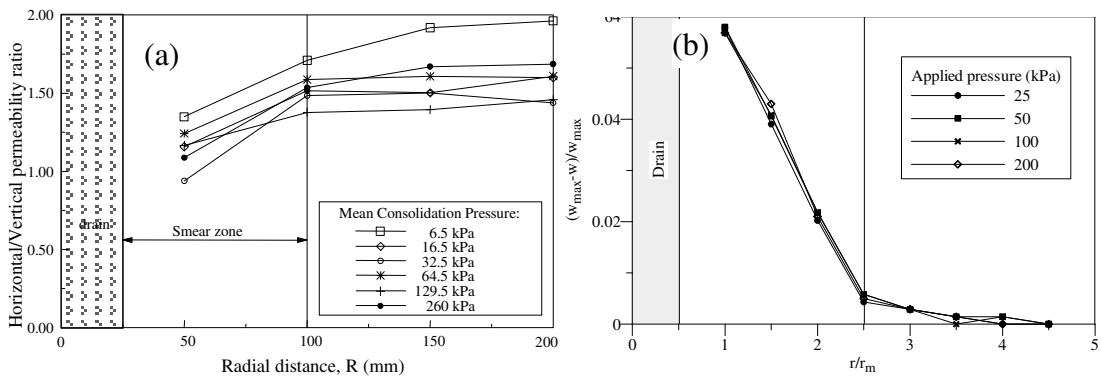


Fig. 7 Smear zone determination (a) permeability approach (Indraratna and Redana, 1998) (b) water content approach (Sathanathan and Indraratna, 2006)

The smear zone extent can be determined either by the permeability or water content variation along the radial distance (Indraratna and Redana, 1998; Sathanathan and Indraratna, 2006). Fig. 7 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. It is seen that the smear zone radius is about 100mm or 2.5 times the mandrel radius which is in agreement the prediction using the cavity expansion theory.

APPLICATION TO A CASE HISTORY

Indraratna et al. (2004) analysed the performance of test embankments constructed at the Second Bangkok International Airport (SBIA), Thailand. At this site, 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. 15-17) for the FEM analysis. The embankment cross section and typical finite element mesh employing the multi-drain analysis are given in Fig. 8. The test embankment was raised and stabilised with PVDs installed in a triangular pattern with 1m spacing to a depth of 15m. The 100mm x 3mm PVDs (Mebra) were used.

The embankment loading was simulated by the sequential construction history (Fig. 9a). The following 4 models were numerically examined under plane strain multi-drain analysis (Indraratna et al., 2004):

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

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

Model 3 – Similar to Model 2, but the vacuum pressure was varied with depth and time as occurs in the field.

Model 4– Conventional surcharge alone with no vacuum pressure.

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

Depth (m)	e_o	λ	κ	ν	M	k_h (m/s)	k'_h (m/s)	k_{hp} (m/s)	k'_{hp} (m/s)	Γ (kN/m ³)
0-2.0	1.8	0.3	0.03	0.3	1.2	30×10^{-8}	3.01×10^{-9}	8.98×10^{-9}	5.86×10^{-10}	16.0
2.0-8.5	2.8	0.73	0.08	0.3	1.0	1.3×10^{-8}	1.27×10^{-9}	3.80×10^{-9}	2.48×10^{-10}	14.5
8.5-10.5	2.4	0.5	0.05	0.25	1.2	6.0×10^{-9}	6.02×10^{-10}	1.80×10^{-9}	1.17×10^{-10}	15.0
10.5-13	1.8	0.3	0.03	0.25	1.4	2.6×10^{-9}	2.55×10^{-10}	7.60×10^{-10}	4.96×10^{-11}	16.0
13-18	1.2	0.1	0.01	0.25	1.4	6.0×10^{-10}	6.02×10^{-11}	4.15×10^{-11}	2.71×10^{-12}	18.0

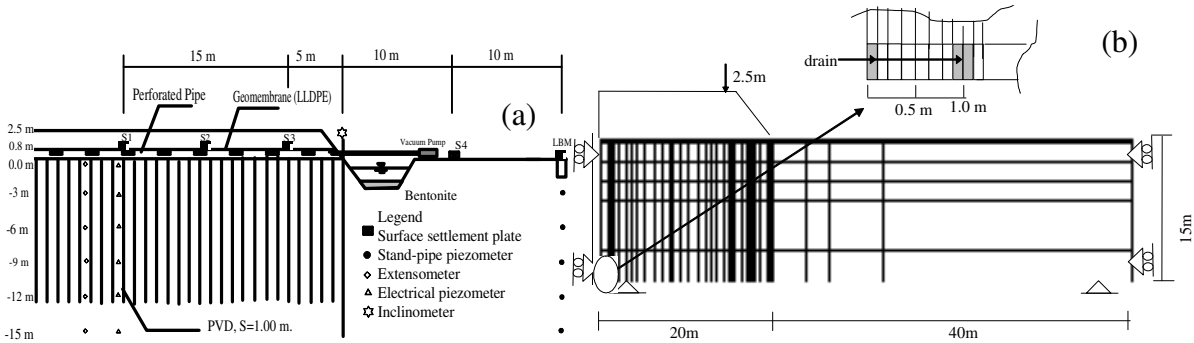


Fig. 8 (a) Cross-section of an embankment with the subsoil profile and (b) finite element discretisation of embankment foundation (Indraratna et al., 2004)

Figure 9b shows settlement predictions together with the measured settlement. Model 3 predictions agree well with the field data. The assumed time-dependent variation of vacuum pressure based on surface measurements improves the accuracy of settlement predictions. The

measured and predicted excess pore pressures along the centerline of the embankment at a depth of 3 m below the ground surface are compared in Fig. 10a. Model 3 that includes the time-dependent vacuum pressure variation is in agreement with the field measurements. All other models that do not consider the time-dependent vacuum pressure variation are unable to predict the field behaviour to an acceptable accuracy.

Measured and predicted lateral deformation for the inclinometer installed away from the centerline of the embankment (after 150 days) is shown in Fig. 10b. All 3 models incorporating vacuum pressure cause 'inward' movements. The effect of the compacted crust is not clearly reflected by the field data suggesting that the depth of the crust is no more than 1m in the field, whereas the numerical analysis assumed crust of 2m in thickness. The effect of the loss of vacuum head causes increased lateral movements more towards Model 4.

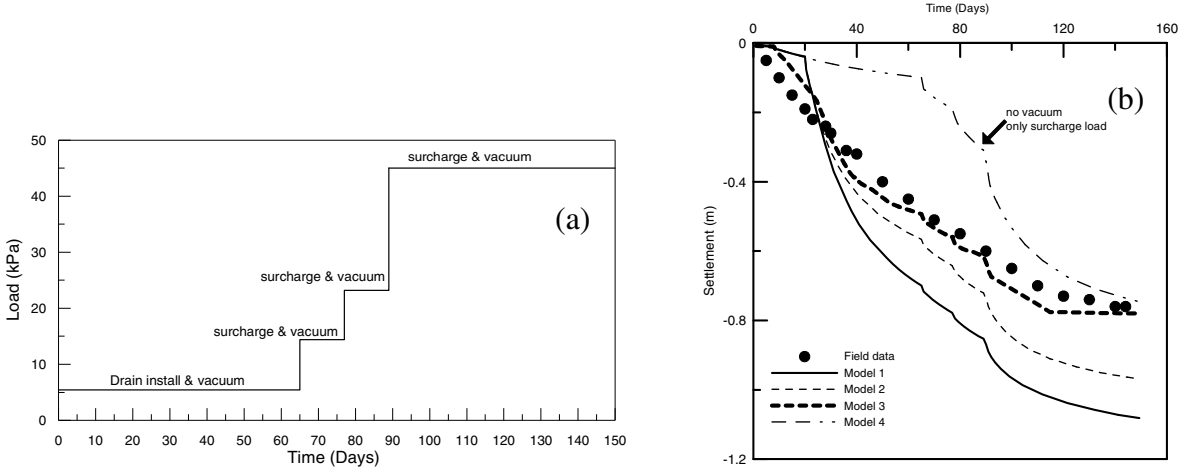


Fig. 9 (a) Stage loading and (b) settlement predictions (Indraratna et al. 2004)

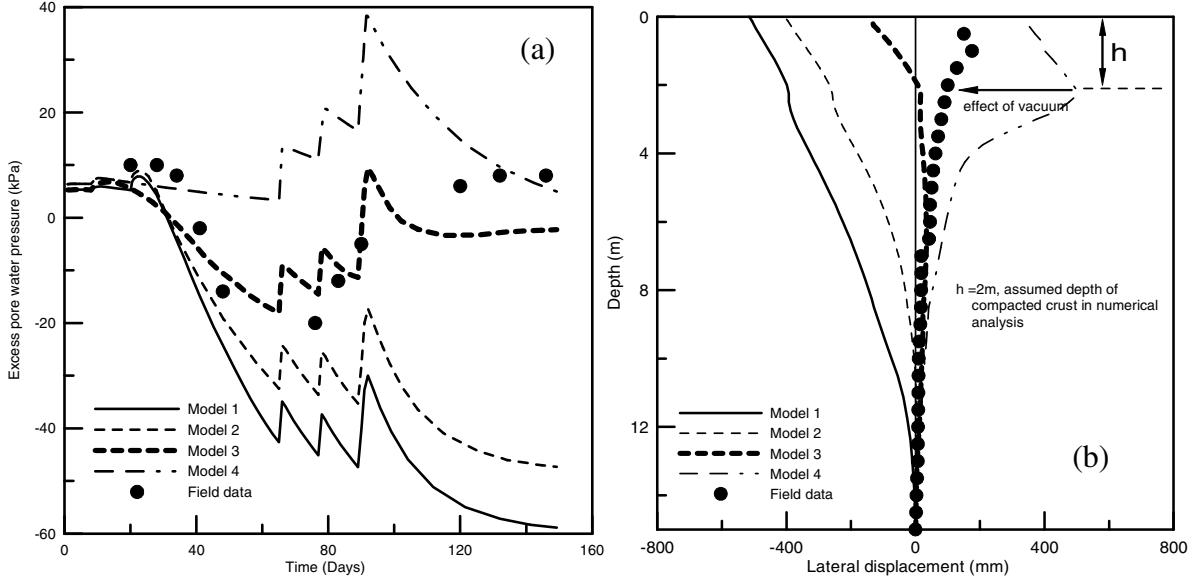


Fig. 10 (a) Excess pore water pressure predictions and (b) lateral displacement predictions (Indraratna et al. 2004)

Field Observation of Retarded Pore Pressure Dissipation

It has been observed in some case studies that in spite of PVDs, excess pore water pressures do not always dissipate as expected. This is often attributed to filter clogging, extreme reduction of the lateral permeability of soil, damage to piezometer tips etc. However, recent numerical analysis suggests that very high lateral strains and corresponding stress redistributions (e.g. substantial heave at the embankment toe) can also contribute to retarded rate of pore pressure dissipation, as shown in Fig 11.

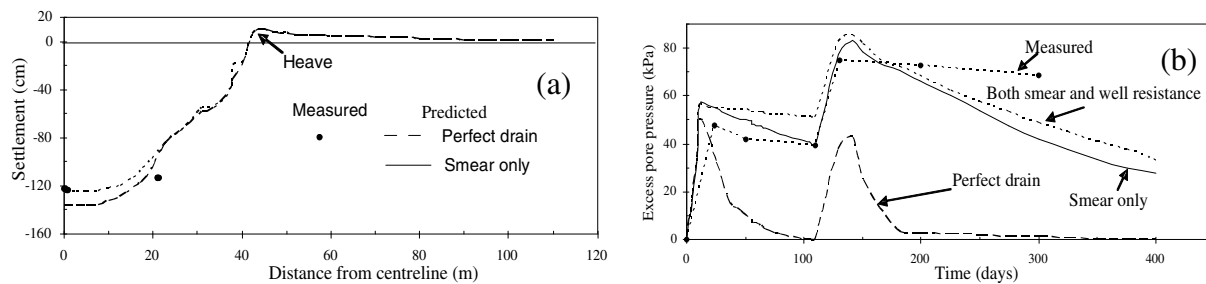


Fig. 11 (a) Surface settlement profile after 400 days and (b) excess pore water pressure variation at piezometer location, P6 (Indraratna & Redana, 2000)

CONCLUSIONS

A system of vertical drains combined with vacuum preloading is an effective method to accelerate soil consolidation by promoting radial flow. A revised analytical model for soft clay improved by vertical drains incorporating the compressibility indices (C_c and C_r) and vacuum surcharge has been introduced. The spectral method was proposed to predict layered soil consolidation. The variation of horizontal permeability coefficient (k_h) with the stress level was also included. The parabolic decay of permeability in the smear zone associated with drain installation is considered to represent the actual variation. The 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 permeability and water content approaches.

The vacuum pressure application increases the rate of pore pressure dissipation due to the increased hydraulic gradient towards the drain. The occurrence of soil unsaturation at the PVD boundary due to 'dry' drain installation could retard the pore pressure dissipation, as clearly reflected by the corresponding difference in settlement curves. The use of appropriate suction-permeability relationships is important in obtaining realistic predictions if the soil adjacent to the PVDs becomes unsaturated. Numerical refinement may require the use of improved procedures for accurate modelling of the surface crust (over-consolidated) that does not strictly obey the modified cam-clay theory.

There is no doubt that a system of vacuum-assisted consolidation via PVDs is a useful and practical approach for accelerating radial consolidation. The vacuum effect may diminish significantly with depth due to various practical limitations such as improper sealing, and the nature of soil conditions (presence of fissures and macro-pores). Therefore, the assumption of decreasing suction values along the drain depth could be justified in finite element modelling. Further investigation with 'instrumented PVD' in the field may provide further insight to the vacuum pressure distribution with depth in the stabilisation of soft clays. Such a system eliminates the need for a high surcharge load, as long as air leaks can be prevented. Accurate modeling of vacuum preloading requires both laboratory and field studies to quantify the nature of vacuum pressure distribution within a given formation and drain system.

ACKNOWLEDGEMENTS

The authors wish to thank the CRC for Railway Engineering (Australia) for its support. A number of current and past PhD students, namely, Dr. I. Redana, Dr. C. Bamunawita, and Dr. I. Sathanathan have also contributed to the contents of this paper. More elaborate details of the contents discussed in this paper can be found in previous publications of the first author and his research students in the ASCE and Canadian Geotechnical Journals, since mid 1990's.

REFERENCES

- Barron, R.A. (1948). Consolidation of fine-grained soils by drain wells. Transactions ASCE, 113, 718-754.
- Cao, L. F., Teh, C. I., and Chang, M. F. (2001). Undrained Cavity Expansion in Modified Cam Clay I: Theoretical Analysis." Geotechnique, 51(4); 323-334.
- 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, 91-118.
- Collins, I. F. and Yu, H. S. 1996. Undrained Cavity Expansion in Critical State Soils. *Int. J. for Numerical and Analytical Methods in Geomechanics*, 20, 489-516.
- 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: 677-682.
- 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(2): 180-185.
- 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: 133-145.
- 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(1), 12-33.
- Indraratna, B., Bamunawita, C., and Khabbaz, H. (2004). Numerical modeling of vacuum preloading and field applications. *Canadian Geotechnical 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., Rujikiatkamjorn C., and Sathananthan, I. (2005b). Radial consolidation of clay using compressibility indices and varying horizontal permeability. *Canadian Geotechnical Journal*, 42, 1330-1341.
- 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, 199-230.
- Jamiolkowski, M., Lancellotta, R., and Wolski, W. (1983). Precompression and speeding up consolidation. *Proc. 8th ECSMFE*, 1201-1206.
- Kjellman, W. (1952). Consolidation of clayey soils by atmospheric pressure. *Proceedings of a conference on soil stabilization, Massachusetts Institute of Technology, Boston*, 258-263.
- Mohamedelhasan, E., and Shang, J.Q. (2002). Vacuum and surcharge combined one-dimensional consolidation of clay soils. *Can. Geotech. J.* 39, 1126-1138.
- Nagaraj, T.S., Pandian, N.S. and Narashima Raju, P.S.R., (1994). Stress-state-permeability relations for overconsolidated clays *Géotechnique*, 44(2), 349-352.
- 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(9), 838-848.
- 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, 879-890.
- 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: 103-118.
- 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. (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(7), 942-945.
- Sharma, J. S. and Xiao, D. (2000). Characterization of a smear zone around vertical drains by large-scale laboratory tests. *Can. Geotech. J.* 37(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, 740-749.
- Tavenas, P., Jean, P., Leblond, P., and Leroueil, S. (1983). The permeability of natural soft clays. Part II: permeability characteristics. *Canadian Geotechnical Journal*. 20, 645-659.
- 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(7), 937-941.