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Soft Clay Stabilization with Geosynthetic Vertical Drains beneath Road and Railway Embankments: A Critical Review of Analytical Solutions and Numerical Analysis

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ABSTRACT

In the wide array of existing ground improvement schemes, the use of vertical drains with vacuum preloading is considered as the most effective and economical method for improving soft clays (normally consolidated to lightly over-consolidated) prior to construction of infrastructure. Vertical drains installed to significant depths promote radial flow inducing consolidation rapidly enhancing the shear strength of the compressed ground. In this paper, the analytical solutions based on lateral soil permeability (parabolic variation) are discussed considering the variation of vacuum pressure with depth along the prefabricated vertical drains (PVD). Using the Cavity Expansion Theory (CET), the *smear zone* caused by the installation of PVD by steal mandrel was predicted and compared with laboratory measurements obtained from large-scale radial consolidation tests. The effects of drain unsaturation and vacuum pressure along the drain length are also discussed. The numerical analyses incorporating equivalent plane strain solutions were performed to predict the soil responses based on two selected case histories in Thailand. The research findings provided insight as to which of the above aspects needed to be simulated accurately in numerical modelling. The application of cyclic loading on PVD stabilized ground was also examined using a finite element approach under railway embankment. It is demonstrated that short drains less than 8 m installed beneath tracks are still useful for effective dissipation of cyclic

pore pressures and curtailing unacceptable lateral movement immediately below the track level, at the same time avoiding excessive settlement of the track in the short-term.

INTRODUCTION

Inevitable coastal urbanization in many countries has compelled engineers to construct heavy infrastructure including major highways and railways over soft marine and alluvial deposits (Indraratna and Chu, 2005). Coastal clays often have low bearing capacity and high compressibility, causing excessive and differential settlements upon loading. Preloading with geosynthetic drains has been usually applied to thick, normally consolidated clayey deposits with relatively low permeability. In brief, preloading is the application of a pre-construction surcharge load on the site until most of the primary consolidation is achieved (Richart, 1959). Prefabricated vertical drains (PVD) promote accelerated radial drainage by shortening the otherwise vertical drainage path (Jamiolkowski et al., 1983; Nicholson and Jardine 1982). In the case of land reclamation sites on which the high surcharge embankments could not be raised, the application of vacuum pressure with surcharge loading would be the most appropriate option (Shang et al., 1998).

In this paper, radial consolidation theories incorporating the effects of the compressibility indices, the variation of soil permeability and the magnitude of vacuum preloading are examined. The smear zone prediction based on the Cavity Expansion Theory is compared with the large scale laboratory results. The conversion procedure for plane strain condition is used in finite element codes, employing the modified Cam-clay theory. Case histories are discussed and analysed, including the site at the New Bangkok International Airport (Thailand) and the predictions are compared with the available field data. The use of short drain to expedite cyclic excess pore pressure under rail tracks is also demonstrated.

CONCEPTS OF VACUUM PRELOADING VIA PREFABRICATED VERTICAL DRAINS

In order to enhance vertical drain performance, the vacuum preloading method was initially proposed in Sweden by Kjellman (1952) for wick drains made of compressed paper. Figure 1a shows a modern vacuum preloading layout with a network of vertical and horizontal drains (Indraratna et al., 2005c). After installing PVDs, the installation of some horizontal drains in the transverse and longitudinal direction under a sand blanket is illustrated, whereby these drains are then connected to the edge of a peripheral Bentonite slurry trench. The vacuum pumps are connected to the discharge system extending from the trenches. The suction head generated by the pump accelerates the dissipation of excess pore water pressure in the soil towards the drains and the surface by imparting a considerable pressure gradient (Qian et al., 1992).

Figure 1b shows the mechanism of the vacuum preloading as a spring analogy (Chu and Yan, 2005). The effective stress increases through vacuum load while the total stress remains constant. The main advantages are as follows (Cognon et al., 1992):

(i)   Increase the shear strength of soil by decreasing the void ratio;
(ii) The lateral movement due to suction is compressive, thereby, decreasing risk of shear failure. However, any excessive 'inward' movement should be carefully monitored.
(iii) The preloading time can be minimised to obtain the same level of post-construction settlement;
(iv) Along the PVD, the vacuum head can distribute to a greater depth of the subsoil; and,
(v)   With vacuum pressure, the unsaturated condition at the soil-drain interface may be improved, resulting in an increased rate of initial consolidation.

$$u_0 = p_a$$
$$\Delta\sigma^{\backprime} = p_a - (u_0 - \Delta u) = \Delta u$$

(a)   (b)
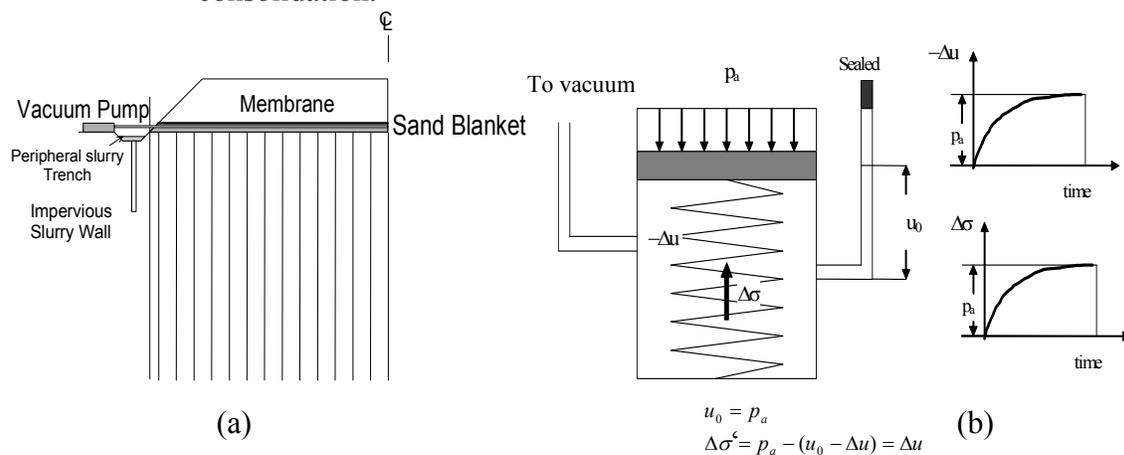
Figure 1 (a) Schematic diagram of PVDs incorporating vacuum preloading system (Indraratna et al., 2005c) and (b) Spring analog of vacuum consolidation (adopted from Chu and Yan, 2005)

FACTORS CONTROLLING THE VERTICAL DRAIN EFFICIENCY

Since the performance of a PVD system can be influenced by many factors, it is essential for designers to account the following factors.

Smear Effect: Smear zone is the inevitable soil disturbance that occurs during the installation of PVD by a steel mandrel, which creates a substantial reduction in soil permeability retarding the rate of consolidation. The disturbance depends on the method of installation, total mandrel cross sectional area and the shape of the drain anchor. The static pushing procedure is preferred for driving the mandrel into the soft ground, whereas the dynamic methods usually cause a greater disturbance of the surrounding soil (e.g. vibrating hammer). The mandrel size should be as close as possible to the size of

the drain to minimise smear, but it should be stiff enough to penetrate the deep soil layers without buckling. Akagi (1977) showed that when a closed-end mandrel is driven into a saturated clay, the build up of high excess pore water pressure associated with ground heave and lateral displacement invariably occurs. Sathananthan (2005) used the undrained Cylindrical Cavity Expansion (CET) theory (Collins & Yu, 1996; Cao et al., 2001) to estimate the extent of smear caused by mandrel driving. Only a short description of the application of CET is given below.

Sathananthan (2005) proposed that the extent of the smear zone can be considered as the region in which the excess pore pressure tends to approach and even exceed the initial overburden pressure (σ'_{v0}) (Figure 2). This is because, in the region surrounding the drains ($r<r_p$), the soil properties (permeability and soil anisotropy) are altered severely at radial distance (r_p) where $\Delta u = \sigma_{v0}$. The excess pore pressure due to mandrel driving (Δu) is simply defined by:

$$\Delta u = (p - p_0) - (p' - p'_0) \tag{1}$$

where, p_0 = initial total mean stress.

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

$$q = \eta p' \tag{3}$$

$$p = \sigma_{rp} - q/\sqrt{3} + 2/\sqrt{3} \int_r^{r_p} q/r \, dr \tag{4}$$

For soil obeying the Modified Cam-Clay (MCC) model, the yielding criterion is(Roscoe and Burland, 1968):

$$\eta = M \sqrt{\left(p'_c / p' \right) - 1} \tag{5}$$

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 (Cao et al., 2001):

$$\ln\left(1 - \frac{(a^2 - a_0^2)}{r^2} \right) = -\frac{2(1+v)}{3\sqrt{3}(1-2v)} \frac{\kappa}{v} \eta - 2\sqrt{3} \frac{\kappa\Lambda}{vM} f(M, \eta, OCR) \tag{6}$$
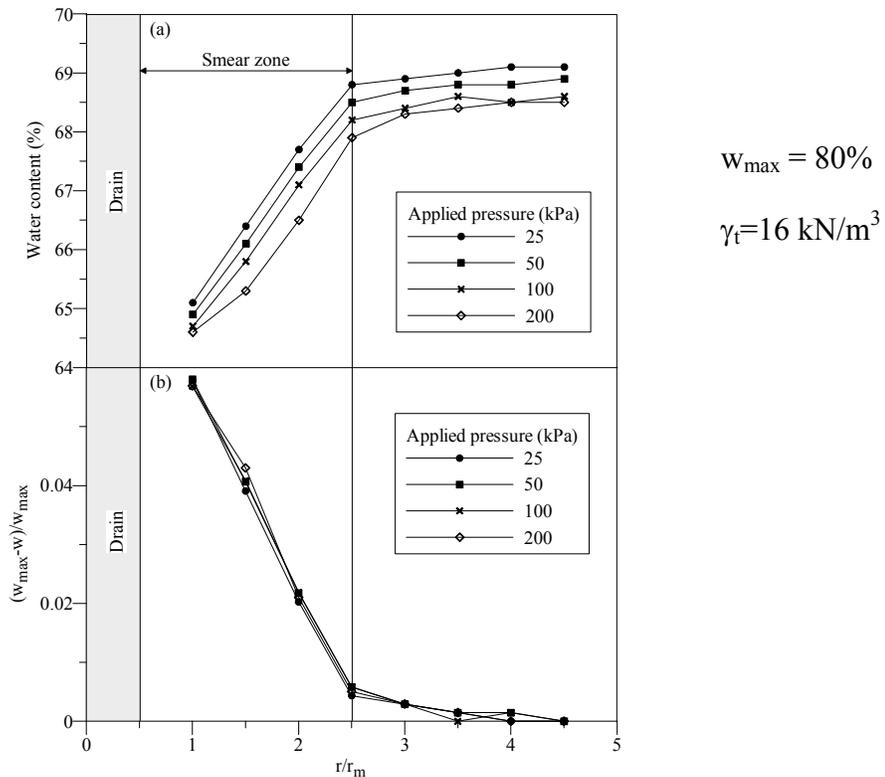
where,

$$f(M, \eta, OCR) = \frac{1}{2} \ln\left[\frac{(M+\eta)(1 - \sqrt{OCR - 1})}{(M-\eta)(1 + \sqrt{OCR - 1})} \right] - \tan^{-1}\left(\frac{\eta}{M} \right) + \tan^{-1}\left(\sqrt{OCR - 1} \right) \tag{7}$$

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, υ = specific volume, OCR = over-consolidation ratio and $\Lambda = 1 - \kappa/\lambda$ (λ is the slope of the normal consolidation line).
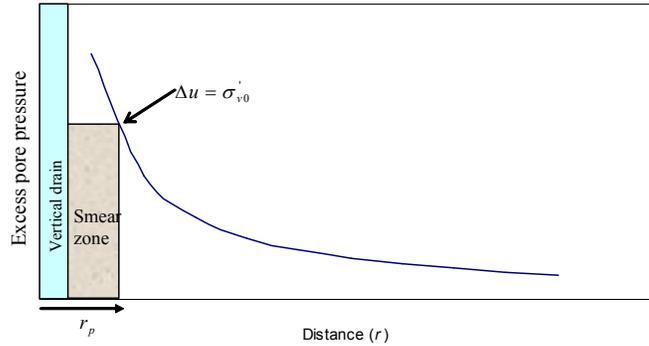
Figure 2 Smear zone prediction by the Cavity Expansion Theory

w_{max} = 80%

γ_t=16 kN/m^3

Figure 3 (a) Water content variation and (b) normalized water content with radial distance at a depth of 0.5m from surface (Sathananthan and Indraratna, 2006)

Indraratna and Redana (1997) and Sathananthan and Indraratna (2006) conducted large-scale consolidometer testing at University of Wollongong to determine the extent

of smear. It was suggested that the smear zone extent could be quantified either by permeability variation or water content variation along the radial distance (Figures 3 and 4). The variation of the water content with radial distance is shown in Figure 3, where it decreases towards the drain. Figure 4 shows the variation of the horizontal to vertical permeability ratio (k_h/k_v) at different consolidation pressures along the radial distance. The relationship between the diameter of the smear zone (d_s) and the equivalent diameter of the drain (d_w) based on the laboratory testing can be given by the expression: $d_s = (3 \text{ to } 4)d_w$. The smear zone parameters proposed by different researchers are summarized in Table 1.

Table 1 Proposed smear zone parameters (Xiao, 2002)

| Source	Extent	Permeability	Remarks
Barron (1948)	$r_s=1.6r_m$	$k_h/k_s=3$	Assumed
Hansbo (1979)	$r_s=1.5\sim3r_m$	Open	available literature
Hansbo (1981)	$r_s=1.5r_m$	$k_h/k_s=3$	Assumed in case study
Bergado et al. (1991)	$r_s=2r_m$	$k_h/k_v=1$	Laboratory investigation and back analysis for Bangkok soft clay
Onoue (1991)	$r_s=1.6r_m$	$k_h/k_s=3$	From test interpretation
Almeida et al. (1993)	$r_s=1.5\sim2r_m$	$k_h/k_s=3\sim6$	Based on experiences
Indraratna (1997)	$r_s=4\sim5r_w$	$k_h/k_v=1.15$	Laboratory investigation (For Sydney clay)
Chai & Miura (1999)	$r_s=2\sim3r_m$	$k_h/k_s=C_f(k_h/k_s)$	C_f the ratio between lab and field values
Hird & Moseley (2000)	$r_s=1.6r_m$	$k_h/k_s=3$	Recommend for design
Xiao (2002)	$r_s=4r_m$	$k_h/k_s=1.3$	Laboratory investigation (For Kaolin clay)

r_s: radius of smear zone, k_s: smear zone permeability, and k_v, k_h: horizontal and vertical permeability
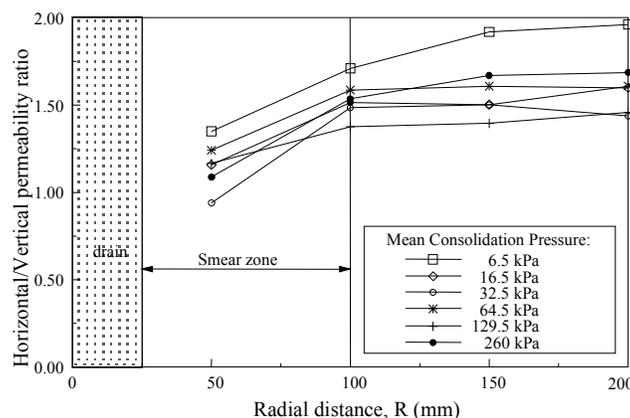
Figure 4 Ratio of k_h/k_v along the radial distance from the central drain (after Indraratna and Redana 1997)

In a similar study by Xiao (2002) using an fully instrumented 1 meter in diameter consolidation tank, the smear zone is also evaluated in terms of the normalized void ratio and excess pore pressure dissipation as shown in Fig. 5a and 5b respectively. Here the normalized void ratio is defined as the ratio of the void ratio in the smear zone to that in the intact zone. A comparison with other studies is also shown in Fig. 5a. The smear zone is at least 4 times the equivalent radius of the drain, r_m in terms of the normalized void ratio. However in terms of excess pore water pressure (Fig. 5b), the smear zone may even be larger. The excess pore pressure variation as shown in Fig. 5b is consistence with the analytical prediction shown in Fig. 2.

Soil Macro Fabric: For soil with a pronounced macro fabric, the k_h/k_v ratio can be very high, whereas the k_h/k_v ratio decreases towards unity within the disturbed (smear) zone. The smear zone can be larger for previously undisturbed soil due to the obvious destruction of the soil structure (Bo et al., 2003). The field monitoring data presented in Bo et al. (2003) show that vertical drain installation generates significant pore pressure in the soil near the drain. The excess pore pressure measured at a distance 1.27 m away from the drain, which is 19 times the equivalent drain diameter, can still be higher than the effective overburden stress. Therefore, the zone influenced by the vertical drain installation can be much larger, although the soil within this zone may not necessarily be smeared. It should also be mentioned that vertical drains are very efficient when the clay layers contain a lot of thin horizontal sand or silt lenses (micro layers), but if they are continuous in the horizontal direction, then there is no real necessity for installing PVDs as rapid drainage of pore water may still occur irrespective of the presence of artificial drainage elements.
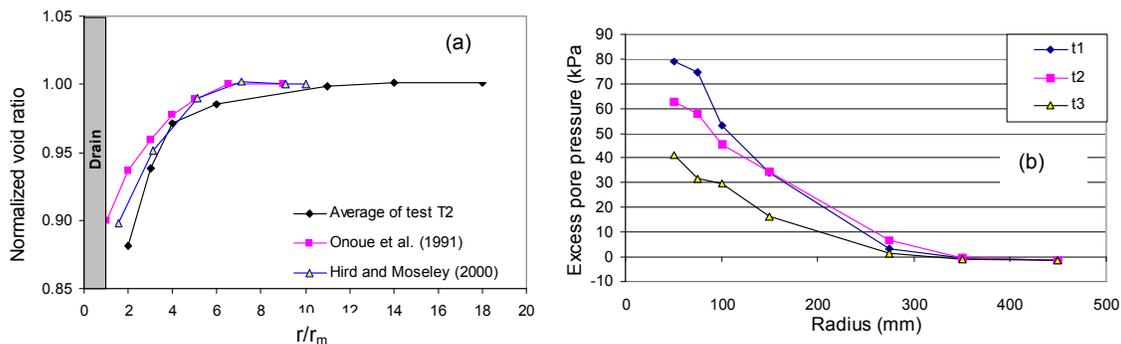
Figure 5 (a) Normalized void ratio distribution at different radial distance after drain installation, and (b) Excess pore pressure distribution at different radial distance from drain after drain installation using a mandrel (After Xiao, 2002)

Drain Unsaturation: Indraratna et al. (2001 and 2004) discussed the effects of unsaturation of soil adjacent to PVD due to mandrel withdrawal and the occurrence of a thin air interface or gap, and they discussed the apparent delay of pore pressure dissipation in large-scale laboratory testing through a series of FEM models. The

consolidation behaviour of soft clay in the large-scale consolidometer was simulated using the modified Cam-clay theory (Roscoe and Burland, 1968). The plane strain finite element mesh used 8-noded linear strain quadrilateral elements (CPE8RP) having 8 displacement nodes and 4 corner pore pressure nodes to minimise calculation time but still give the accurate prediction is shown in Figure 6a. It is sufficient to model half of the cell exploiting symmetry. The soil moisture characteristic curve (SMCC) including the effect of drain unsaturation was captured by a thin layer of interface elements. The following 3 models were analyzed (Indraratna et al, 2004):

Model 1 – Fully saturated soil with linear vacuum pressure distribution along the drain length.
Model 2 – The soil is primarily fully saturated. With the application of linearly varying vacuum pressure, a layer of unsaturated elements is simulated at the drain boundary, which is simulated by a thin unsaturated elastic layer ($E = 1000$ kPa, $\nu = 0.25$).
Model 3 – Prescribed conditions are similar to Model 2. The variation of vacuum pressure with time (vacuum removal and reloading) is considered in the analysis.


(a) **(b)**
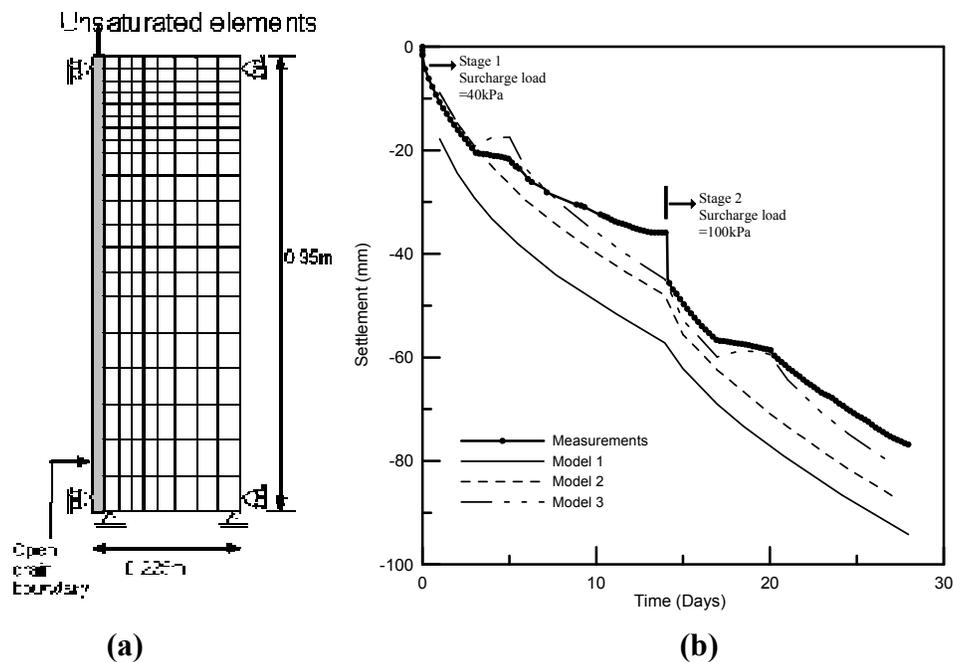
Figure 6 (a) FEM discritisation for plane strain analysis in large-scale consolidometer and (b) Predicted and measured settlement at the top of consolidometer (Indraratna et al. 2004)

The predicted and measured values of surface settlements are presented in Figure 6b. Models 2 and 3 agree well with the laboratory observations. Model 1 (fully saturated) yields the highest settlement, suggesting that the unsaturated soil-drain boundary causes the retardation of pore pressure dissipation. As expected, the potential drain unsaturation

is an important aspect that should be captured in numerical modelling, especially for relatively dry vertical drain installation, which is the usual practice.

Distribution of Vacuum Pressure: Indraratna et al. (2005a) observed from the large-scale consolidometer that vacuum pressure propagates down the drain length (Figure 7). For short drains this effect is immediate. The loss of vacuum at the bottom of the drain is approximately 15-20% of the applied vacuum pressure at the surface. It is noted that the rate of development of vacuum pressure along the PVDs depends on the drain length and the type of plastic core and filter (Bo et al., 2003). The measured pore pressure distribution will be used as a boundary condition in the proposed analytical solutions.


(a) (b)
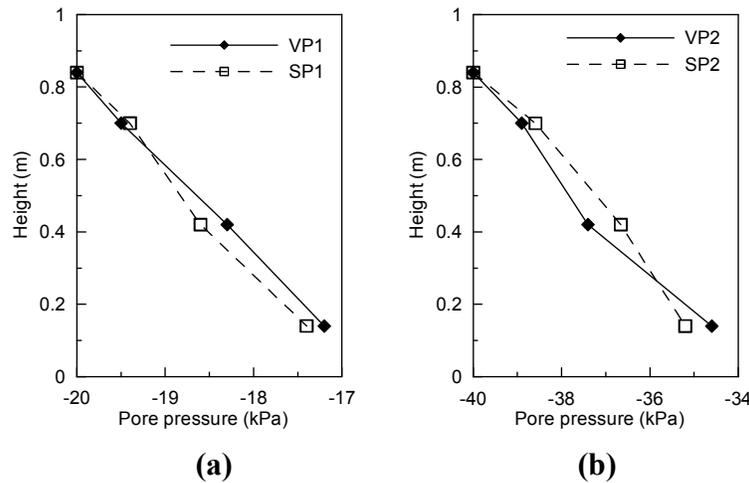
Figure 7 Distributions of measured negative pore water pressure along drain boundary (a) 20 kPa vacuum pressure and (b) 40 kPa vacuum pressure

RADIAL CONSOLIDATION THEORIES

Axisymmetric Condition: The original axisymmetric analysis for vertical drains (Barron, 1948) was further modified by Hansbo (1981) to include the effect of smear and well resistance. Mohamedelhassan and Shang (2002) developed a solution for the application of vacuum pressure with a surcharge load on the surface but without any vertical drains. Indraratna et al. (2005a, 2005b) and Walker and Indraratna (2006) proposed a comprehensive analytical solution for radial consolidation to include the effects of non-linear soil compressibility, soil permeability, parabolic permeability variation in the smear zone and linear vacuum pressure distribution. In this solution, the use of compressibility indices (C_c and C_r), which define the slopes of the e-logσ' relationship, and the variation of horizontal permeability coefficient (k_h) with void ratio (e) were captured.

The key postulations of the analysis are as summarized below (Indraratna et al. 2005b):

- Soil is homogenous and fully saturated, and the Darcy's law is adopted but only radial (horizontal) flow is permitted.
- According to laboratory measurements, 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 (Figure 8).
- The relationship between the average void ratio and the logarithm of average effective stress in the normally consolidated range (Figure 9a) can be expressed by the conventional expression: $\bar{e} = e_0 - C_c \log(\sigma'/\sigma'_i)$. If the current vertical effective stress (σ') is smaller than p'_c, then the recompression index (C_r) is used.
- For radial drainage, the horizontal permeability of soil decreases with the average void ratio (Figure 9b). The relevant relationship is given by Tavenas et al. (1983) by the expression: $\bar{e} = e_0 + C_k \log(k_h / k_{hi})$.
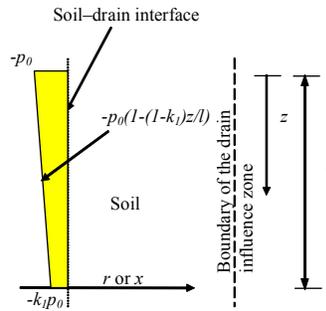
Figure 8 Distributions of vacuum pressure along the vertical drain (Indraratna et al. 2005a)

(a)   **(b)**
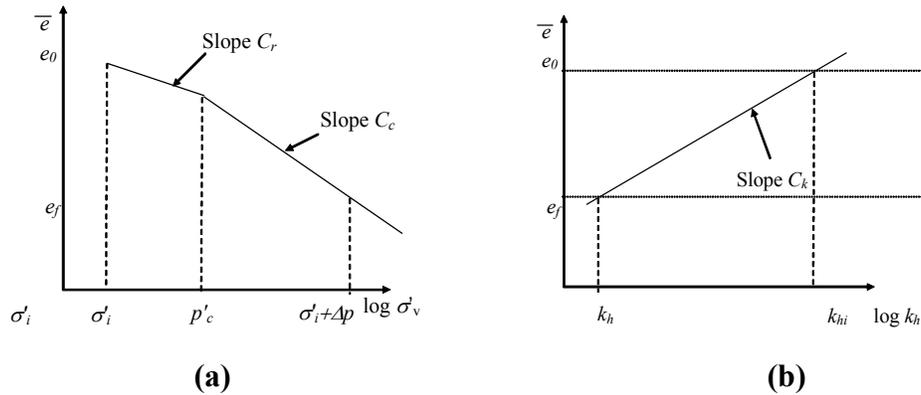
Figure 9 (a) Soil compression curve and (b) Semi-log permeability-void ratio (after Indraratna et al., 2005b)

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:

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

In the above expression,

$$T_h{}^* = P_{av}T_h \tag{9}$$

$$P_{av} = 0.5\left[1 + \left(1 + \Delta p/\sigma'_i + p_0(1+k_1)/2\sigma'_i\right)^{1-C_c/C_k}\right] \tag{10}$$

$$T_h = c_h t / d_e^2 \tag{11}$$

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 undistrubed zone (m/s), and k'_h = average horizontal permeability in the smear zone (m/s), Δp = preloading pressure, T_h is the dimensionless time factor for consolidation due to radial drainage, and μ = a group of parameters representing the geometry of the vertical drain system and smear effect.

A reduced horizontal permeability is assumed to be constant throughout the smear zone (Hansbo, 1981). The corresponding Hansbo's μ parameter is then given by:

$$\mu = \ln n/s + k_h/k'_h \ln s - 0.75 \tag{12}$$

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 of the soil in the smear zone continually increases towards the drain. Walker and Indraratna (2006) employed a parabolic decay of the horizontal permeability towards the drain representing a more realistic variation of soil permeability within the smear zone (Figure 10). The revised μ parameter is now given by:

$$\mu = \ln\left(\frac{n}{s}\right) - \frac{3}{4} + \frac{\kappa(s-1)^2}{(s^2 - 2\kappa s + \kappa)}\ln\left(\frac{s}{\sqrt{\kappa}}\right) - \frac{s(s-1)\sqrt{\kappa(\kappa-1)}}{2(s^2 - 2\kappa s + \kappa)}\ln\left(\frac{\sqrt{\kappa} + \sqrt{\kappa-1}}{\sqrt{\kappa} - \sqrt{\kappa-1}}\right) \tag{13}$$

In the above expression, $\kappa = k_h/k_0$. When the value of C_c/C_k approaches unity and p_0 becomes zero, the solution converges to the conventional solution proposed by Hansbo (1981):

$$R_u = \exp(-8T_h/\mu) \tag{14}$$

Conversion from Axisymmetric to Equivalent Plane Strain: In order to conduct multi-drain analyses, Hird et al. (1992) Indraratna and Redana (2000) Chai et al. (2001) and Indraratna et al. (2005a) proposed an appropriate conversion procedure described below. Hird et al. (1992) showed that the average equivalent plane strain permeability in the unit cell can be written as:

$$k_{h,pl}/k_{h,ax} = \frac{2B^2}{3r_e^2\left[\ln(r_e/r_s)+\left(k_{h,ax}/k_{s,ax}\right)\ln(r_s/r_w)-0.75\right]} \tag{15}$$

Chai et al. (2001) proposed an equivalent vertical hydraulic conductivity k_{ve} which can be used to 1D conventional FEM analysis without any drainage element. The equivalent vertical hydraulic conductivity k_{ve} can be expressed as:

$$k_{ve} = \left(1+\left(2.5l^2/\mu d_e^2\right)\left(k_{h,ax}/k_v\right)\right)\,k_v \tag{16}$$

Using the geometric transformation to explicitly model the smear zone (Figure 11), the ratio of the smear zone permeability to the undisturbed zone permeability is obtained by (Indraratna et al., 2005a):

$$\frac{k_{s,ps}}{k_{h,ps}} = \beta\left/\left(\frac{k_{h,ps}}{k_{h,ax}}\left[\ln\left(\frac{n}{s}\right)+\frac{k_{h,ax}}{k_{s,ax}}\ln(s)-\frac{3}{4}\right]-\alpha\right)\right. \tag{17}$$

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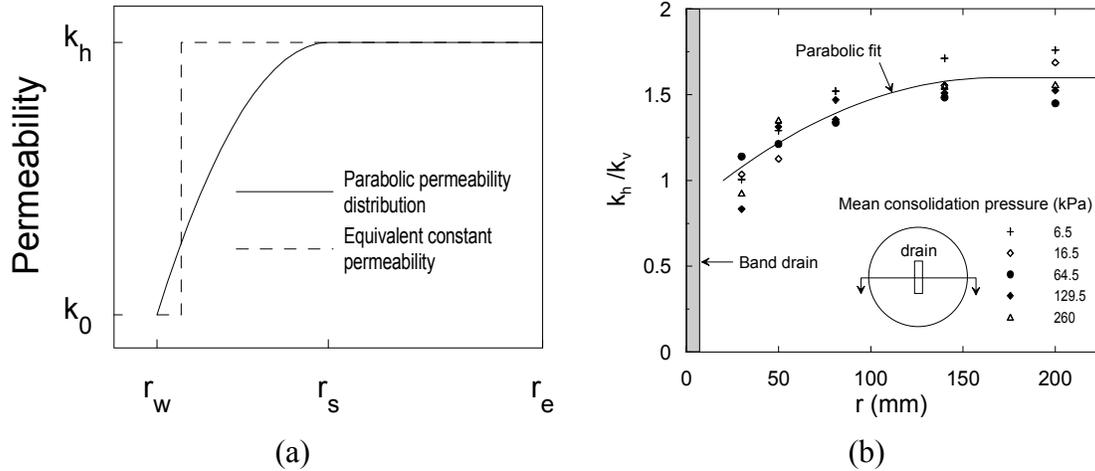

Figure 10 (a) Permeability distribution and (b) Ratio of horizontal to vertical permeability along radial distance from drain in large scale consolidometer (Walker and Indraratna, 2006)

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,

$$k_{h,ps}/k_{h,ax} = 0.67(n-1)^2/\,n^2\left[\ln(n)-0.75\right] \tag{18}$$

The equivalent vacuum pressure $p_{0,ps}$ can now be expressed by:

$$p_{0,ps} = p_{0,ax} \tag{19}$$

Flow contact area = $\pi d_w l$

Flow contact area = $2l$

Unit width
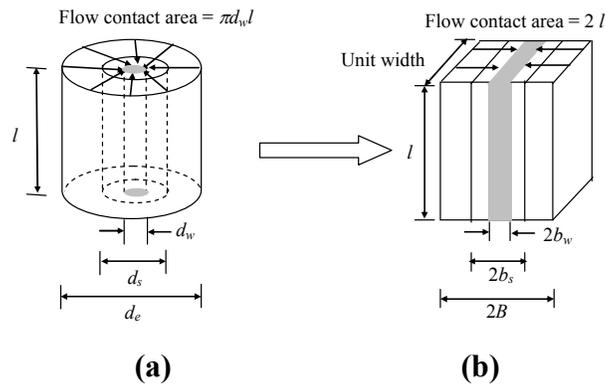
Figure 11 Unit cell analysis: (a) axisymmetric condition, (b) equivalent plane strain condition (after Indraratna et al., 2005a)

SALIENT ASPECTS OF NUMERICAL MODELING: APPLICATION TO CASE HISTORIES

### Test Embankments Stabilized with Vertical Drains on Soft Clay:

Indraratna & Redana (2000) and Indraratna et al. (2005c) investigated the effect of ground improvement by vertical drains at the Second Bangkok International Airport (SBIA), Thailand. To improve the soft soil conditions, PVDs with and without vacuum preloading were installed beneath 2 Embankments, namely, TS3 and TV2, respectively. PVDs were installed at 12 m deep in a triangular pattern with 1.0 m spacing. The constant values of k_h/k_s and d_s/d_w were assumed to be 2 and 6, respectively. For the plane strain FEM simulation, the equivalent permeability inside and outside the smear zone and vacuum pressure were determined using Equations (17)-(19). The discharge capacity (q_w) is assumed high enough and can be neglected (Indraratna and Redana, 2000). The finite element mesh contained elements having 8-node bi-quadratic displacement and bilinear pore pressure shape functions (Figure 12). Table 2 summarizes the soil properties at this site.

The predictions of ground settlement at the embankment centerline are shown in Figure 13. The analysis based on the proposed conversion procedure including the smear effect can predict settlement accurately. It can be seen that the time required to achieve the desired settlement can be reduced from 250 days to 120 days, if vacuum pressure is applied together with a surcharge load. This is because, the embankment construction together with vacuum pressure application does not require several construction stages as in the case of constructing an embankment without foundation stabilization (Indraratna et al., 1992). Figure 14 illustrates the excess pore water pressure variation with time. As expected, the vacuum loading creates negative excess pore pressures, thereby reducing the risk of any shear failure. It has been observed that in spite of PVDs, excess pore water pressures sometimes do not dissipate as expected. This is often attributed to filter clogging, extreme reduction of the lateral permeability of soil and

damage to piezometer tips. 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 the retarded rate of pore pressure dissipation (Indraratna, 2005).

Table 2 Selected soil parameters (adopted from Indraratna et al., 2005c)

| Depth (m)	λ	κ	ν	e_0	γ kN/m^3	k_v 10^{-9} m/s	k_h 10^{-9} m/s	k_s 10^{-9} m/s
0.0-1.0	0.3	0.03	0.30	1.8	16	15.1	30.1	89.8
1.0-8.5	0.7	0.08	0.30	2.8	15	6.4	12.7	38.0
8.5-10.5	0.5	0.05	0.25	2.4	15	3.0	6.0	18.0
10.5-13.0	0.3	0.03	0.25	1.8	16	1.3	2.6	7.6
13.0-15.0	1.2	0.10	0.25	1.2	18	0.3	0.6	1.8
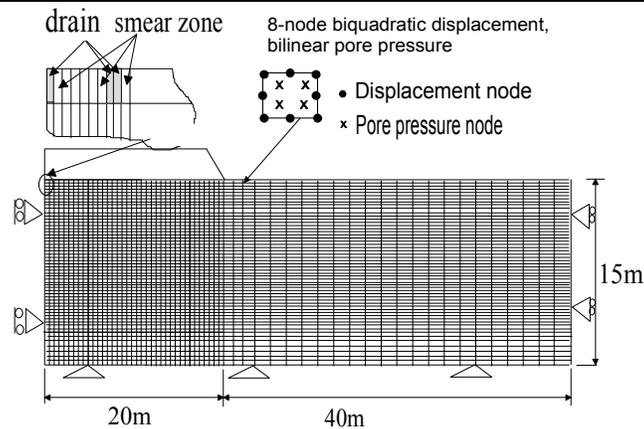
Figure 12 Finite element mesh of embankment for plane strain analysis at Second Bangkok International Airport (after Indraratna et al. 2005c)
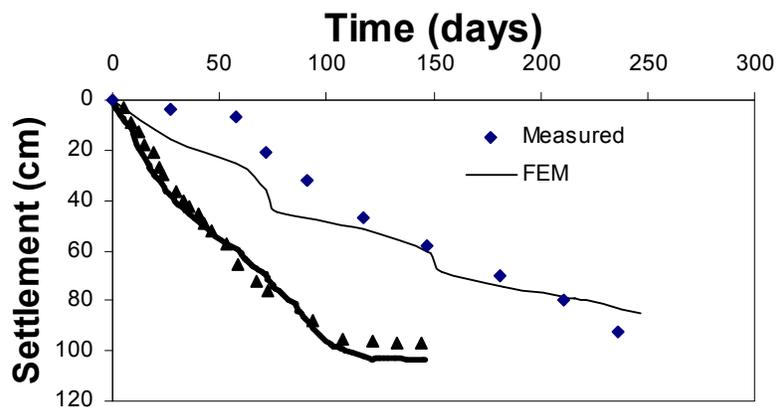

Figure 13 Settlement at the centre-line for embankments (after Indraratna and Redana, 2000 and Indraratna et al., 2005c)

The comparisons between predicted and measured lateral movement at the end of construction for embankments TS3 and TV2 are shown in Figure 15. The plane strain FEM model enables good prediction of the lateral yield beneath embankment TS3. For embankment TV2, the predicted lateral movement agrees with the field data at a depth below 4 m. However, the discrepancies between the predicted and measured results occur mainly at the weathered surface crust (about 0-2 m depth). Comparison between the cases of with and without vacuum pressure clearly indicates that vacuum preloading causes an inward lateral movement (i.e. towards the embankment centerline). In Figure 15, the stiffness of the compacted crust at TV2 is not properly modeled in the FEM analysis, hence the significant deviation from the field data close to the surface.
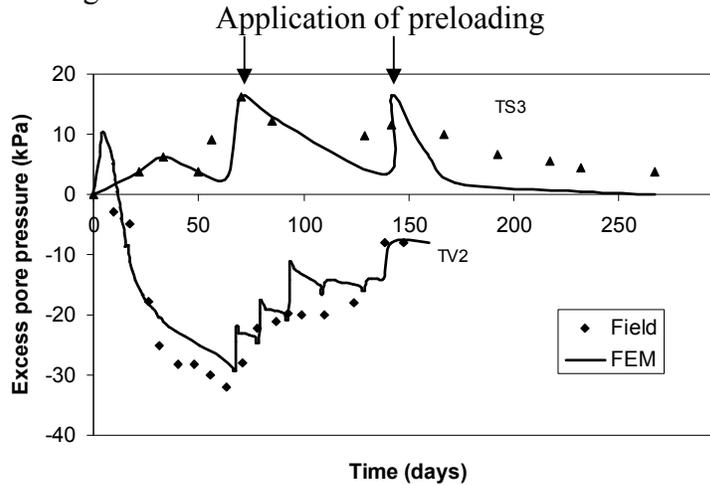
Figure 14 Excess pore pressure variation (after Indraratna and Redana, 2000 and Indraratna et al., 2005c)
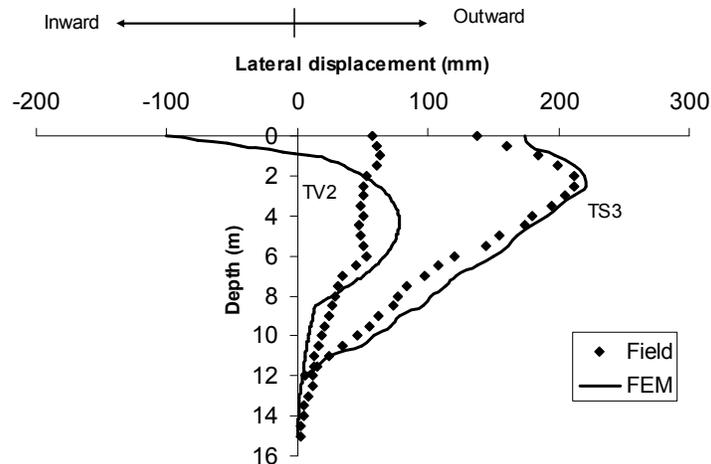
Figure 15 Calculated and measured lateral displacements at the end of construction (after Indraratna and Redana, 2000 and Indraratna et al., 2005c)

Performance of Short Vertical Drains Subjected to Cyclic Train Loads:

In the coastal areas, the rail tracks are often constructed on embankments overlying soft and compressible formation soils, such as estuarine and marine clays. The passage of heavy haul trains imparting cyclic loads at increased speeds causes excessive settlement and significant reduction in the load bearing capacity of the track. In the past, railway planners have avoided placing tracks on these deposits, selecting longer routes on better formations. However, the use of short PVDs can allow the construction of tracks over these soft clays. As described earlier, vertical drains accelerate consolidation, curtail lateral movements and thereby increase the track stability. In the initial stages, excessive initial settlement of track over deep estuarine deposits must be controlled by: (a) keeping the PVD length relatively short, and (b) optimising the drain spacing and the pattern of installation. In this way, while the settlements are controlled for routine maintenance, the reduction in lateral strains and the gain in shear strength of the soil immediately beneath the track improve its stability considerably. The following numerical analysis demonstrates the application of short vertical drains subjected to a strip train load.

A typical cross-section of the formation beneath the rail track is shown in Figure 16. The top most soil crust (1m in thickness) and the ballast layer (500mm thick) are modeled by Mohr-Coulomb theory. The two layers of soft normally consolidated clays beneath the track level are modeled by the MCC theory. Approximately 65% excess pore pressure dissipates within the first 4-5 months (Figure 17a), demonstrating the effectiveness of PVDs. Figure 17(b) shows a considerable reduction in lateral displacement of the PVD stabilized soil underlying the ballast layer (about 25%).
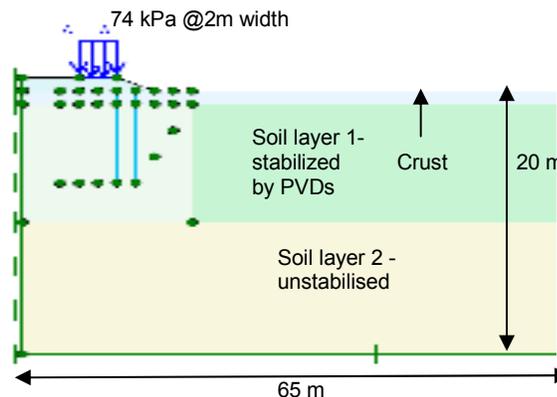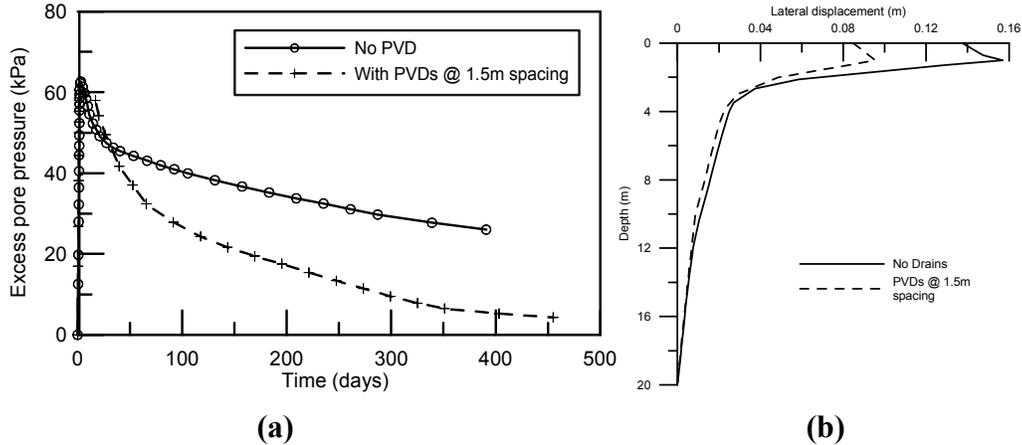

Figure 16 Vertical cross section of track and formation

Figure 17 (a) Excess pore pressure dissipation at 2 m depth at centre line of loading strip and (b) Lateral displacement profiles near the embankment toe

CONCLUSIONS

A system of geosynthetic prefabricated vertical drains combined with vacuum preloading is an effective method to accelerate consolidation by promoting radial flow. The key factors controlling the vertical drain efficiency including the occurrence of smear zone, influence of soil macro fabric, effects of drain unsaturation and the distribution of vacuum pressure were discussed with the objective of incorporating them in modern design. A more realistic analytical model for soft clay improved by vertical drains incorporating the compressibility indices (C_c and C_r) and vacuum preloading were introduced and their limitations were presented. The variation of horizontal permeability coefficient (k_h) with the applied stress level was also included. The parabolic decay of permeability in the smear zone associated with drain installation was considered to represent a more realistic field situation. The cavity expansion theory was introduced as a tool to predict the extent of the smear zone based on the excess pore pressure to effective overburden pressure ratio, which was found to be in agreement with the laboratory data based on permeability and water content approaches. It was found that the equivalent diameter of the smear zone was at least 3-4 times that of the mandrel.

The vacuum pressure application increases the rate of pore pressure dissipation due to the increased hydraulic gradient towards the drain. The numerical analysis confirmed that the occurrence of soil unsaturation at the drain-soil boundary due to both mandrel withdrawal and dry drain installation could retard the pore pressure dissipation. The use of appropriate suction-permeability relationships and revised FEM procedures are important in obtaining realistic predictions, if the soil adjacent to the PVDs becomes unsaturated in this manner. Numerical refinement also requires the use of improved procedures for accurate representation of the heavily over-consolidated surface crust that does not obey either the modified cam-clay or the conventional Mohr-Coulomb theory.

A system of vacuum-assisted consolidation via PVDs is a practical approach for accelerating radial flow and vertical consolidation when high surcharge fill cannot be placed, for example on soft dredged deposits. However, the vacuum effect may diminish significantly with depth due to various practical limitations such as improper membrane sealing and the nature of soil conditions such as the presence of fissures and macro-pores. Therefore, the assumption of decreasing suction values along the drain depth could be justified by field data and modeled using the finite element approach. Further study with 'instrumented PVD' in the field may provide more information of the vacuum pressure distribution with depth in the stablisation of soft clays. Such a system eliminates the need for a high surcharge load, as long as significant air leaks from the membrane sealing 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.

In the past, the ground improvement techniques using PVDs has not been popular for railway environments, but numerical analysis shows that they work effectively even under cyclic loads, where the formation soil consists of a high percentage of clayey soils. Short prefabricated vertical drains (PVDs) can be used under typical loads to dissipate cyclic excess pore pressure and to curtail lateral displacements, at the same time avoiding unacceptable initial settlement that can occur if long PVDs are used.

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