ADVANCES IN EMBANKMENT ENGINEERING THROUGH SOFT GROUND IMPROVEMENT

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ABSTRACT: In many countries, essential infrastructure is built along congested coastal regions that are composed of highly compressible soft and weak soils up to significant depths. For instance, alluvial and marine clay deposits have very low bearing capacity and excessive settlement characteristics, posing obvious design and maintenance challenges for constructing tall structures and large commercial buildings, as well as for developing port and transport infrastructure. Stabilising these very soft deposits is essential before commencing construction of infrastructure. A system of prefabricated vertical drains (PVD) combined with vacuum pressure and surcharge preloading has become an attractive ground improvement alternative in terms of both cost and effectiveness. This technique accelerates consolidation by promoting rapid radial flow which decreases the excess pore pressure while increasing the effective stress. In recent decades, natural prefabricated vertical drains (NPVDs) made from biodegradable fibre such as jute and coir have also become increasingly popular because of their distinct advantages such as comparative discharge capacity and environmental friendliness. However, NPVDs can sometimes decay rapidly in an adverse environment, e.g., in highly acidic soil, which retards the dissipation of excess pore pressure. In this keynote paper, an analytical solution considering the influence of biodegradation of NVPDs on soil consolidation is proposed in addition to the overview of theoretical and practical developments of soft ground improvement via PVD and NPVD together with vacuum preloading.

KEYWORDS: PVD, surcharge preloading, vacuum preloading, soft soil, consolidation, NPVD

1 INTRODUCTION

The ever-increasing population and inevitable construction boom in coastal and metropolitan areas have necessitated the exploitation of previously undeveloped low-lying areas for sustainable construction (Indraratna et al. 1992; Indraratna 2010). The low bearing capacity and high compressibility of these soil deposits affects the long term stability of tall buildings, major highways, rail tracks, and other civil infrastructure (Johnson 1970). It is imperative to stabilise these soils before commencing construction to prevent unacceptable differential settlement. However, attempts to improve deep bearing strata may not commensurate with the overall cost of infrastructure (Bo et al. 2003). In the past, various types of vertical drains such as sand drains, sand compaction piles, PVDs (geo-synthetic), stone columns, and gravel piles have commonly been used. Certain types of granular piles and deep stone columns may indeed significantly enhance the load bearing capacity and stability of the soil. However, because of the minimum risk of damage to utilities from lateral soil movement and generally affordable prices of flexible PVDs now produced in mass scale, they have often been preferred to more rigid compacted sand drains, gravel piles, stone columns etc. Their installation can significantly reduce the preloading period by decreasing the length of the drainage path, sometimes by a factor of 10 or more. More significantly, PVDs can be installed quicker with minimum environmental implications and quarrying requirements when compared to granular columns.

Preloading (surcharge embankment) is one of the most successful techniques for improving the shear strength of low-lying areas, because, it compresses the soil in advance to induce a greater part of the ultimate settlement prior to construction (Richart 1957; Indraratna and Redana 2000; Indraratna et al. 2005a). In order to control the development of excess pore pressure, a surcharge embankment is usually raised as a multi-stage exercise, with rest periods between the loading stages (Jamiolkowski et al. 1983). Since most compressible low-lying formations have very low permeability and are often thick, a lengthy time period is usually needed to achieve the desired primary degree of consolidation (>95%). In these instances, the height
of the surcharge can be excessive from both an economic perspective and stability consideration (Indraratna et al. 1994). One drawback of this surcharge technique is that the preload should be applied for a sufficiently long period, which may at times become impractical or uneconomical due to stringent construction schedules and deadlines. When PVDs combined with surcharge preloading is applied, a much shorter drainage path is created in the radial direction reducing the required preload period significantly. PVDs are often cost-effective and can be readily installed in moderate to highly compressible soils (up to 30m deep) that are normally consolidated or lightly over-consolidated. PVDs have very limited benefits if installed in stiff or heavily over-consolidated clays.

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

This keynote paper includes selected salient aspects of more than 20 years of active research conducted at the University of Wollongong in the area of soft soil stabilisation using PVDs and vacuum preloading, as well as NPVDs. This is undoubtedly a vital Australian contribution to the field of soft soil improvement worldwide, plus offering a significant component of higher education training through almost a dozen doctoral studies to date, promoting the advancement of current industry practices in infrastructure development in coastal and low lying areas.

2 PRINCIPLES OF VACUUM CONSOLIDATION VIA PVD

The vacuum preloading (VP) method for vertical drains was arguably first introduced in Sweden by Kjellman (1952). Since then, it has been used extensively to accelerate the consolidation of soft ground worldwide, for instance at the Philadelphia International Airport, USA; Tianjin port, China; North South Expressway, Malaysia; Reclamation world in Singapore and Hong Kong, China; Second Bangkok International Airport, Thailand; Ballina Bypass New South Wales and the Port of Brisbane, Queensland in Australia, among many other projects (Holtan 1965; Choa 1990; Jacob et al. 1994; Bergado et al. 2002; Chu et al. 2000; Yan and Chu 2003; Indraratna et al., 2011, 2012). When a high surcharge load is needed to achieve the desired undrained shear strength, and this cost becomes substantial due to an excessively high embankment and a long preloading period in order to achieve 95% or more consolidation, the optimum solution is to adapt to a combined vacuum and fill surcharge approach. In soft clays with low undrained strength, where a high surcharge embankment cannot be raised without affecting stability (i.e. large lateral movement) or having to work within a tight construction schedule, the application of vacuum pressure is quite often the most appropriate choice.

This PVD coupled system is designed to distribute the vacuum (suction) pressure to deep layers of the subsoil to increase the consolidation rate of reclaimed land and deep estuarine plains (e.g. Indraratna et al. 2005b; Chu et al. 2000).

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

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

2. The vacuum head can propagate to a greater depth of subsoil via the PVD system, and the suction can propagate beyond the PVD tips and the drain boundary.
3.1 Smear zone

Installing vertical drains with a steel mandrel remoulds the subsoil to a large extent, especially in its immediate vicinity. This disturbed annulus in the smear zone then shows a reduced lateral permeability and increased compressibility. In varved clays, the finer and more impervious layers are often dragged down and smeared over the more pervious layers which in turn tend to decrease the soil permeability near the PVD periphery. Barron (1948) suggested the concept of reduced permeability by arbitrarily lowering the apparent value of the coefficient of consolidation. Hansbo (1979) included a further explicit smear zone with a reduced permeability near the drain, surrounded by an outer undisturbed zone.

Based on constant but reduced permeability in the smear zone, Jamiołkowski et al. (1983) proposed that the diameter of the smear zone \( d_s \) and the cross section of the mandrel can be related by:

\[
d_s = (2.5 \text{ to } 3) \ d_m
\]

In the above, \( d_m \) is the diameter of the circle with an area equal to the cross section of the mandrel (i.e., equivalent mandrel diameter). Based on the results of Akagi (1979) and Hansbo (1987), the smear zone is often evaluated by the simple expression:

\[
d_s = 2 \ d_m
\]

Onoue et al. (1988) introduced a three zone hypothesis defined by (a) a highly disturbed (smear) zone close to the drain where the soil is remoulded during installation, (b) a plastic zone where the permeability is reduced moderately, and (c) an outer undisturbed zone where the soil is relatively unaffected by PVD installation. For practical purposes a two-zone approach is generally sufficient.

On the basis of their experimental work, Indraratna and Redana (1998) proposed that the estimated smear zone is at least 3-4 times larger than the cross section of the drain. This proposed relationship was verified using a specially designed large-scale consolidometer (the schematic section is shown in Fig. 2). Fig. 3 shows the variation of the ratio of horizontal to vertical permeability \( (k_h/k_v) \), and the water content along a radial distance from the central drain in the large-scale consolidation apparatus (Indraratna and Redana 1998; Sathananthan and Indraratna 2006; Walker and Indraratna 2006). The radius of the smear zone is about 2.5 times the equivalent radius of the mandrel. The lateral permeability (within the smear zone) is 61%-92% of the outer undisturbed zone, which is similar to Hansbo (1987) and Bergado et al. (1991) recommendations. Sathananthan et al. (2008) used the cavity expansion theory (CET), obeying the modified Cam-clay model, to analyse the extent of the smear zone caused by mandrel driven vertical drains. Their predictions were verified by large-scale laboratory tests where the extent of the smear zone was quantified based on (a) response of excess pore pressure generated while driving the steel mandrel, (b) change in lateral permeability, and (c) obvious reduction in the water content towards the drain.

Inspired by field observations, a lower boundary for drain spacing seems to exist herein, below which there is no discernible gain in the rate of consolidation.
Walker and Indraratna (2007) studied the effect of overlapping smear zones by considering a linear permeability distribution. As shown in Fig. 4, two smear zones will overlap when the spacing parameter \( n \) is less than the parameter \( S \) of the smear zone. In relation to undisturbed soil properties, a modified expression \( \mu \) representing the effect of overlapping smear zones can then be defined.

Fig. 3 Smear zone determination using: (a) permeability ratio; and (b) water content (Sathananthan and Indraratna 2006, with permission from ASCE).

Fig. 4 Schematic of overlapping smear zones (Walker and Indraratna 2007).

Fig. 5 indicates the time required to reach 90% consolidation \( t_{90} \) for various interacting smear zone configurations. It is shown that a range of drain spacing values exist, across which the time required to attain a given degree of consolidation does not change. These results are also in agreement with some test data reported by Saye (2001). This new analytical model for overlapping smear provides a rational basis for selecting the minimum drain spacing, below which the rate of consolidation may decrease. It appears that this radius of minimum influence is about 0.6 times the value of the radius of the linearized smear zone assumed for non-overlapping smear zones.

Madhav et al. (1993) proposed that the zone affected by mandrel driving can be divided into two distinct zones, namely the smear zone immediately surrounding the PVD and the outer transition zone surrounding the smear zone. Basu et al. (2010) studied the effects of both transition and smear zones on the rate of consolidation with a 2D finite element model. Instead of the equivalent circular drain and influential area, the actual band shape of PVD and rectangular shape of the influential zone were considered in that analysis.

The numerical analysis was validated by comparison with the experimental data of Indraratna and Redana (1998), in which the outer diameter of the smear and transition zones were approximately \( 2d_m \) and \( 7d_m \), respectively. The settlement history was obtained with and without considering the transition zone, and the results are plotted in Fig. 6. Also plotted in Fig. 6 are the test data of the settlement measured by Indraratna...
and Redana (1998). The results with expanded smear zone agreed reasonably well with the measured data. When the transition zone was ignored, a significant disagreement to the test measurement was observed in the prediction in the latter part of consolidation.

![Fig. 6 Comparison of settlement versus time data obtained from analysis capturing transition zone with the experimental data of Indraratna and Redana (1998) (adopted from Basu et al. 2010)](image)

### 3.2 Non-linear effects and soil structure characteristics

Although it is a convenient way to obtain a linear governing equation for vertical drain consolidation, the non-linear properties of the soils should be considered for greater accuracy. The non-linear variation of soil permeability and compressibility with the void ratio was incorporated into the equal-strain governing equation for PVD-based consolidation by Indraratna et al. (2005d). The non-linear relations between the void ratio \( e \) and the soil properties (permeability and compressibility) are:

\[
e = e_0 + C_1 \log \left( \frac{k}{k_0} \right)
\]

(3a)

\[
e = e_0 - C_2 \log \left( \frac{\sigma'}{\sigma'_0} \right)
\]

(3b)

where \( k_0 \) and \( \sigma'_0 \) are the soil permeability and vertical effective stress corresponding to \( e_0 \).

Besides the non-linear permeability and compressibility of the soil, the exponential non-Darcian flow was considered by Walker et al. (2012). The governing equation with the average excess pore pressure \( \bar{u} \) as the unknown was obtained as:

\[
\bar{u} = (\gamma_w r_n)^{-1/3} \beta \left( -\frac{r^2}{2C_s} \frac{\partial \bar{p}}{\partial r} \right)^{1/3}
\]

(4)

Where, \( \bar{u} \) is the average excess pore pressure, \( n \) is the non-Darcian flow exponent, and \( C_s \) is the coefficient of consolidation, and \( \beta \) is the non-Darcian radial consolidation parameter defined by Walker et al. (2012).

A laboratory test using the large-scale consolidometer (with a height of 950 mm and a diameter of 450 mm) at the University of Wollongong was carried out with reconstituted alluvial clay from the coastal town of Moruya, NSW in Australia. The values of \( C_s \) and \( C_r \) were tested to be 0.29 and 0.45, respectively. Results of the two tests with different pre-consolidation pressure and load were compared against the predictions of Indraratna et al. (2005d) and Walker et al. (2012), as shown in Fig. 7. The comparison indicates that the analytical solutions agree well with the test data, and the results using the formulation proposed by Walker et al. (2012) matches slightly better with the test data than that of Indraratna et al. (2005d).

![Fig. 7 Comparison between test data and predictions (Walker et al. 2012).](image)

It was also found that the ratio of \( C_s/C_r \) had a significant effect on the consolidation rate. For \( C_s/C_r<1 \), the actual consolidation was faster than the conventional linear solution; for \( C_s/C_r > 1 \), the actual consolidation was slower than the linear solution.

Although Eq. (3b) is more realistic than the linear relation adopted in most conventional approaches, it is only valid for reconstituted soils. The in situ behaviour of soft clays could be very different from laboratory studies, which is the reason why Rujikiatkamjorn and Indraratna (2014) developed an analytical solution for radial consolidation considering the soil-structure interaction characteristics. A conceptual model for soil disturbed by mandrel action was adopted (Rujikiatkamjorn et al. 2013), as shown in Fig. 8.

Using the field data at the Ballina Bypass project, this analytical solution was verified and compared against past research studies, but without considering disturbance to the soil structure, including Walker and Indraratna (2007). The Ballina Bypass was located on
highly compressible and saturated estuarine and alluvial clays, and thus PVDs with vacuum preloading were used to accelerate consolidation. The measured settlement data given in Fig. 9 have been compared with two analytical solutions proposed by Walker and Indraratna (2007) and Rujikiatkamjorn, et al. (2014). Walker and Indraratna (2007) captured a linear variation of permeability within the smear zone, but they ignored the effect of soil disturbance on the compressibility. In contrast, Rujikiatkamjorn and Indraratna (2014) considered the effects of soil disturbance on both permeability and compressibility in the smear zone, and these results indicated that the method capturing the effect of soil disturbance on both permeability and compressibility would provide a better match with the field data confirming the importance of soil structure characteristics in realistic field situations.

![Fig. 8 Conceptual compression behaviour of soil in different extent of disturbance (modified from Rujikiatkamjorn et al. 2013, 2014).](image)

![Fig. 9. Predicted and measured settlement in SP1 at Ballia bypass (adopted from Rujikiatkamjorn and Indraratna, 2014).](image)

### 3.3 Large strain effect

Soft estuarine deposits may be subjected to large strain when consolidated via PVD under extensive surcharge and vacuum preloading. Since traditional analytical solutions are mostly based on the assumptions of constant coefficient of consolidation, Darcian flow, and small strains, Indraratna et al. (2016a) proposed a new model to incorporate varying permeability and compressibility, non-Darcian flow as well as large strain effect for PVD-based consolidation. The non-Darcian flow may take place in low-porosity soils or when the hydraulic gradient is very low, and it can be considered by an exponential relationship between the seepage velocity and the hydraulic gradient (Eq. (4)).

Based on the large strain coordinate system, the Lagrangian coordinate $a$ and the convective coordinate $\xi$ have the relationship $\frac{\partial \xi}{\partial a} = \frac{1 + e}{1 + e_0}$ where $e$ is the void ratio and $e_0$ is the initial void ratio. A large-strain governing equation with radial flow has been established as:

$$\frac{1}{1+e} \frac{\partial e}{\partial t} \, da + v_r(r) \frac{2r}{r^2 - r_0^2} \frac{\partial \xi}{\partial a} \, da = 0$$

(5)

where $r$ is the radius, $r_c$ is the radius of the influential area, $t$ is the time, and $v_r(r)$ is the inward seepage velocity at radius $r$.

![Fig. 10. Comparison between large-strain solution and small-strain solution at Ballina Bypass. (Indraratna et al. 2016a, with permission from ASCE)](image)

This new approach has been applied to analyse the case study for the bypass route at Ballina, Australia, for the Pacific Highway linking Sydney and Brisbane that crossed over a floodplain of highly compressible estuarine soils. A comparison between the large-strain and small-strain results has been presented (Fig. 10), which indicates that the proposed large-strain solution gives an acceptable prediction of the settlement at each location.

### 4 ANALYTICAL SOLUTION FOR RADIAL CONSOLIDATION WITH NATURAL (FIBRE) DRAIN BIODEGRADATION

In normal soil, NPVDs are expected to discharge the excess pore pressure until the design target is achieved (Fig. 11), and they should degrade completely to an organic part of the soil. However, in an adverse environment, the discharge capacity of the drain can decrease too quickly due to biodegradation, consequently retarding the dissipation of excess pore pressure before the desired consolidation is achieved.
Therefore, Indraratna et al. (2016b) created a solution at the design stage that can predict the consolidation behaviour of soil with reference to the degradation of drain.

As shown by Indraratna et al. (2005a), the average excess pore pressure $u$ of the axisymmetric unit cell can be given by:

$$\int_0^t f_1 \int_{r_0}^{r_s} u_s (2\pi r) dr dz + \int_0^t f_2 \int_{r_2}^{r_s} u_i (2\pi r) dr dz$$

Integrating and re-arranging Eq. (6) result in the average excess pore pressure $u(t)$ as follows:

$$u = \frac{\gamma_u d_s^2}{2k_h} \left[ \frac{\mu_{n,s}}{4} + \frac{\mu_q}{d} \right] \frac{d \varepsilon}{dt}$$

(7)

In the above, expression $\gamma_u$ is the unit weight of the pore fluid and $\varepsilon$ is the vertical strain of the soil mass. $\mu_{n,s}$ is a parameter considering the effects of the smear and influence zones while $\mu_q$ is a function representing the reduction of discharge capacity over time: $\mu_q(t) = (2\pi k_h t^2) / 3 q_w(t)$ where $q_w(t)$ is the function of drain discharge capacity.

The general solution of Equation (7) can be given by:

$$u(t) = u_o \exp \left( -\int_0^t f(t) dt \right)$$

(8)

where $f(t)$ is written as:

$$f(t) = \frac{\mu_{n,s}}{q_w(t)} + \frac{\lambda}{q_w(t)}$$

(9)

where, $\lambda = (2\pi k_h t^2) / 3$; $\chi = d_s^2 / (8 c_0)$. Eq. (8) represents the excess pore pressure at time $t$ with an initial excess pore pressure $u_o$ and any given degradation function of drain discharge capacity $q_w(t)$.

While various biological studies (Means et al. 1985; Gamage and Asaeda 2005; Manzoni et al. 2012) have used exponential models to capture the organic decay process (e.g., jute, straw, coir), some other studies (Pronk et al. 1992; Harvey and Crundwell 1997) have reported the exponential growth of bacteria in ferric and ferrous environments such as in pyritic acid sulphate soil. Here, an exponential form is adopted to describe the reduction of discharge capacity of drain: $q_w(t) = q_{w0} e^{-\omega t}$, in which $\omega$ is the decay coefficient that represents the rate of degradation of the drain discharge capacity. Clearly $\omega$ should be within the range $[0, +\infty]$; and $q_{w0}$ is the initial discharge capacity of the drain. The exact solution estimating the consolidation degree of soil in axisymmetric condition with respect to exponential degradation of drain discharge capacity can then be written as:

$$u(t) = u_o \exp \left( -\frac{8T_h}{\mu_{n,s}} \left[ \frac{1}{2\mu_{n,s}^2} \ln \left( \frac{\mu_{n,s}}{\mu_{q0}} + e^{\omega t} \right) - \ln \left( \frac{\mu_{n,s}}{\mu_{q0}} + 1 \right) \right] \right)$$

(10)

It is interesting that when $\omega$ approaches zero (no degradation of drain), Eq. (10) approaches the conventional solution of Hansbo (1981).
magnitudes of $\omega$. For the most serious decay of drain ($\omega = 0.03$ day$^{-1}$) in which the discharge capacity of the drain decreased to $1.0 \times 10^{-3}$ m$^3$/s in 200 days, the dissipation of excess pore water pressure started to reduce considerably after about 80 days, and then reached a critical state after 200 days. With smaller values of decay coefficient, i.e., $\omega = 0.02$ and 0.01 day$^{-1}$, the consolidation curves started to decrease after 300 days and 500 days, respectively. The critical state of decay means a period where the drain permeability becomes very small to impede pore pressure dissipation substantially.

![Graph showing fitted curve and measurement data for discharge capacity and consolidation](image)

**Fig. 13** Compare the proposed analytical method with previous studies: a) Reduction curve of drain discharge capacity; b) Dissipation of excess pore pressure (Indraratna et al. 2016b).

In comparison with previous studies (Kim et al. 2011; Deng et al. 2013) which have investigated the effect of the decreasing discharge capacity on soil consolidation experimentally and analytically, the current solution offers an acceptable agreement (Fig.13). In the first 7 days, the analytical curve is approximately 5% higher than the experimental one, and then gradually approaches the experimental trend until it becomes slightly lower towards the end of the study. This difference could be attributed to the deviation in the NPVD discharge capacity between the fitted and recorded curves (Fig.13a). During the initial period, the exponential equation employing $\omega = 0.259$ day$^{-1}$ could not describe accurately the steep reduction in the discharge capacity, resulting in a gap between the analytical and experimental data (Fig.13b). When the exponential curve becomes closer to the measured one, the predicted pore pressure dissipation becomes more accurate. This indicates an important role of determining the degradation function of discharge capacity $q(t)$ in predicting the corresponding soil consolidation.

The current results shown in Fig.13b indicate a very similar response of excess pore pressure with time predicted by Deng et al. (2013). There is a slight difference between the two methods since Deng et al. (2013) used an approximate approach to solve the governing equation, whereas an exact solution is proposed in the current study. Moreover, the present solution introduces a general form $q(t)$ and incorporated it into the governing equation for excess pore pressure dissipation that makes the solution more flexible to accommodate various forms of discharge capacity reduction.

## 5 EXPERIMENTAL STUDY ON JUTE DRAINS

In order to measure the consolidation characteristics of natural jute fibre drains, a large consolidation cell with a radius of 650 mm and height of 450 mm was adopted. A 50 mm thick piston was placed on top and a hollow load cell was set up between the piston and the air cylinder to monitor the applied load. Seven pore pressure transducers were located in a staggered arrangement around the drain (i.e. 15, 30, 45, 60, 100, 150, 200 mm from the centreline) at a height of 100 mm from the bottom of the cell to measure the pore water pressure at different radii. A linear variable differential transformer (LVDT) was used to measure soil settlement. Sample height, moisture content and permeability were measured both prior to and after consolidation testing. Soil was extracted from a low-lying floodplain from a depth of 2m from Ballina Bypass whose properties have been described in detail by Indraratna et al. 2015.

The degree of consolidation with time is shown in Fig.14 and is compared with the theoretical calculations assuming linear and parabolic variations of permeability in the smear zone. The degree of consolidation ($U_t$) was derived using the theory of Carrillo (1942), i.e.

$$U_t = 1 - \sum_{m=1}^{\infty} \frac{8}{(2m + 1)^4 \pi^2} \times \exp \left( - \left( \frac{2m + 1}{2} \right)^2 \frac{k_e t}{m_s Y_w T} + \frac{8 \cdot k_h t}{\mu m_s Y_w d_s^2} \right)$$

(11)

The proposed theoretical calculations based on the variations in permeability are found to be in acceptable agreement with the measured data. The experimental data obtained from this proposed system agrees well with the parabolic trend observed for the later part of the consolidation process, and with the linear trend for the initial consolidation stage. The variation of
permeability in the smear zone thus plays an important role in determining the proper consolidation response.

Fig. 14 Time-Settlement curve of jute drain-soil system and its comparison with theoretical calculations based on linear and parabolic permeability profile in the Smear Zone (Choudhary et al. 2016).

Fig. 15 Variation of permeability and normalized water content from the centreline (Choudhary et al. 2016)

The pore water pressure variation along the radial distance was used to calculate a number of properties of the soil-drain system, on the basis of which a new method of evaluating the smear zone was formulated (Choudhary et al., 2016). The variation of normalized permeability \((k_r/k_o)\) and normalized moisture content \(\left(\frac{w_{\text{max}} - w}{w_{\text{max}}}\right)\) have been presented in Fig.15, which have then been compared with the data sourced from Sathananthan and Indraratna (2006). Also, \(r\) and \(r_m\) denote the distance from the centreline and the effective radius of the mandrel, respectively. It can be observed that the normalized permeability increases from a value of 1.6 at \(r = 15\) mm to a value of 2.2 at \(r = 100\) mm, and then remains constant thereafter. This demarcates the extent of the smear zone. A similar trend is also found in the plot for normalized water content indicating the smear zone radius to be nearly 100 mm, which is in agreement with Sathananthan and Indraratna (2006).

6 CONCLUSIONS

Since the past two decades in particular, vacuum assisted consolidation in conjunction with PVD has been an effective soil improvement method successfully used for large-scale reclamation works, tall buildings and transport infrastructure development in low-lying coastal formations. The research outcomes over many years confirm that the extent of surcharge fill can be decreased by up to 50-60% to achieve the same amount of degree of consolidation by PVD combined with vacuum preloading, apart from effectively controlling the lateral yield of soft formations. Nevertheless, the efficiency of this soil improvement method depends on various factors including the soil properties, groundwater conditions and vacuum system hardware.

Since early 1990s the design and operation of a novel large-scale consolidation process simulation test rig at University of Wollongong was able to quantify the appropriate smear zone characteristics associated with mandrel-driven PVD, the distribution of vacuum pressure along the PVD length, as well as the role of soil and drain properties. The research outcomes have shown that the equivalent radius of the smear zone could be even larger than 3 times the equivalent mandrel radius, and the soil permeability within the smear zone could be lower than that of the undisturbed zone by a factor of 2. The large cell consolidation tests with jute drains suggested that the extent of smear zone could be at almost 2-5 times the effective mandrel radius.

A novel method to evaluate the smear zone was proposed based on the hydraulic gradient changes derived from the measured excess pore pressure in large-scale consolidation chambers. By plotting the rate of change of excess pore pressure with the radial distance, the size of the smear zone could be estimated, and this approach was in excellent agreement with the past two techniques of quantifying the smear zone based on the reduced lateral permeability and normalised water content towards the drain. This method is most useful in the field where an adequate number of piezometers is available and requires no auguring of high quality soil specimens to be tested in the laboratory, hence quicker and cost-effective.

The drain spacing is of paramount importance in design. Overly closely spaced PVD will lead to overlapping of adjoining smear zones, a phenomenon which has been studied and quantified through a rigorous analytical model that offers a rational basis for
evaluating the minimum drain spacing. In fact, overlapping of smear will cause retarded rate of consolidation making the use of PVDs redundant and a design adversity. For most soft soils tested by the first Author for more than two decades in many countries, it is rare that the drain spacing would be made less than a meter, and for most soft estuarine clays 1.2-1.5m spacing is the norm.

Analytical modelling of PVD combined with vacuum preloading to simulate the consolidation of a unit cell of soil surrounding a single vertical drain has been essential for incorporating in numerical analysis, including the conversion of 2D plane strain equivalence. In order to predict realistic field behaviour, the computational scope has been further enhanced by the development of models quantifying the effects of non-linear permeability and compressibility properties, the occurrence of non-Darcian flow for soils with very low permeability, as well as the large strain effects experienced by viscoplastic soft soils. Although conventional liner Darcy flow is valid in most cases, non-Darcian flow in soils that have already undergone significant compression plays an important role at relatively low levels of permeability and hydraulic gradients. Moreover, the possible unsaturation at the soil-drain boundary would retard the pore pressure dissipation during initial consolidation in the same way as drain clogging, and will also cause non-conventional flow-deformation conditions.

The large strain analysis has demonstrated that the conventional small-strain solution may over-predict the degree of consolidation, especially for highly compressible clays subjected to large external loads. The inclusion of large strain constitutive models significantly increases the accuracy of predicting the correct deformation behaviour including lateral displacements of very soft plastic clays.

Soil structure disturbance (remoulding or destructuration) caused by mandrel movement can adversely affect both permeability and compressibility within the smear zone, impeding the drain effectiveness. The analytical models incorporating the soil disturbance can provide more realistic solutions for the deformation of soft clay foundations stabilised by PVD. Naturally, this effect is more pronounced for structured clays with carbonate and other mineralogical bonds. A comparison between analyses with and without soil structure characteristics indicated that the inclusion of soil disturbance would provide a better match with the field data for PVD and VP embankments.

In an environmentally conscious modern day era, the use of biodegradable materials has become increasingly more canny and prudent, whereby extensive experimental studies in tandem with complex mathematical solutions to model the time-dependent biodegradation process of natural fibre (jute and coir) drains and subsequent coupling with the soil consolidation theory can be regarded as a recent breakthrough in soft soil improvement led by the researchers at University of Wollongong. These solutions incorporate the gradually diminishing drain discharge capacity and the blatant implications on pore pressure dissipation, hence the consequential settlement. The recent tests of jute PVD biodegradation using large scale consolidation cells (650mm diameter and 450mm in height) using Ballina clay (acidic) demonstrate somewhat retarded excess pore pressure dissipation compared to conventional geosynthetic PVDs after a significant period in the ground (> 6-8 months or so), especially when the clay is acidic (e.g. pyritic estuarine, acid sulphate soils). The analytical results agreed well with those obtained from experimental approaches, with less than 5% deviation.

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