1-1-2014

Overview of preloading methods for soil improvement

Jian Chu
Iowa State University

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

Shuwang Yan
Tianjin University

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

Overview of preloading methods for soil improvement

Abstract

A review of the recent developments in soft soil improvement through consolidation or preloading is presented in this paper. The topics covered range from fundamental analysis to methods of implementation. Various methods and processes related to vertical drains, vacuum preloading or combined vacuum and fill surcharge, and dynamic consolidation with enhanced drainage or vacuum are compared and discussed. Factors affecting the design and analyses for the methods discussed are also elaborated.

Keywords

soil, methods, improvement, preloading, overview

Disciplines

Engineering | Science and Technology Studies

Publication Details

Chu, J., Indraratna, B., Yan, S. & Rujikiatkamjorn, C. (2014). Overview of preloading methods for soil improvement. Proceedings of the Institution of Civil Engineers: Ground Improvement, 167 (3), 173-185.

Ground Improvement
Volume 167 Issue GI3

Overview of preloading methods for soil improvement
Chu, Indraratna, Yan and Rujikiatkamjorn

ice | proceedings

Proceedings of the Institution of Civil Engineers
Ground Improvement 167 August 2014 Issue GI3
Pages 173–185 http://dx.doi.org/10.1680/grim.13.00022
Paper 1300022
Received 17/04/2013 Accepted 25/09/2013
Published online 18/01/2014
Keywords: geotechnical engineering/geotextiles, membranes & geogrids/reviews

ice
Institution of Civil Engineers
publishing

Overview of preloading methods for soil improvement

Jian Chu PhD
Professor and James M. Hoover Chair in Geotechnical Engineering, Department of Civil, Construction and Environmental Engineering, Iowa State University, Ames, Iowa, USA

Buddhima Indraratna MSc, DIC, PhD, FTSE, FIEAust, FASCE, FGS
Professor of Civil Engineering, School of Mining and Environmental Engineering, Research Director of Centre for Geomechanics and Railway Engineering, University of Wollongong, Wollongong City, NSW, Australia

Shuwang Yan PhD
Professor and Director of Geotechnical Research Institute, Tianjin University, People's Republic of China

Cholachat Rujikiatkamjorn BEng (Hons), MEng (AIT), PhD
Associate Professor, Centre for Geomechanics and Railway Engineering, University of Wollongong, Wollongong City, NSW, Australia

A review of the recent developments in soft soil improvement through consolidation or preloading is presented in this paper. The topics covered range from fundamental analysis to methods of implementation. Various methods and processes related to vertical drains, vacuum preloading or combined vacuum and fill surcharge, and dynamic consolidation with enhanced drainage or vacuum are compared and discussed. Factors affecting the design and analyses for the methods discussed are also elaborated.

Notation

B	half width of unit cell
b_s	half width of smear zone
b_w	half width of drains
C_f	ratio between laboratory and field values
c_h	coefficient of consolidation of soil in the horizontal direction
c_v	coefficient of consolidation of soil in the vertical direction
d_e	diameter of soil cylinder dewatered by a drain; related to drain spacing
d_m	equivalent diameter of mandrel
d_s	diameter of the smear zone
d_w	equivalent diameter of idealised circular drain
e	void ratio of soil
e_0	initial void ratio of soil
$F(n)$	function of n
F_s	factor of safety
H_{clay}	clay thickness
k_h	horizontal permeability of soil
k_{hp}	equivalent coefficient of soil permeability
k'_{hp}	equivalent coefficient of permeability in the smeared zone
k_s	permeability of smeared zone
k_w	permeability of drain
k_{wp}	permeability of drain under plane-strain
l	length of the drain
n	d_e/d_w
q_w	discharge capacity of drain
q_z	equivalent plane strain discharge capacity
R	axisymmetric radius
r	radial distance
r_s	axisymmetric radius
r_w	radius of equivalent drain
S	settlement due to surcharge preloading only
S_t	settlement of a given time t_1 including the settlement component due to vacuum pressure
s	drain spacing
T_h	time factor in the horizontal direction
t	time
U_h	average degree of consolidation in the horizontal direction
z	depth
α	coefficient
β	coefficient
θ	coefficient
κ	k_h/k_s

1. Introduction

It is well known that the compressibility and shear strength of soil can be greatly improved if the water content in the soil can be significantly reduced. One common method for improving soft soil is to reduce the water content of the soil through consolidation. For consolidation to occur there must be an increase in effective stress. This can be achieved by increasing the total stress or reducing the pore-water pressure. The former is the so-called fill surcharge preloading method. The latter can be achieved through vacuum preloading. When a surcharge pressure is applied, the increase in the effective stress is dependent on the dissipation of excess pore-water pressures generated as a response to the application of this surcharge. To accelerate the dissipation of pore-water pressure, prefabricated vertical drains (PVDs) are normally used. PVDs are also used together with the vacuum preloading method to distribute vacuum pressure and facilitate the dissipation of pore water. Therefore, PVD techniques become part of the core technologies in the fill surcharge or vacuum preloading methods. PVDs have been used successfully in many soil improvement and land reclamation projects in the world (Arulrajah *et al.*, 2009; Bergado *et al.*, 1991, 1996, 2002; Bo *et al.*, 2003, 2005, 2007; Choa *et al.*, 2001; Chu *et al.*, 2000, 2006, 2009a, 2009b, 2009c; Hansbo, 1981, 2005; Holtz *et al.*, 1991; Indraratna, 2009; Indraratna *et al.*, 2005a, 2011, 2012; Kitazume, 2007; Li and Rowe, 2001; Pothiraksanon *et al.*, 2010; Seah, 2006; Varaksin and Yee, 2007; Yan *et al.*, 2009). Therefore, the theories, design and construction methods for PVDs become the core technical issues in the preloading or consolidation methods for soft soil improvement.

Depending on how a preload is applied, the preloading methods can be subdivided into preloading using fill, preloading using vacuum pressure and combined fill, and vacuum preloading methods. In addition to preloading, PVDs have also been used for some other relatively new methods such as dynamic consolidation for clays. In both cases, the main purpose of using PVDs is to reduce the drainage path so that the time taken for the consolidation of soft soil or the dissipation of excess pore-water pressure can be substantially reduced. In this paper, some recent developments on soft soil consolidation and soft soil improvement are reviewed. According to the soil classification system adopted by TC211 (Chu *et al.*, 2009c), soil improvement through consolidation or preloading belongs to the category of 'ground improvement without admixtures in cohesive soils'. This category is further divided into the following seven subcategories (Chu *et al.*, 2009c)

(*a*) replacement/displacement (including load reduction using lightweight materials)
(*b*) preloading using fill (including the use of vertical drains)
(*c*) preloading using vacuum (including combined fill and vacuum)
(*d*) dynamic consolidation with enhanced drainage (including the use of vacuum)
(*e*) electro-osmosis or electro-kinetic consolidation
(*f*) thermal stabilisation using heating or freezing
(*g*) hydro-blasting compaction.

In this paper, only the following selected topics are discussed due to page limit: (*a*) vertical drains; (*b*) preloading using vacuum including combined fill and vacuum; and (*c*) dynamic consolidation with enhanced drainage including the use of vacuum. A more comprehensive review on soil improvement methods involved consolidation and preloading is given in a state-of-the-art report by Chu *et al.* (2012).

2. Prefabricated vertical drains

2.1 Vertical drain theories

A number of analytical solutions have been developed in the past for consolidation of ground improved with vertical drains (Barron, 1948; Carillo, 1942; Hansbo, 1981; Onoue *et al.*, 1991; Walker *et al.*, 2012; and Yoshikuni and Nakanodo, 1974; Zeng and Xie, 1989). Most of the theories adopted a 'unit cell' model as shown in Figure 1. In this model, the band-shaped drain is idealised into a circular drain with an equivalent diameter of $d_w = 2(a + b)/\pi$ as proposed by Hansbo (1979). A few other methods were proposed to calculate the equivalent diameter of PVD as reviewed by Indraratna *et al.* (2005a). However, the differences in different methods are small and Hansbo's method is commonly adopted.

Radial consolidation theories such as those proposed by Carillo (1942) formed the basic equations for the analysis of radial consolidation of soil. When PVDs are used, other factors need to be taken into consideration. Two of the major factors are the smear effect and well resistance. When PVDs are installed in the

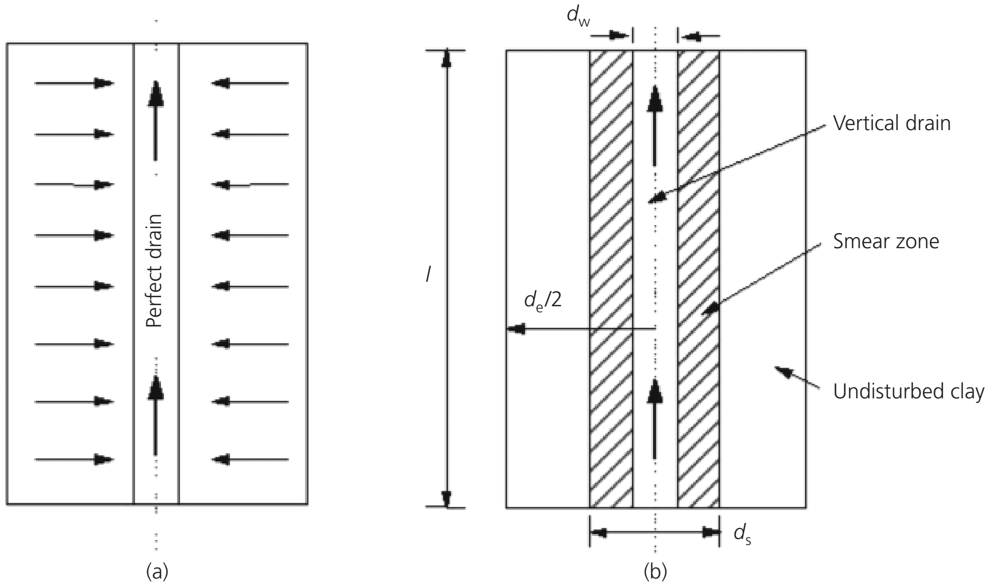


Figure 1. Unit cell model of (a) a perfect drain and (b) a drain with smear zone

soil, the penetration of the steel mandrel disturbs the soil surrounding the PVD. This smear effect causes a reduction in the permeability and coefficient of consolidation of the soil within the smear zone. When the discharge capacity of PVDs is limited, head loss will occur when water flows along the drain and delays the consolidation process. This unfavourable effect has been called the well resistance. Taking the smear effect and well resistance into account, the well-known Barron (1948) and Hansbo (1981) equations have been proposed and used for PVD design. As an example to illustrate the parameters that affect the consolidation of soil using PVDs, Hansbo's equation (Hansbo, 1981) is written as follows

1. $$U_h = 1 - \exp\left(\frac{-8T_h}{F(n)}\right)$$

2. $$F(n) \approx \ln(n) - 0{\cdot}75 + \ln(s)\left(\frac{k_h}{k_s} - 1\right) + \pi z(2l - z)\frac{k_h}{q_w}$$

3. $$T_h = \frac{c_h t}{d_e^2}, \quad n = \frac{d_e}{d_w}, \quad s = \frac{d_s}{d_w}$$

where c_h is the coefficient of consolidation of soil in the horizontal direction; t is time; d_e is the diameter of soil cylinder dewatered by a drain, which is related to the drain spacing: $d_e = 1{\cdot}128s$ for a square grid and $d_e = 1{\cdot}05s$ for a triangle grid; $F(n)$ is a function of d_e, d_w, the diameter of the smear zone, d_s, the horizontal permeability of the soil, k_h, the permeability of the smeared zone, k_s, the discharge capacity of the drain, q_w, the length of the drain, l, and the depth z. The last term in Equation 2 represents the well resistance. It can be seen from Equations 1 and 2 that the factors affecting the consolidation of soil around PVDs are the soil parameters, c_h and k_h, the properties of the smear zone, d_s and k_s, and the properties of the PVD, q_w. The effects of those factors will be discussed separately in the next section.

Equations 1 and 2 were derived based on Darcy flow, that is, by assuming Darcy's law is valid. Flow in soil can be non-Darcian, as shown by Hansbo (1960) and Holtz and Broms (1972) in both laboratory and in the field. Discharge capacity tests on vertical drains using a drain tester (Chu *et al.*, 2004) have also shown that water flow in PVDs is non-Darcian in general (Bo *et al.*, 2003; Lee and Kang, 1996). Hence consolidation theories for non-Darcian flow soil should be used in general, although it may not always be necessary in practice. Consolidation theories based on non-Darcian flow have been proposed by Hansbo (2001) and Walker *et al.* (2012). Using several case studies, Hansbo (2005) demonstrated that the consolidation process based on non-Darcian flow yields better agreement with the pore pressure observations than the theory based on the assumed effect of creep. For the test area IV of the well-known Skå-Edeby test field case in Sweden (Hansbo, 1960), the consolidation based on Darcian flow over-predicted the excess pore-water pressure distribution in the ground in 14 years, whereas the prediction based on non-Darcian flow matches the field monitoring data better as shown by Hansbo (2005). However, the predictions of the pore-water pressure distribution in the ground in 1·5 years by the two theories are nearly the same (Hansbo, 2005). This is probably due to the fact that the hydraulic gradient at the beginning of consolidation is relatively higher. More studies or field verification are required to establish whether non-Darcian flow consolidation theories have to be applied in general for more accurate pore pressure prediction.

Most of the practical consolidation problems are three-dimensional (3D). Therefore, the 'unit cell' theory needs to be modified to be used for numerical modelling of practical problems. For simplicity, two-dimensional (2D) plane strain solutions are commonly adopted. To employ a realistic 2D plane strain analysis for vertical drains, the appropriate equivalence between the plane strain and axisymmetric analysis needs to be established in terms of consolidation settlement. Figure 2 shows the conversion of an axisymmetric vertical drain into an equivalent drain wall. This can be achieved in several ways (Basu *et al.*, 2010; Hird *et al.*, 1992; Indraratna and Redana, 1997; Rujikiatkamjorn *et al.*, 2008): (*a*) geometric matching – the drain spacing is matched while maintaining the same permeability coefficient; (*b*) permeability matching –coefficient of permeability is matched while keeping the same drain spacing; and (*c*) combination of (*a*) and (*b*), with the plane strain permeability calculated for a convenient drain spacing. Examples of these approaches by Bergado and Long (1994), Chai *et al.* (1995, 2013), Hird *et al.* (1992), and Indraratna and Redana (1997) were reviewed and further advanced by Indraratna *et al.* (2005a).

The method by Indraratna and Redana (1997) is based on the conversion of the vertical drain system shown in Figure 2 into an equivalent parallel drain wall using an equivalent coefficient of soil permeability, k_{hp}. They assumed that the half width of unit cell B; the half width of drains b_w; and the half width of smear zone b_s are the same as their axisymmetric radii R, r_w and r_s, respectively. The equivalent permeability of the model is then determined by

4. $$k_{hp} = \frac{k_h\left[\alpha + (\beta)(k_{hp}/k'_{hp}) + (\theta)(2lz - z^2)\right]}{\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k'_h}\right)\ln(s) - 0{\cdot}75 + \pi(2lz - z^2)\frac{k_h}{q_w}\right]}$$

5a. $$\alpha = \frac{2}{3}\frac{(n-s)^3}{(n-1)n^2}$$

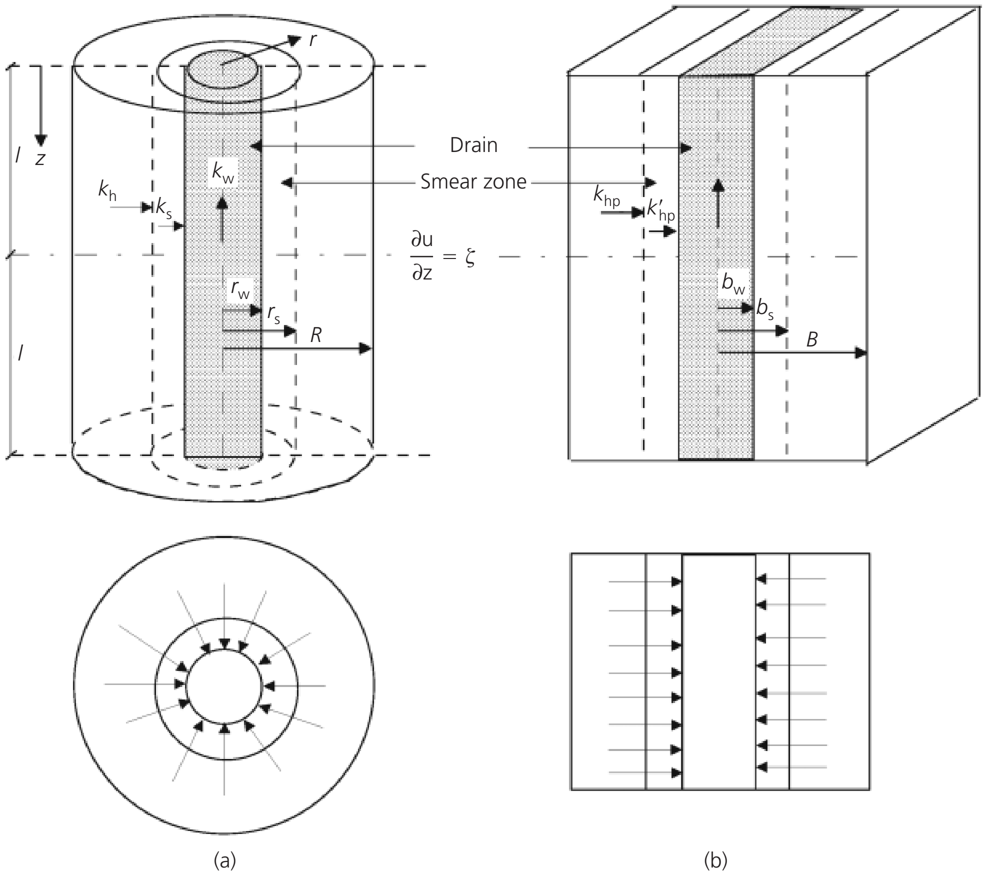


Figure 2. Conversion of (a) an axisymmetric unit cell into (b) plane strain condition (adapted from Hird *et al.* (1992) and Indraratna and Redana (1997))

5b. $$\beta = \frac{2}{3}\,\frac{(s-1)}{(n-1)n^2}\,\left[3n(n-s-1)+(s^2+s+1)\right]$$

5c. $$\theta = \frac{2k_{hp}}{Bq_z}\left(1-\frac{1}{n}\right)$$

where, $q_z = 2q_w/\pi B$ is the equivalent plane strain discharge capacity.

It should be pointed out that the equivalent coefficient of permeability k_{hp} appears in both sides of Equation 4. The solution thus has to be obtained by iteration with an initially assumed k_{hp}/k'_{hp} ratio, where k'_{hp} is the equivalent coefficient of permeability in the smeared zone.

2.2 Factors affecting the consolidation of soil around PVDs

As discussed above, the main factors affecting the consolidation of soil around PVDs are the soil parameters, c_h and k_h, the properties of the smear zone, d_s and k_s, and the properties of PVD, q_w. The influences of these factors are discussed as follows.

2.2.1 Soil parameters c_h and k_h

Once the consolidation theories are in place, the next design step appears to be as straightforward as putting in the soil parameters to obtain the answer. However, the determination of soil parameters is still one of the most challenging tasks facing geotechnical engineers. On one hand, it is necessary to obtain a value for each soil parameter. On the other hand, few soil parameters are constant. For example, the coefficient of consolidation, c_v or c_h, is assumed to be a constant in either Terzaghi's or Barron's consolidation theory. However, in practice, neither c_v nor c_h for soft soil is a constant. Its value is affected by many factors, such as the overconsolidation ratio, the stress state, the fabric of the soil, and even the method of determination (Chu *et al.*, 2002). As such, the selection of c_v or c_h has to be based on its in situ stress conditions and the anticipated stress changes. Therefore, it is also necessary to establish relationships between the coefficient of permeability and void ratio, and relationships between the coefficient of consolidation and the stress state. A proper site investigation should be planned not only to determine the soil parameters but also to understand how the soil parameters vary with stress and loading conditions. The coefficient of permeability is another key parameter required for vertical drain design. However, it happens that the coefficient of permeability of soil is one of the most difficult soil parameters to determine. This is partially because the coefficient of permeability of the soil has the widest range of variation among all the soil parameters. Its value can vary from 10^{-11} m/s for soft clay to 10^{-3} m/s for sand and gravel, a change of 10^8 times. Although the permeability of the soil that has to be treated with vertical drains is normally low, the error involved in the permeability estimation can still range from 10 to 100 times. This is not unusual as the permeability of the same soil can change by a factor of 10 to 100 during the process of consolidation. An error of one order of magnitude in permeability can result in an error of the same order of magnitude in the time taken to achieve a specific degree of consolidation based on Terzaghi's consolidation theory, as shown by Bo *et al.* (2003). Therefore, it makes sense economically to conduct some proper site investigation work and determine the soil parameters as accurately as possible. Generally the consolidation parameters of soil can be determined using laboratory tests, in situ tests, back-calculation from field measurements, or a combination of these. The types of laboratory and in situ tests that are suitable to the determination of consolidation properties are discussed in detail in Chu and Raju (2012).

Consolidation theories to consider the variation of c_h and k_h with stress or void ratio of soil have also been proposed (e.g. Walker *et al.*, 2012). In this case, the relationships between c_h and void ratio or k_h with void ratio need to be established.

2.2.2 Smear zone

Consolidation of soil around PVDs is affected by the smear effect. However, it is not an easy task to determine the diameter of smear zone, d_s, and the permeability in the smear zone, k_s, because the smear effect is affected by many factors, including the type of mandrel used, the method used to penetrate the mandrel and the type of soil. The smear effect is due not only to the disturbance to the soil, but also the compressibility of the soil. To reduce the smear effect, the cross-section of the mandrel should be as small as possible. On the other hand, a mandrel is a slender tube and it has to have a certain stiffness to be structurally stable. The influence of different types of mandrel and anchor shoes has been evaluated by Bo *et al.* (2003) and Basu and Prezzi (2007). In terms of method used to penetrate the mandrel into soil, static pushing is better than vibration. Soil type is probably one of the most important factors. The smear effect in sensitive or cemented soil can be much greater than that in recently deposited soil (for example, clay fill used for land reclamation). A number of studies on smear effect have been carried out in the past (Abuel-Naga *et al.*, 2012; Abuel-Naga and Bouazza, 2009; Almeida and Ferreira, 1993; Basu *et al.*, 2010; Basu and Prezzi, 2007; Bergado *et al.*, 1991; Bo *et al.*, 2003; Chai and Miura, 1999; Hansbo, 1979, 1981; Hird and Moseley, 2000; Indraratna and Redana, 1998; Madhav *et al.*, 1993; Onoue *et al.*, 1991; Sathananthan and Indraratna, 2006; Xiao, 2002). A summary of different studies is given in Table 1. The values given in Table 1 are proposed for the smear model shown in Figure 1(b).

It should be pointed out that when soil is disturbed in the smear zone, there is a remoulding zone and transition zone. The remoulding zone is caused by the displacement of the mandrel as the soil within this zone is completely remoulded. The transition zone is the zone outside the mandrel which is disturbed by the penetration of the mandrel. The degree of disturbance should be transitional or change with the distance away from the mandrel – the further away from the drain, the smaller the disturbance. This

Source	Extent	Permeability	Remarks
Barron (1948)	$d_s = 1{\cdot}6d_m$	$k_h/k_s = 3$	Assumed
Hansbo (1979)	$d_s = 1{\cdot}5{\sim}3d_m$	Open	Based on available literature at that time
Hansbo (1981)	$d_s = 1{\cdot}5d_m$	$k_h/k_s = 3$	Assumed in case study
Bergado *et al.* (1991)	$d_s = 2d_m$	$k_h/k_v = 1$	Laboratory investigation and back analysis for soft Bangkok clay
Onoue *et al.* (1991)	$d_s = 1{\cdot}6d_m$	$k_h/k_s = 3$	From test interpretation
Almeida and Ferreira (1993)	$d_s = 1{\cdot}5{\sim}2d_m$	$k_h/k_s = 3{\sim}6$	Based on experience
Indraratna and Redana (1998)	$d_s = 4{\sim}5d_m$	$k_h/k_v = 1{\cdot}15$	Laboratory investigation (for Sydney clay)
Chai and Miura (1999)	$d_s = 2{\sim}3d_m$	$k_h/k_s = C_f(k_h/k_s)$	C_f the ratio between laboratory and field values
Hird and Moseley (2000)	$d_s = 1{\cdot}6d_m$	$k_h/k_s = 3$	Recommended for design
Xiao (2002)	$d_s = 4{\sim}6d_m$	$k_h/k_s = 1{\cdot}3$	Laboratory investigation (for kaolin clay)
Bo *et al.* (2003)	$d_s = 11d_m$	$k_h/k_s = 2{\sim}10$	Based on field tests in marine clay

Note: d_s: diameter of smear zone; d_m: equivalent diameter of mandrel; k_h: permeability of intact soil; k_s: permeability of the smeared soil.

Table 1. Proposed smear zone parameters (modified from Indraratna *et al.*, 2005a)

is what has been observed from the model tests. The results of four different studies are as shown in Figure 3, where the normalised void ratio is plotted against the distance from the centre of the drain, r, normalised by the radius of the equivalent drain, r_w. The normalised void ratio is defined as the void ratio of the soil measured at different positions in the soil over the initial void ratio of the soil, e/e_0. The smear effect causes the void ratio to reduce. However, this effect becomes less significant when the distance to the drain becomes larger, as can be seen from Figure 3. The change in the permeability of soil in this transition zone follows the same trend (Abuel-Naga *et al.*, 2012; Sathananthan and Indraratna, 2006). For this reason, it has been proposed by Abuel-Naga *et al.* (2012) to model the smear effect using both a smear and transition zone. Laboratory tests conducted using a large-scale consolidometer by Indraratna and Redana (1998), Onoue *et al.* (1991) and Xiao (2002) have suggested that the disturbance in the 'smear zone' continuously intensifies towards the drain, and a linear or piecewise assumption is not realistic. 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 parabolic curve that satisfies the above conditions, shown schematically in Figure 4, is given by

6. $$k'_h(r) = k_s(\kappa - 1)(A - B + Cr/r_w)(A + B - Cr/r_w)$$

where $\kappa = k_h/k_s$, $A = \sqrt{\kappa/(\kappa - 1)}$, $B = s/(s - 1)$ and $C = 1/(s - 1)$.

It is not convenient to use a variable permeability for vertical drain design unless the finite-element method is adopted. For this reason, the equivalent unit cell method proposed by Abuel-Naga *et al.* (2012) is useful. Nevertheless, the smear zone plus transitional zone idea does provide some limiting values for the selection of the diameter of the smear zone. It will have to be

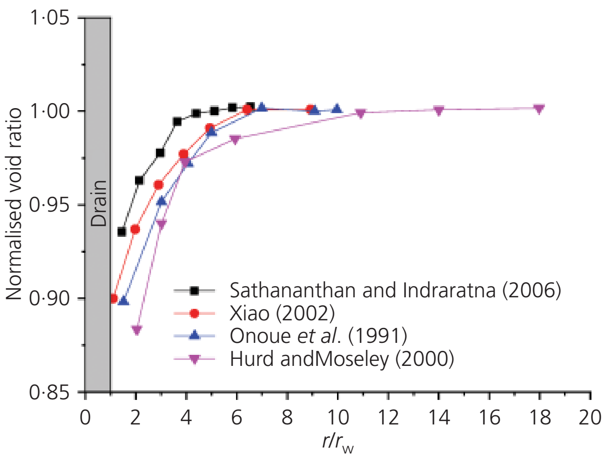


Figure 3. Change in void ratio at different radial distance as a result of smear effect (modified from Xiao (2002))

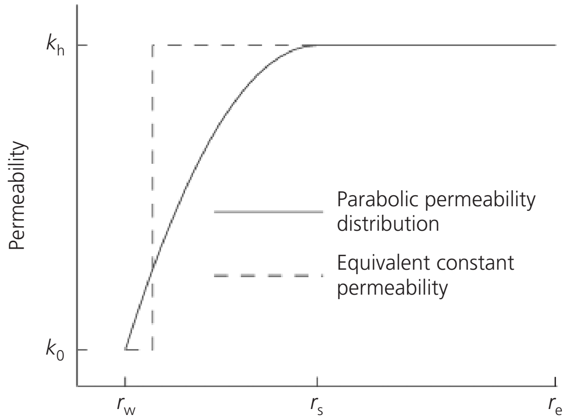


Figure 4. Parabolic permeability distribution (Walker and Indraratna, 2006)

greater than the equivalent mandrel diameter. The dimension of a typical rectangular mandrel is 120 mm long by 60 mm wide (Bo *et al.*, 2003). Using the same method for PVD, the equivalent diameter of the mandrel, d_m, can be calculated as 115 mm. If the equivalent diameter of the PVD, d_w, is 66 mm (by taking the width and thickness of PVD as 100 mm and 4 mm respectively), then the diameter of the smear zone, d_s, will be at least $1{\cdot}7d_w$. If the transition zone is taken into consideration, the diameter of the smear zone will be at least $(2{\sim}3)d_w$. It should be pointed out that most of the studies mentioned in Table 1 were based on laboratory tests using reconstituted soil. However, the use of laboratory reconstituted or remoulded soil samples tends to underestimate the smear effect, as the effect of destruction of soil structure or fabric cannot be reflected. Therefore, the smear effect should also be assessed by field measurements. Unfortunately, field studies of the influence of PVD installation on the soil properties are rare. One such study at a reclaimed site in Singapore was reported by Bo *et al.* (2003). In this study, it was reported that a large amount of pore pressure was measured at a location 1·27 m away from the drain in the horizontal direction and pore-water pressure was still measured at a distance as far as 2·85 m away from the drain. The drains used were 100 mm wide and 4 mm thick. Thus the equivalent diameter d_w was 66 mm. However, a point 1·27 m away from the drain was $32d_w$. It is debateable whether this should be an indication of the boundary for transition zone or smear zone. If it is the transition zone, then the distance of 2·85 m should be used and this was $43d_w$! Bo *et al.* (2003) also reported that the reduction in permeability ranges from 1·8 to 11·0 times. Hence for intact soil in situ, the real reduction in permeability can be much greater than the values suggested by the methods shown in Table 1. More field data are required to verify whether the data obtained in this study are typical. Nevertheless, the study of Bo *et al.* (2003) does illustrate the point that the diameter of the smear zone in the field can be much greater than that determined by laboratory model tests using remoulded soil samples.

Because of the smear effect, it is not always beneficial to use a close drain spacing to reduce the consolidation time, unless the soil to be consolidated has been deposited recently. For one project reported by Chu *et al.* (2002), the back-calculated c_h based on field monitoring data was even smaller than the c_v determined by laboratory oedometer tests. This could be because of the smear effect, as discussed by Chu *et al.* (2002).

2.2.3 Well resistance

Well resistance refers to the finite permeability of the vertical drain with respect to the soil. Head loss occurs when water flows along the drain and delays radial consolidation. A number of studies have been made in the past on the modelling of well resistance as summarised by Indraratna *et al.* (2005a). Theoretically, the well effect is modelled by the last term of Equation 2: $\pi z(2l - z)k_h/q_w$. Therefore, the well resistance is controlled by the length of the drain, the discharge capacity of the drain q_w and the permeability of the soil k_h. However, if q_w is sufficiently large, then this term $\pi z(2l - z)k_h/q_w$ can be small enough to be ignored. The good news is there are PVD products that can provide enough q_w to make the effect of well resistance insignificant (Chu *et al.*, 2004). The required value of q_w for well resistance to be ignored will be discussed in the next section. However, it should be pointed out that the discharge capacity of PVDs can deteriorate with time due to deformation, clogging, biochemical reaction and so on. Therefore, if PVDs are to be used for a long time (e.g. more than 6 months) or subjected to very large deformation (e.g. when used for very soft soil), the reduction in the discharge capacity of PVDs needs to be evaluated.

3. Vacuum preloading

3.1 Vacuum consolidation systems

When the ground is very soft or when the fill surcharge has to be applied in stages to maintain the stability of the fill embankment, the vacuum preloading method becomes a good alternative. Vacuum preloading is also used when there is no fill or the use of fill is costly, when there is no space on site to place the fill and when slurry or soft soil is used as fill for reclamation. The idea of vacuum preloading was proposed by Kjellman (1952). Since then, the vacuum preloading method has evolved into a mature and efficient technique for the treatment of soft clay. This method has been successfully used for many soil improvement or land reclamation projects all over the world (Bergado *et al.*, 1996, 2002; Chen and Bao, 1983; Chu *et al.*, 2000; Chu and Yan, 2005a, 2005b; Cognon, 1991; Holtz, 1975; Indraratna *et al.*, 2005a, 2010, 2012). Varaksin and Yee (2007) and Yan and Chu (2003, 2005) also argued that vacuum preloading is more sustainable as its carbon footprint is much smaller compared with other soil improvement methods.

The schematic arrangement of the vacuum preloading system adopted in China is shown in Figure 5. PVDs are normally used to distribute vacuum load and discharge pore water. Soil improvement work using the vacuum preloading method is normally carried out as follows. A 0·3 m sand blanket is first placed on the ground surface. PVDs are then installed on a square grid at a spacing of 1·0 m in the soft clay layer. Corrugated flexible pipes (50 to 100 mm in diameter) are laid horizontally in the sand blanket to link the PVDs to the main vacuum pressure line. The pipes are perforated and wrapped with a non-woven geotextile to act as a filter layer. Three layers of thin polyvinyl chloride (PVC) membranes are laid to seal each section. Vacuum pressure is then applied using jet pumps. The size of each section is usually controlled in the range of 5000–10 000 m^2. Field instrumentation is an important part of the vacuum preloading technique, as the effectiveness of vacuum preloading can only be evaluated using field monitoring data. Normally piezometers, settlement gauges and inclinometers are used to measure the pore-water pressure changes, settlement at ground surface and/or different depths in the soil and lateral displacement. More details are presented in Chu *et al.* (2000) and Yan and Chu (2003).

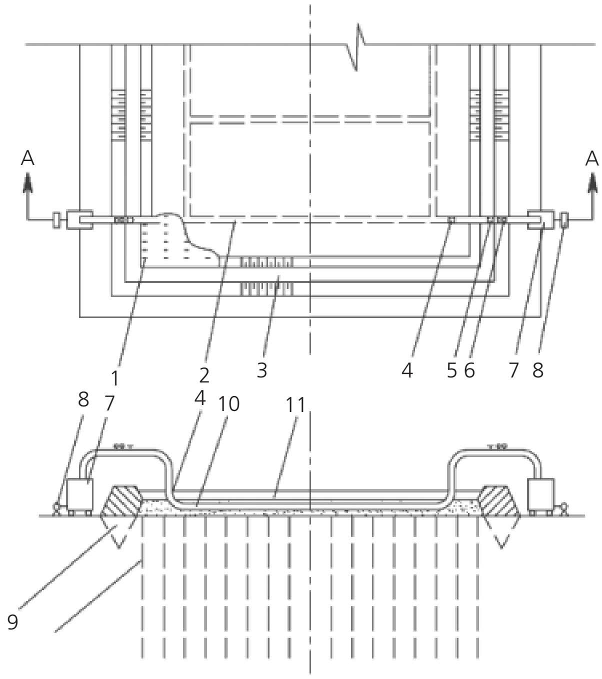


Figure 5. Vacuum preloading system used in China (after Chu *et al.*, 2000): 1, drains; 2, filter piping; 3, revetment; 4, water outlet; 5, valve; 6, vacuum gauge; 7, jet pump; 8, centrifugal gauge; 9, trench; 10, horizontal piping; 11, sealing membrane

In Europe, the Menard vacuum consolidation system has been developed in France by Cognon (1991). The details of this system can be found in Varaksin and Yee (2007). The uniqueness of this system is the dewatering below the membrane, which permanently keeps a gas phase between the membrane and the lowered water level. Therefore, the Menard vacuum consolidation system adopts combined dewatering and vacuum preloading methods to maintain an unsaturated pervious layer below the membrane.

The vacuum preloading method may not work well when the subsoil is inter-bedded with sand lenses or permeable layers that extend beyond the boundary of the area to be improved, such as the improvement of soft soil below sand fill for reclaimed land. In this case, a cut-off wall is required to be installed around the boundary of the entire area to be treated. One example is given by Tang and Shang (2000), in which a 120 cm wide and 4·5 m deep clay slurry wall was used as a cut-off wall in order to improve the soft clay below a silty sand layer. However, installation of cut-off walls is expensive when the total area to be treated is large. One solution to this problem is to connect the vacuum channel directly to each individual drain. This so-called BeauDrain system has been developed in the Netherlands (Kolff *et al.*, 2004). This method has evolved in the past few years and the later version is shown in Figure 6. In this method, the top of each vertical drain is connected to a plastic pipe as shown in Figures 6(a) and 6(b). In this way, the channel from the top of the PVD to the vacuum line is sealed using the plastic pipe and thus goes through a sand layer without causing leak in vacuum. A special connector, as shown in Figure 6(b), is used for this purpose. The plastic pipes are connected directly with the vacuum line at the ground surface as shown in Figure 6(c). Thus, a sand blanket and membranes, as used in the conventional vacuum methods shown in Figure 5, are not required. This method has been used for the construction of the new Bangkok Suvarnabhum international airport (Seah, 2006) and other projects (Chai *et al.*, 2008). One shortcoming of this method is that it is difficult to achieve a high vacuum pressure in soil. This could be caused by two factors. The first is the difficulty in ensuring every drain is completely sealed. The second is the head loss in the sealed plastic pipe (see Figure 6(a)). This method also requires a more detailed soil profile as the length of each PVD has to be predetermined to match the depth of the clay layer at each PVD location. The production rate is also thus lower.

3.2 Comparison of membrane and membraneless vacuum preloading systems

Numerical and analytical modelling of vacuum preloading considering membrane and membraneless systems have been described previously by Indraratna *et al.* (2005b), and more elaborately by Geng *et al.* (2012) very recently, where both vertical and horizontal drainage were captured to reflect in situ conditions. The placing of the surface sand blanket and the installation of a completely air-tight membrane is imperative for the membrane type vacuum system in order to create and sustain a desired uniform vacuum pressure on the soil surface, and thereby ensure the speedy propagation of this vacuum head down the PVDs to consolidate the clay layer. The permeability of the sand layer plays an important role in this process as it governs the effectiveness of vacuum pressure propagation from the upper soil boundary to the PVDs to consolidate the clay layer. The roles of permeability of the sand blanket in a membrane system and the adverse effect of vacuum loss with depth in a membraneless system have been analysed by Geng *et al.* (2012). Figure 7 illustrates the effect of the sand blanket permeability in a membrane system. As expected, when permeability decreases, the time for consolidation increases. For relatively short PVDs (less than 10 m), Figure 7(a) shows that the permeability of the sand blanket should not be less than 0·01 times the permeability of the PVD and at least 10^4 times the permeability of the clay to maintain an acceptable consolidation time for a degree of consolidation (DOC) of 90%. With longer drains (Figure 7(b)),

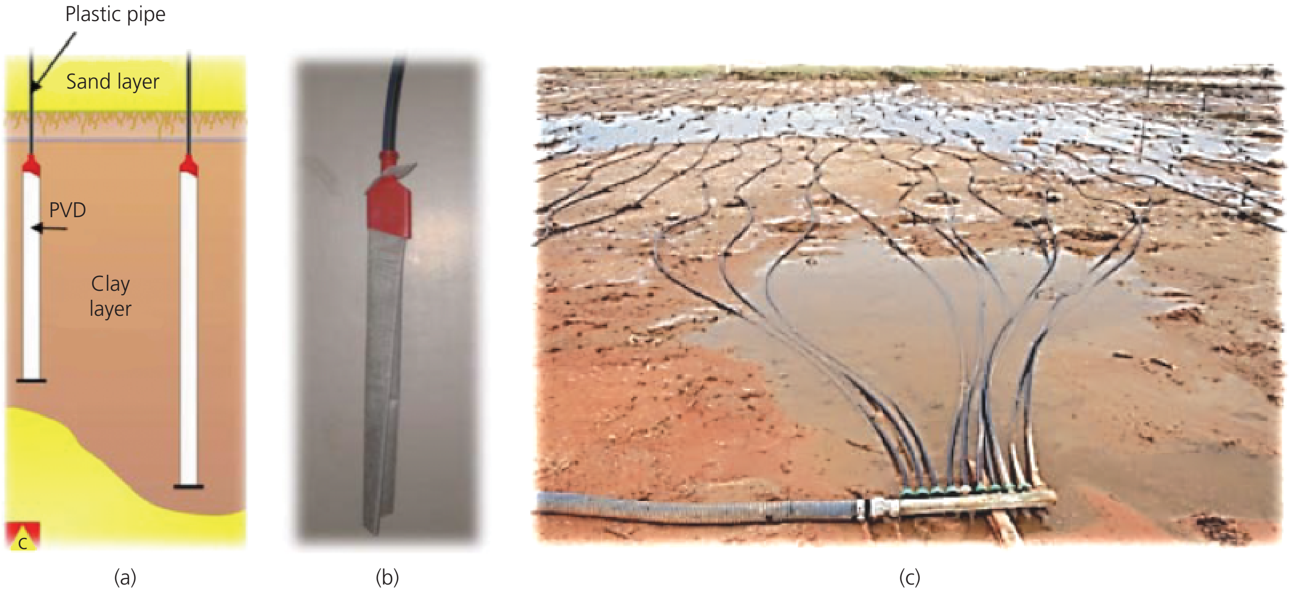
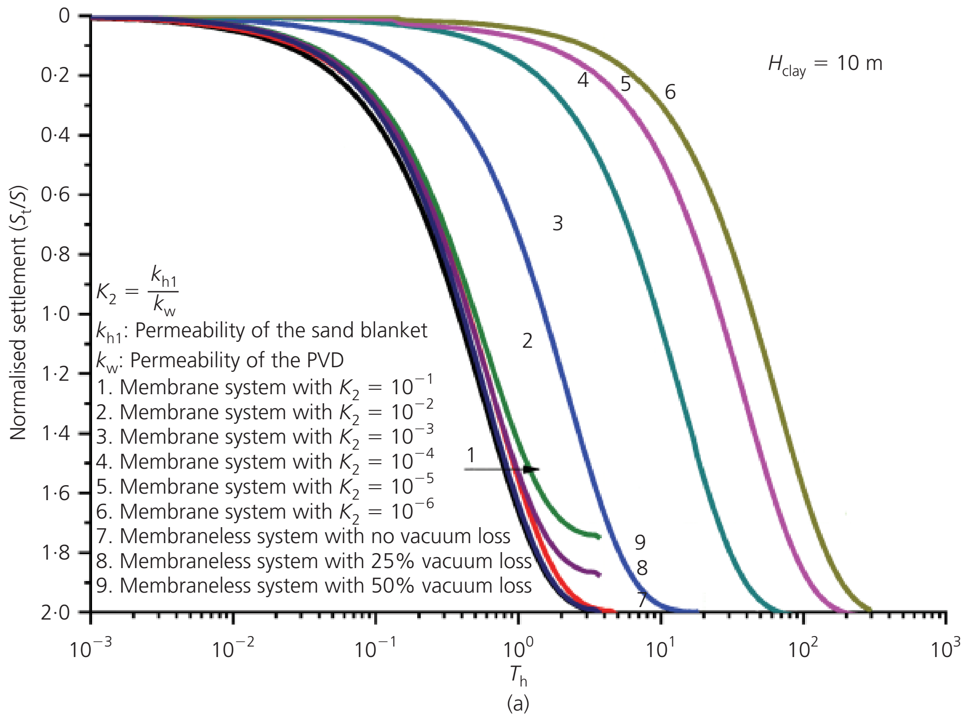


Figure 6. BeauDrain vacuum preloading system (a) concept (Courtesy of Cofra, Holland); (b) direct connection of PVD with plastic pipe for vacuum application; and (c) connection of plastic pipes to a vacuum pump



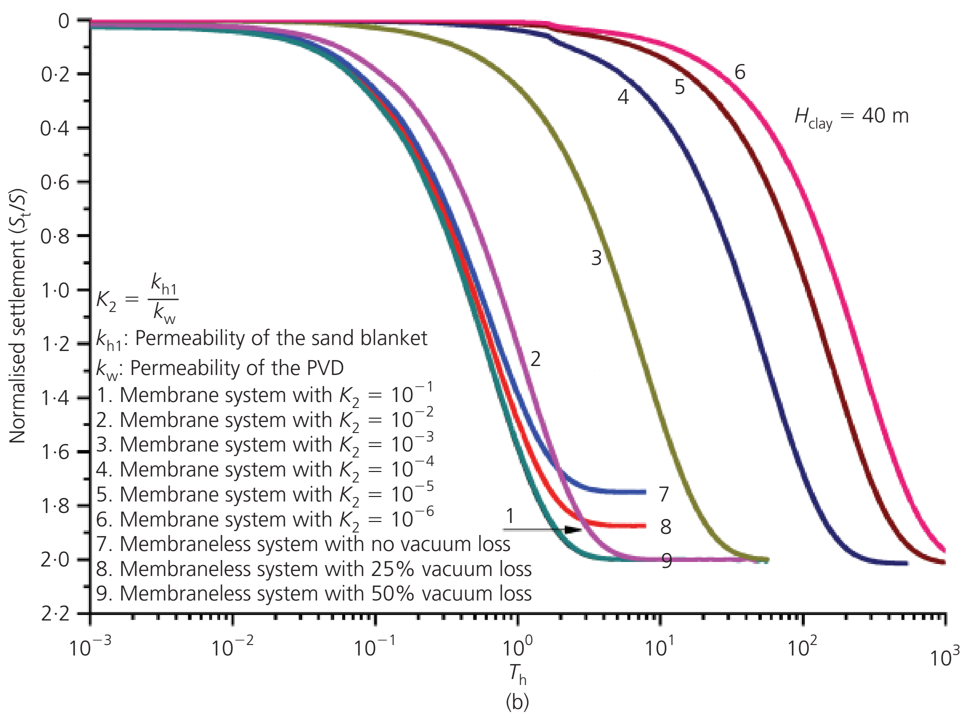


Figure 7. Normalised settlement–time factor curves for varying the permeability of the sand blanket (for membrane system) and the vacuum loss (for membraneless system): (a) clay thickness of 10 m; (b) clay thickness of 40 m (after Geng *et al.*, 2012)

the permeability ratio of the sand blanket to PVD should be greater than 0·1, and the permeability ratio of the sand blanket to the clay layer should be at least 10^5. For a membraneless system, the possible reduction in vacuum along the length of long PVDs increases the consolidation time for a given DOC. Where there is no vacuum loss with depth, the membraneless system has the same efficiency as the membrane-type system, as shown in Figure 7 for relatively shallow (10 m) and very thick (40 m) clay layers.

4. Dynamic consolidation with enhanced drainage or vacuum

When the term 'dynamic consolidation' was coined by Menard (Menard and Broise, 1975), he envisaged the method would be used for fine-grained soils as well. However, it is now generally believed that the dynamic compaction (DC) method using heavy tamping is not suitable for fine-grained soils, particularly for soils with a plasticity index larger than 10 (Mitchell, 1981). The main reasons for the failure of DC to be used for clay are: (*a*) it is difficult for pore-water pressure to dissipate and (*b*) the impact load damages the structure and fabric of soil. To overcome this problem, a combined DC with PVD method has been proposed by Zheng *et al.* (2004). In this method, a proper drainage system is installed before compaction. For compaction, it is suggested to begin the process with low compaction energy for the first pass and then increase the energy gradually for the subsequent passes. The rationale is to consolidate the top soil to form a 'hard crust' first. Once a 'hard crust' has been formed, larger compaction energy can be applied and soil at a greater depth can be compacted. A case study was presented by Zheng *et al.* (2004) in which the drainage-enhanced dynamic consolidation method was used to treat a site consisting of soft silty clay of 2–7 m deep with a sandy clay below. The PVD spacing was 1·7–2 m in a square grid. The sand blanket was 1·5 m thick. The cone penetration test (CPT) tip resistance has increased two to three times up to 5·5 m after dynamic compaction. Similar techniques have also been used in other countries (Lee and Karunaratne, 2007; Perucho and Olalla, 2006). A similar effect of using vibration on top of the fill used for a combined vacuum and fill surcharge project has also been adopted by Varaksin and Yee (2007).

A variation of the above technique is to use deep dewatering wells together with dynamic compaction for soft clay (Xu *et al.*, 2003). In this method, the soil is compacted using surface compaction or small energy dynamic compaction first to generate excess pore-water pressures. Deep well points are then installed to dissipate the excess pore-water pressures. After the excess pore-water pressures are reduced, the deep well points are removed and the second round of dynamic compaction and dewatering is carried out. This method is more effective than the use of PVDs alone, as suction creates a much higher hydraulic gradient to speed up the dissipation of excess pore-water pressure. The well points can also be installed at the points where the excess pore-water pressure is the highest. The holes left after the withdrawal of the pipes for dewatering also helps in the dissipation of excess pore-water pressure generated in the subsequent compaction. This method has been used for a number of projects in China. However, the method may only be effective when the depth of soil to be improved is less than 8 m, which is inherently the limitation of dynamic compaction with the common level of compaction energy. It may also be less effective for soils with high plasticity index (probably higher than 20). Another method that combines deep blasting with shallow compaction and deep dewatering well has also been patented by Liu and Xu (2007). However, those methods have yet to be applied in practice on a large scale. More field studies with proper instrumentations are required.

5. Conclusions

An overview of some recent developments in the areas of preloading using PVDs, vacuum consolidation and dynamic consolidation with enhanced drainage is presented in this paper. The main points discussed are summarised below.

(*a*) Theories for consolidation of soil using PVDs based on both Darcian and non-Darcian flow, and solutions or numerical procedures to consider the non-linear variation of permeability with stress or void ratio of soil, have been proposed. These theoretical improvements will in theory allow better prediction of the excess pore-water pressure or the degree of consolidation to be achieved.

(*b*) Factors affecting the consolidation of soil around PVDs include the soil parameters, c_h and k_h, the properties of the smear zone and the properties of the PVD. Both c_h and k_h are stress-history- or stress-state-dependent parameters and thus have to be selected based on the stress conditions. For the same reason, the variation of c_h and k_h with stress state or void ratio should be modelled using analytical or numerical models. The smear zone properties are difficult to determine as this zone is affected by the mandrel used, the method used to insert the mandrel and the type of soil. Various studies indicate that the diameter of the smear zone d_s ranges from 1·5 to 6 times the equivalent diameter of the mandrel d_m, or $d_s = (1{\cdot}5 \text{ to } 6)d_m$ based on laboratory model tests. However, the values measured in the field can be even higher, $d_s = (1{\cdot}5 \text{ to } 11)d_m$. The difference between the field and laboratory measurements reflects the effect of soil structure or fabric. The ratio between the permeability of the intact soil k_h and that of the smeared soil k_s is between 2 and 10, or $k_h/k_s = 2{\sim}10$, with the higher values measured in the field.

(*c*) Well resistance effects may be ignored if the discharge capacity, q_w, is sufficiently large. The required q_w value may be calculated as $q_{req} \geqslant 7{\cdot}85F_s k_h l_m^2$, where F_s is a factor of safety to consider the effect of buckling and large deformation of PVD on q_w.

(*d*) The vacuum preloading system normally requires a membrane to be used to seal the soil to be consolidated, such as the China or the Menard system. Membraneless vacuum systems have also been developed. This includes the BeauDrain system, in which each PVD is connected directly to the vacuum pump through plastic pipes, and the low-level vacuum preloading method. Each method has its own advantages and disadvantages. The suitability of the methods is project specific and should be evaluated based on cost and reliability of the method for the given site conditions.

(*e*) It is possible to use dynamic compaction for the improvement of fine-grained soil if PVDs and drainage blanket are used to facilitate the dissipation of excess pore-water pressure. Pumping well dewatering can be adopted to accelerate the dissipation of pore water.

REFERENCES

Abuel-Naga HM and Bouazza A (2009) On the equivalent diameter of prefabricated vertical drain: numerical study. *Geotextiles and Geomembranes* **27(3)**: 227–231.

Abuel-Naga HM, Pender MJ and Bergado DT (2012) Design curves of prefabricated vertical drains including smear and transition zones effects. *Geotextiles and Geomembranes* **32(3)**: 1–9.

Almeida MSS and Ferreira CAM (1993) Field in situ and laboratory consolidation parameters of a very soft clay. In *Predictive Soil Mechanics, Proceedings of the Worth Memorial Symposium*. Thomas Telford, London, UK, pp. 73–93.

Arulrajah A, Bo MW and Chu J (2009) Instrumentation at the Changi land reclamation project, Singapore. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **162(1)**: 33–40.

Barron RA (1948) Consolidation of fine-grained soils by drain wells. *Transactions of ASCE* **113(2346)**: 718–754.

Basu D and Prezzi M (2007) Effect of the smear and transition zones around prefabricated vertical drains installed in a triangular pattern on the rate of soil consolidation. *International Journal of Geomechanics, ASCE* **7(1)**: 34–43.

Basu P, Basu D and Prezzi M (2010) Analysis of PVD-enhanced consolidation with soil disturbance. *Proceedings of the Institution of Civil Engineers – Ground Improvement* **163(4)**: 237–249.

Bergado DT and Long PV (1994) Numerical analysis of embankment on subsiding ground improved by vertical drains and granular piles. *Proceedings of the 13th International Conference Soil Mechanics and Foundation Engineering, New Delhi, India*, pp. 1361–1366.

Bergado DT, Asakami H, Alfaro MC and Balasubramaniam AS (1991) Smear effects of vertical drains on soft Bangkok clay. *Journal of Geotechnical Engineering, ASCE* **117(10)**: 1509–1530.

Bergado DT, Anderson LR, Miura N and Balasubramaniam AS (1996) *Soft Ground Improvement in Lowland and Other Environments*. ASCE Press, New York, USA.

Bergado DT, Balasubramaniam AS, Fannin RJ and Holtz RD (2002) Prefabricated vertical drains (PVDs) in soft Bangkok clay: a case study of the New Bangkok International Airport project. *Canadian Geotechnical Journal* **39(2)**: 304–315.

Bo MW, Chu J, Low BK and Choa V (2003) *Soil Improvement: Prefabricated Vertical Drain Technique*. Thomson Learning, Singapore.

Bo MW, Chu J and Choa V (2005) Changi East reclamation and soil improvement project. In *Ground Improvement – Case Histories* (Indraratna B and Chu J (eds)). Elsevier, Amsterdam, The Netherlands, Ch. 9, pp. 247–276.

Bo MW, Arulrajah A and Nikraz H (2007) Preloading and prefabricated vertical drains design for foreshore land reclamation projects: a case study. *Proceedings of the Institution of Civil Engineers – Ground Improvement* **11(2)**: 67–76.

Carillo N (1942) Simple two- and three -dimensional cases in the theory of consolidation of soils. *Journal of Mathematics and Physics* **21(1)**: 1–5.

Chai JC, Miura N, Sakajo S and Bergado DT (1995) Behaviour of vertical drain improved subsoil under embankment loading. *Soils and Foundations* **35(4)**: 49–61.

Chai JC and Miura N (1999) Investigation of factors affecting vertical drain behavior. *Journal of Geotechnical Engineering, ASCE* **125(3)**: 216–226.

Chai JC, Miura N and Bergado DT (2008) Preloading clayey deposit by vacuum pressure with cap-drain: Analyses versus performance. *Geotextiles and Geomembranes* **26(3)**: 220–230.

Chai JC, Bergado DT and Shen SL (2013) Modelling prefabricated vertical drain improved ground in plane strain analysis. *Proceedings of the Institution of Civil Engineers – Ground Improvement* **166(2)**: 65–77.

Chen H and Bao XC (1983) Analysis of soil consolidation stress under the action of negative pressure. *Proceedings of the European Conference on Soil Mechanics and Foundation Engineering*, vol. 2, pp. 591–596.

Choa V, Bo MW and Chu J (2001) Soil improvement works for Changi East reclamation project. *Ground Improvement* **5(4)**: 141–153.

Chu J and Raju V (2012) Prefabricated vertical drains. In *Ground Improvement* (Kirsch K and Bell A (eds)), 3rd edn. CRC Press, Florida, USA, pp. 87–167.

Chu J and Yan SW (2005a) Estimation of degree of consolidation for vacuum preloading projects. *International Journal of Geomechanics, ASCE* **5(2)**: 158–165.

Chu J and Yan SW (2005b) Application of the vacuum preloading method in land reclamation and soil improvement projects. In *Ground Improvement – Case Histories* (Indraratna J and Chu J (eds)). Elsevier, Amsterdam, The Netherlands, Ch. 3, pp. 91–118.

Chu J, Yan SW and Yang H (2000) Soil improvement by the vacuum preloading method for an oil storage station. *Géotechnique* **50(6)**: 625–632.

Chu J, Bo MW, Chang MF and Choa V (2002) Consolidation and permeability properties of Singapore marine clay. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* **128(9)**: 724–732.

Chu J, Bo MW and Choa V (2004) Practical consideration for using vertical drains in soil improvement projects. *Geotextiles and Geomembranes* **22(1–2)**: 101–117.

Chu J, Bo MW and Choa V (2006) Improvement of ultra-soft soil using prefabricated vertical drains. *Geotextiles and Geomembranes* **24(6)**: 339–348.

Chu J, Bo MW and Arulrajah A (2009a) Soil improvement works for an offshore land reclamation. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **162(1)**: 21–32.

Chu J, Bo MW and Arulrajah A (2009b) Reclamation of a slurry pond in Singapore. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **162(1)**: 13–20.

Chu J, Varaksin S, Klotz U and Mengé P (2009c) Construction processes. *Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering, Alexandria, Egypt*, vol. 4, pp. 3006–3135, state-of-the-art report.

Chu J, Indraratna B, Yan SW and Rujikiamjorn C (2012) Soft soil improvement through consolidation: An overview. *Proceedings of the International Conference on Ground Improvement and Ground Control, Wollongong, Australia*, pp. 251–280, state-of-the-art report.

Cognon JM (1991) Vaccum consolidation. *Révue Française Géotechnique* **57(3)**: 37–47.

Geng XY, Indraratna B and Rujikiatkamjorn C (2012) Analytical solutions for a single vertical drain with vacuum and time-dependent surcharge preloading in membrane and membraneless systems. *International Journal of Geomechanics, ASCE* **12(1)**: 27–42.

Hansbo S (1960) Consolidation of clay, with special reference to influence of vertical sand drains. *Proceedings of the Swedish Geotechnical Institute 18, Linkoping, Sweden*.

Hansbo S (1979) Consolidation of clay by band-shaped prefabricated drains. *Ground Engineering* **12(5)**: 16–25.

Hansbo S (1981) Consolidation of fine-grained soils by prefabricated drains. *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, Sweden, vol. 3, pp. 677–682.

Hansbo S (1997) Aspects of vertical drain design: Darcian or non-Darcian flow. *Géotechnique* **47(5)**: 983–992.

Hansbo S (2001) Consolidation equation valid for both Darcian or non-Darcian flow. *Géotechnique* **51(1)**: 51–54.

Hansbo S (2005) Experience of consolidation process from test areas with and without vertical drains. In *Ground Improvement – Case Histories* (Indraratna J and Chu (eds)). Elsevier, Amsterdam, The Netherlands, Ch. 1, pp. 3–50.

Hird CC and Moseley VJ (2000) Model study of seepage in smear zones around vertical drains in layered soil. *Géotechnique* **50(1)**: 89–97.

Hird CC, Pyrah IC and Russel D (1992) Finite element modeling of vertical drains beneath embankments on soft ground. *Géotechnique* **42(3)**: 499–511.

Holtz RD (1975) Preloading by vacuum: current prospects. *Transportation Research Record* **48(5)**: 26–79.

Holtz RD and Broms BB (1972) Long-term loading tests and Ska-Edeby, Sweden. *Proceedings of the ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, Purdue University*, Indiana, USA, vol. 1(1), pp. 435–464.

Holtz RD, Jamiolkowski M, Lancellotta R and Pedroni R (1991) *Prefabricated Vertical Drains: Design and Performance*. Butterworth-Heinemann, London, UK, CIRIA ground engineering report.

Indraratna B (2009) *Recent Advances in the Application of Vertical Drains and Vacuum Preloading in Soft Soil Stabilization*. Australian Geomechanics Society, Australia, E. H. Davis Lecture.

Indraratna B and Redana IW (1997) Plane strain modelling of smear effects associated with vertical drains. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* **123(5)**: 474–478.

Indraratna B and Redana IW (1998) Laboratory determination of smear zone due to vertical drain installation. *Journal of Geotechnical Engineering, ASCE* **124(2)**: 180–184.

Indraratna B and Redana IW (2000) Numerical modelling of vertical drains with smear and well resistance installed in soft clay. *Canadian Geotechnical Journal* **37(1)**: 132–145.

Indraratna B, Rujikiatkamjorn C, Balasubramaniam AS and Wijeyakulasuriya V (2005a) Predictions and observations of soft clay foundations stabilised with geosynthetic drains and vacuum surcharge. In *Ground Improvement – Case Histories* (Indraratna B and Chu J (eds)). Elsevier, Amsterdam, The Netherlands, pp. 199–229.

Indraratna B, Rujikiatkamjorn C and Sathananthan I (2005b) Analytical and numerical solutions for a single vertical drain including the effects of vacuum preloading. *Canadian Geotechnical Journal* **42(4)**: 994–1014.

Indraratna B, Rujikiatkamjorn C, Kelly R and Buys H (2010) Sustainable soil improvement via vacuum preloading. *Proceedings of the Institution of Civil Engineers – Ground Improvement* **163(1)**: 31–42.

Indraratna B, Rujikiatkamjorn C, Ameratunga J and Boyle P (2011) Performance and prediction of vacuum combined surcharge consolidation at Port of Brisbane. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* **137(11)**: 1009–1018.

Indraratna B, Rujikiatkamjorn C, Kelly R and Buys H (2012) Soft soil foundation improved by vacuum and surcharge loading. *Proc. of the ICE - Ground Improvement* **165(2)**: 87–96.

Kitazume M (2007) Design, execution and quality control of ground improvement in land reclamation. Keynote lecture. *Proceedings of the 13th Asian Regional Conference on Soil Mechanics and Foundation Engineering*. Allied Publishers, New Delhi, India.

Kjellman W (1952) Consolidation of clayey soils by atmospheric pressure. *Proceedings of a Conference on Soil Stabilization, MIT, Boston, USA*, pp. 258–263.

Kolff AHN, Spierenburg SEJ and Mathijssen FAJM (2004) BeauDrain: a new consolidation system based on the old concept of vacuum consolidation. *Proceedings of the 5th International Conference on Ground Improvement Techniques, Kuala Lumpur, Malaysia*.

Lee CH and Kang ST (1996) *Discharge Capacity of Prefabricated Vertical Band Drains*. Final year project report, Nanyang Technical University, Singapore.

Lee SL and Karunaratne GP (2007) Treatment of soft ground by Fibredrain and high-energy impact in highway embankment construction. *Proceedings of the Institution of Civil Engineers – Ground Improvement* **11(4)**: 181–194.

Li AL and Rowe RK (2001) Combined effects of reinforcement and prefabricated vertical drains on embankment performance. *Canadian Geotechnical Journal* **38(6)**: 1266–1282.

Liu HL and Xu SL (2007) *Combined Shallow Compaction, Deep Blasting and Vacuum Dewater Method for Ground Improvement*. Chinese Patent CN200710024872·1.

Madhav MR, Park YM and Miura N (1993) Modelling and study of smear zones around band shaped drains. *Soils and Foundations* **33(4)**: 135–147.

Menard L and Broise Y (1975) Theoretical and practical aspects of dynamic consolidation. *Géotechnique* **25(1)**: 3–18.

Mitchell JK (1981) State of the art – soil improvement. *Proceedings of the 10th International Conference Soil Mechanics and Foundation Engineering, Stockholm, Sweden*, vol. 4, pp. 509–565.

Onoue A, Ting NH, Germain JT, Whitman RV (1991) Permeability of Disturbed Zone Around Vertical Drains. In *Proceedings of American Society of Civil Engineering, Congress, Colorado, USA*, pp. 879–890.

Perucho A and Olalla C (2006) Dynamic consolidation of a saturated plastic clayey fill. *Proceedings of the Institution of Civil Engineers – Ground Improvement* **10(2)**: 55–68.

Pothiraksanon C, Saowapakpiboon J, Bergado DT, Voottipruex P and Abuel-Naga HM (2010) Soft ground improvement with solar-powered drainage. *Proceedings of the Institution of Civil Engineers – Ground Improvement*. **163(1)**: 23–30.

Rujikiatkamjorn C, Indraratna BN and Chu J (2008) 2D and 3D numerical modeling of combined surcharge and vacuum preloading with vertical drains. *International Journal of Geomechanics* **8(2)**: 144–156.

Sathananthan I and Indraratna B (2006) Laboratory evaluation of smear zone and correlation between permeability and moisture content. *Journal of Geotechnical and Geoenvironmental Engineering* **132(7)**: 942–945.

Seah TH (2006) Design and construction of ground improvement works at Suvarnabhumi Airport. *Geotechnical Engineering Journal of Southeast Asian Geotechnical Society* **37(3)**: 171–188.

Tang M and Shang JQ (2000) Vacuum preloading consolidation of Yaoqiang Airport runway. *Géotechnique* **50(6)**: 613–623.

Varaksin S and Yee K (2007) Challenges in ground improvement techniques for extreme conditions: Concept and performance. *Proceedings of the 16th Southeast Asian Geotechnical Conference, Kuala Lumpur, Malaysia*, pp. 101–115.

Walker R and Indraratna B (2006) Vertical drain consolidation with parabolic distribution of permeability in smear zone. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* **132(7)**: 937–941.

Walker R, Indraratna B and Rujikiatkamjorn C (2012) Vertical drain consolidation with non-Darcian flow and void-ratio-dependent compressibility and permeability. *Géotechnique* **62(11)**: 985–997.

Xiao DP (2002) *Consolidation of Soft Clay Using Vertical Drains*. PhD thesis, Nanyang Technological University, Singapore.

Xu SL, Lu XM, Liu CM and Liu YY (2003) Field trials of the vacuum compaction method for soil improvement. *Proceedings of the 9th National Geotechnical Conference, Beijing, China*, vol. 2, pp. 736–739.

Yan SW and Chu J (2005) Soil improvement for a storage yard using the combined vacuum and fill preloading method. *Canadian Geotechnical Journal* **42(4)**: 1094–1104.

Yan SW and Chu J (2003) Soil improvement for a road using the vacuum preloading method. *Proceedings of the Institution of Civil Engineers – Ground Improvement* **7(4)**: 165–172.

Yan SW, Chu J, Fan QJ and Yan Y (2009) Building a breakwater with prefabricated caissons on soft clay. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **162(1)**: 3–12.

Yoshikuni H and Nakanodo H (1974) Consolidation of fine-grained soils by drain wells with finite permeability. *Soil Mechanics and Foundation Engineering* **14(2)**: 35–46.

Zeng GX and Xie KH (1989) New development of the vertical drain theories. *Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, the Netherlands*, vol. 2, pp. 1435–1438.

Zheng YR, Chu J, Lu X and Feng YX (2004) Improvement of soft ground by dynamic compaction. *Geotechnical Engineering, Journal of Southeast Asian Geotechnical Society* **35(1)**: 39–46.