Performance Monitoring of Vacuum Preloading for Stabilising Soft Foundations for Transportation and Port Infrastructure

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ABSTRACT

A system of prefabricated vertical drains (PVDs) 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. These recent techniques have been applied to various real life projects in Australia and Southeast Asia. Some of the new design concepts include the role of overlapping smear zones due to PVD-mandrel penetration, single drain and multi-drain analyses. These recent advances enable greater accuracy in the prediction of excess pore water pressure, and lateral and vertical displacement of the stabilised ground. This keynote paper presents an overview of the theoretical and practical developments and salient findings of soft ground improvement via PVD and vacuum preloading, with applications to selected case studies in Australia.

1. INTRODUCTION

Preloading of soft clay using vertical drains is one of the most commonly employed techniques to improve the shear strength of soil and to curtail its post-construction settlement. Most estuarine soils are characterized by very low hydraulic conductivity and considerable thickness. Therefore, the time required to attain the desired settlement or shear strength can sometimes be too long to comply with the need for rapid construction (Johnson, 1970 and Indraratna et al. 1992). Using wick drains, the drainage length is reduced from the soft soil thickness to one-half of the drain spacing in the horizontal direction. In addition, for most soil deposits, the horizontal hydraulic conductivity is much greater than the vertical hydraulic conductivity, hence, the consolidation process can be accelerated (Jamiolkowski et al., 1983). This system has been employed successfully to stabilised foundation soils for embankments, airports, railways and highways (Kjellman, 1948).

The performance of various types of vertical drains including sand drains, prefabricated vertical drains (PVDs) and gravel piles have been investigated in the past three decades. Sand drains were firstly employed in practice around 1920’s. The laboratory and in-situ tests of the sand drains have been conducted by the California Division of Highways in 1933. Since 1940, Kjellman introduced prefabricated band shaped drains and cardboard wick drains for ground improvement. Subsequently, several types of prefabricated band drains were developed such as: Geodrain (Sweden), Alidrain (England), Mebradrain (Netherlands) etc. Typically, the PVDs are a combination of a plastic core
with a longitudinal channel functioning as drainage, and a sleeve of paper or fibrous material as a filter protecting the silt intrusion. Figure 1 shows a typical scheme of vertical drains installation and monitoring instruments required to monitor the performance of the soil foundation beneath an embankment. Before installing the vertical drains, general site preparation including the removal of peat and surficial debris, establishing site grading and constructing a sand blanket is essential. The sand blanket system is employed to drain water from the PVDs and to provide a sound-working mat.

**Fig. 1:** Basic Instrumentation of Embankment (After Indraratna et al. 2005)

Application of vacuum (suction) load via PVDs can further speed up the rate of settlement via increased hydraulic gradient (Indraratna et al. 2004). When a high surcharge pressure is required to minimize secondary consolidation settlement, the optimum solution is the combination of vacuum and fill surcharge. In very soft clays, the vacuum pressure application is often the obvious choice, especially when the soft clays have very low undrained shear strength within a stringent construction time schedule. The PVD system has been used to facilitate vacuum pressure to deep subsoil layer, thereby improving the consolidation rate of reclaimed land on deep estuarine plains (e.g. Indraratna et al. 2005, Chu et al. 2000; Rankine et al. 2008). The mechanism of the vacuum preloading can be described by Fig. 2. In this mechanism the effective stress increases due to the vacuum load, while the total stress remains constant.

**2. VACUUM PRELOADING SYSTEM**

For vacuum preloading system, the installation of some horizontal drains in the transverse and longitudinal directions is required after installing the sand platform in order to equally distribute the applied suction (generated using vacuum pump). Subsequently, these drains can be linked at the edge with a peripheral Bentonite slurry trench, which is normally sealed by an impervious membrane (membrane system) (Fig. 3a). The trenches can then be filled with water to improve the sealing between the membrane and the Bentonite slurry. The vacuum pumps are then connected to the discharge system extending from the trenches.

**Fig. 2:** Consolidation Process: (a) Conventional loading (b) Idealised Vacuum Preloading (Inspired by Indraratna et al. 2005)

**Fig. 3:** Vacuum-assisted Preloading System (a) Membrane System and (b) Membraneless System (After Indraratna et al. 2005)
When an area has to be separated into a number of sections to facilitate the installation of the PVDs, the vacuum preloading can only be carried out in one section at a time. This procedure becomes increasingly unmanageable when the vacuum preloading method is used for instance in land reclamation over a very large area. One way of overcoming this issue is to connect the vacuum pump directly to each individual PVD using a tubing system (Membraneless system). In this system, each individual PVD is attached directly to the drain collector (Fig. 3b). Unlike in the membrane system where any air leak can affect all the drains, in this system each drain acts independently. However, the extensive tubing for hundreds of drains can affect the installation time and cost (Seah, 2006).

3. FACTOR AFFECTING PERFORMANCE OF PRELOADING

Smear Zone

The smear zone is created during the installation of PVD by steel mandrel using displacement installation technique. Horizontal soil hydraulic conductivity adjacent to PVDs can reduce substantially, which in turn reduces the excess pore pressure dissipation rate and retards the consolidation process. Indraratna and Redana (1998) and Indraratna and Sathananthan (2006) showed that the reduction in soil permeability can be determined via the ratio of average soil permeability in the smear zone and the in-situ horizontal soil permeability ($k_h/k_s$) or the variation of water content (Fig. 4). This $k_h/k_s$ ratio and the extent of smear zone ($d$) vary between 2-10 and 3-4 times of mandrel size, respectively, depending on the soil sensitivity, mandrel shape, installation speed, and the soil macro fabric. Walker and Indraratna (2007) demonstrated that the smear effects can overlap creating more soil disturbance when vertical drains are installed too close (Fig. 5).

![Fig. 4: Smear Zone Determination Using (a) Permeability Ratio and (b) Water Content (After Sathananthan and Indraratna, 2006)](image)

![Fig. 5: Overlapping Smear Zones Between Adjacent Drains (After Walker and Indraratna, 2007)](image)

Recently, the smear zone around rectangular and rhomboidal mandrels was characterized by Ghandeharioon et al. (2010) using an elliptical cavity expansion theory (CET) that incorporates a modified Cam clay model for soft soil. Comparing the laboratory test results on soil permeability conducted by Sathananthan and Indraratna (2006), Hird and Moseley (2000) and Indraratna and Redana (1998), the plastic shear strain normalized by the rigidity index, $I_r = \sqrt{3G/q_f}$, can be used to characterize the extent of smear zone surrounding the mandrel-driven prefabricated vertical drains. As depicted in Fig. 6, the ratio $\gamma_q I_r$ is approximately 0.86%-1.05% at the boundary of total remoulded soil. The smear zone, where the horizontal hydraulic conductivity changes rapidly, expands to where the normalized plastic shear strain is about 0.10%-0.17%. Based on the range of strains identified previously and illustrated in Fig. 6, the diameter of the smear zone is determined to be 3.07 times the equivalent diameter of the mandrel.

![Fig. 6: The Distribution Pattern for the Normalized Plastic Shear Strain to the Rigidity Index in Relation to the Normalized Radial Distance Characterizing the Smear Zone Surrounding a PVD (Ghandeharioon et al. 2010)](image)
Drain Unsaturation
Unsaturation of soil adjacent to the drain can occur due to mandrel withdrawal (air gap) and dry condition of PVDs. The apparent retardation of pore pressure dissipation and consolidation can be found at the initial stage of loading (Indraratna et al. 2004).

Vacuum Loss
Indraratna et al. (2004) showed that the vacuum pressure is distributed along the drain length. The efficiency of distribution of vacuum pressure along the drain length depends on the sealing (i.e. no air leaks) and the type of soil around the drain. The laboratory measurements at a few points along the drain in the large-scale consolidometer clearly indicated that the magnitude of vacuum pressure gradually decreases along the drain length, resulting in an overall reduction of consolidation settlement. This effect will be included in the theoretical development in the following section.

Determination of Degree of Consolidation
The degree of consolidation can be determined based on excess pore pressure or settlement. The degree of consolidation based on settlement can be calculated as:

\[ U = \frac{s_t}{s_{ult}} \tag{1} \]

where, \( s_t \) = settlement measured at time \( t \), \( s_{ult} \) = ultimate settlement.

The degree of consolidation based on excess pore pressure (Fig. 7) can be expressed as:

\[ U_p = \frac{h_r \gamma_r + h_{fill} \gamma_{fill} - u_t}{h_{fill} \gamma_{fill} + \text{VP}} = \frac{\Delta u_t}{\Delta \sigma + \text{VP}} \tag{2} \]

where, \( h_r \), \( \gamma_r \), \( h_{fill} \), \( \gamma_{fill}\), \( u_t \) = effective stress, effective density, pore pressure, effective density, excess pore pressure, and VP = Vacuum Pressure.

**Fig. 7:** Pore Pressure Distribution with Depth

Where, VP= Vacuum Pressure.

4. CONSOLIDATION THEORY CONSIDERING RADIAL DRAINAGE

Single Drain Theory
A rigorous radial consolidation theory combining both the smear effect and well resistance was proposed by Barron (1948) and Hansbo (1981). Application of vacuum pressure with vertical drainage (i.e. no vertical drains), was modelled by Mohamedelhassan and Shang (2002) based on Terzaghi’s one-dimensional consolidation theory. The above mathematical models are based on small strain theory over a given stress range with constant volume compressibility \((m)\) and a constant coefficient of horizontal permeability \((k_p)\). However, the value of \( m \) non-linearly changes over a wide range of applied pressure \((\Delta p)\). In the same manner, \( k_p \) also varies with the void ratio \((e)\). In this section, the use of compressibility indices \((C_e\) and \(C_{ic})\), which present the slopes of the \( e\)-\( \log \sigma' \) relationship, and the variation of lateral permeability coefficient \((k_{vl})\) with void ratio \((e)\) are included in the radial consolidation theory with the effects of vacuum pressure distribution and parabolic permeability variation in the smear zone.

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

1. According to laboratory observation, at a few points along the length of drain in the large-scale consolidometer (Fig. 8a), the vacuum pressure clearly decreases down the drain length (Indraratna et al., 2004). In the analysis, vacuum pressure is assumed to vary linearly from \( p_o \) at the top of the drain to \( k_1 p_o \) at the bottom of the drain.

2. Homogenous soil is fully saturated and the Darcy’s law is adopted. At the external periphery of the unit cell, flow is not allowed to occur (Fig. 8b), hence, only radial (horizontal) flow is permitted.

3. The relationship between the average void ratio and the logarithm of average effective stress in the normally consolidated range (Fig. 9a) can be expressed by: \( \bar{e} = e_0 - C_e \log(\sigma/\sigma'_1) \).

4. For radial drainage, the horizontal permeability of soil decreases with the average void ratio (Fig. 9b). The relationship between these two parameters is given by Tavenas et al. (1983) by: \( \bar{e} = e_0 + C_k \log(k_h/k_{hl}) \). The permeability index \((C_k)\) is generally considered to be independent of stress history \((p')\) as explained by Nagaraj et al. (1994).

**Fig. 8:** (a) Distributions of Measured Negative Pore Pressure Along Drain Boundary in Laboratory Testing, (b) Distributions of Vacuum Pressure in Model (Indraratna et al. 2005)
The dissipation rate of average excess pore pressure \( R_u = \frac{\bar{u}_t}{\Delta p} \) at any time factor \( (T_h) \) can be expressed as:

\[
R_u = \left(1 + p_0(1 + k_i)/2\Delta p\right) \exp\left(-8T_h^* / \mu\right) - p_0(1 + k_i)/2\Delta p
\]  

(3)

In the above expression,

\[
T_h^* = P_{av}T_h
\]  

(4)

\[
P_{av} = 0.5\left[1 + (1 + \Delta p/\sigma'_i + p_0(1 + k_i)/2\sigma'_i)^{-1}C_0/C_i\right]
\]  

(5)

\[
T_h = c_h t / d_e^2
\]  

(6)

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

\[
\mu = \ln n / s + k_h / k'_h \ln s - 0.75
\]  

(6a)

However, laboratory testing conducted using large-scale consolidometer by Onoue et al. (1991), Indraratna and Redana (1998) and Sharma and Xiao (2000) suggests that the disturbance in the ‘smear zone’ increases towards the drain (Fig. 10). 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 \( \mu \) parameter can be given by:

\[
\mu_p = \ln\left(\frac{n}{s}\right) + \frac{3}{4} + \frac{\kappa(s-1)^2}{s^2 - 2\kappa s + \kappa^2} \ln\left(\frac{s}{\sqrt{k - 1}}\right)
\]

\[
- \frac{s{s-1}}{2(s^2 - 2\kappa s + \kappa^2)} \ln\left(\frac{\sqrt{k} + \sqrt{k - 1}}{\sqrt{k} - \sqrt{k - 1}}\right)
\]  

(6b)

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

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

\[
U_p = 1 - R_u
\]  

(7)

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

\[
U_s = \rho / \rho_m
\]  

(8)

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

\[
\rho = HC_{v_i}/(1 + e_0) \log(\sigma'/\sigma'_i), \quad \sigma'_i \leq \sigma' \leq \rho'_c
\]  

(9a)

\[
\rho = HC_{v_i}/(1 + e_0) \left[C_i \log(\sigma'/\sigma'_i) + C_e \log(\sigma' / p'_c)\right], \quad \rho'_c \leq \sigma' \leq \sigma'_i + \Delta p
\]  

(9b)

\[
\rho = HC_{v_i}/(1 + e_0) \log(\sigma'/\sigma'_i) \quad \text{for normally consolidated clay}
\]  

(9c)

It is noted that \( \rho_m \) can be obtained by substituting \( \sigma' = \sigma'_i + \Delta p \) into the above equations, where: \( \rho = \) settlement at a given time, \( \rho = \) total primary consolidation settlement, \( \sigma' = \) effective in-situ stress, \( \sigma'_i = \) effective stress, \( C_v = \) compression index, \( C_e = \) recompression index and \( H = \) compressible soil thickness.

When the value of \( C/C_i \) approaches unity and \( p'_c \) becomes zero, the authors’ solution converges to the conventional solution proposed by Hansbo (1981):

\[
R_u = \exp(-8T_h / \mu)
\]  

(10)
5. MULTI-DRAIN ANALYSIS FOR A PLANE STRAIN EMBANKMENT

Indraratna et al. (2005) transformed the vertical drain system from axisymmetric (3D) to plane strain condition by determining the equivalent coefficient of soil permeability. A relationship between \( k_{hp} \) and \( k'_{hp} \) can be modelled as:

\[
\frac{k'_{hp}}{k_{hp}} = \frac{\beta}{\ln \left( \frac{n}{s} \right) + \left( \frac{k_{hp}}{k_{hp}'} \right) \ln (s) - 0.75} - \alpha
\]

(11)

\[
\mu_r = \left[ \alpha \cdot (\beta) \frac{k_{hp}'}{k_{hp}} \right]
\]

(12)

\[
\alpha = \frac{2}{3} \left( \frac{2b}{B} \left( 1 - \frac{b}{B} + \frac{b^2}{3B^2} \right) - \frac{1}{B^2} (b_s - b_u)^2 + \frac{b_s}{3B^2} (3b_u^2 - b_s^2) \right)
\]

(13)

\[
\beta = \frac{1}{B^2} (b_s - b_u)^2 + \frac{b_s}{3B^2} (3b_u^2 - b_s^2)
\]

(14)

where, \( k_{hp} \) and \( k'_{hp} \) are the undisturbed horizontal coefficient of permeability and the corresponding equivalent coefficient of permeability in smear zone, respectively.

If the smear and well resistance are ignored in the above expressions, the simplified ratio of plane strain to axisymmetric permeability can be obtained, as also proposed earlier by Hird et al. (1992), to be:

\[
\frac{k_{hp}}{k_{hp}'} = \frac{0.67}{\ln \left( \frac{n}{s} \right) - 0.75}
\]

(15)

For vacuum preloading, the equivalent vacuum pressures under plane strain and axisymmetric conditions are the identical.

6. APPLICATION TO CASE HISTORIES

Port of Brisbane

The Port of Brisbane is located at the mouth of the Brisbane River at Fisherman Islands. An expansion of the Port includes a 235 hectare area to be progressively reclaimed and developed over the next 20 years using dredged materials from the Brisbane River and Moreton Bay shipping channels. The site contains compressible clays of over 30 m in thickness. At least 7 m of dredged mud capped with 2 m of sand was used to reclaim the sub-tidal area. Generally, the complete consolidation of the soft deep clay deposits may well take in excess of 50 years, if surcharging was the only treatment, with associated settlements of probably 2.5-4 m likely. To reduce the consolidation period, the method of PVDS and vacuum pressure combined with surcharge or PVD (at sites where stability is of a concern) was chosen to be trialled. Three contractors undertook the trial works being Austress Menard, Van Oord and Cofra/Boskalis. All three trialled PVDs and surcharge with Austress Menard and Cofra/Boskalis also trialling their respective proprietary membrane and membrane-less vacuum systems. Figure 10 shows the final layout of a typical trial area with the PVDs design results for each area.

The typical soil properties are summarised in Table 1.

Table 1: Typical Soil Properties (Port of Brisbane Corporation and Austress Menard 2008)

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Type</td>
<td>Dredged Mud</td>
<td>Upper Holocene Sand</td>
<td>Upper Holocene Clay</td>
<td>Lower Holocene Clay</td>
</tr>
<tr>
<td>( \gamma' ) (kN/m³)</td>
<td>14</td>
<td>19</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>( C_s / (1 + e_u) )</td>
<td>0.3</td>
<td>0.01</td>
<td>0.18</td>
<td>0.235</td>
</tr>
<tr>
<td>( C_v (m^2/yr) )</td>
<td>1</td>
<td>5</td>
<td>5</td>
<td>0.8</td>
</tr>
<tr>
<td>( C_h (m^2/yr) )</td>
<td>1</td>
<td>5</td>
<td>2</td>
<td>1.9</td>
</tr>
<tr>
<td>( k_h / k_s )</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>( s = d_i / d_w )</td>
<td>1</td>
<td>1</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>( C_a / (1 + e_u) )</td>
<td>0.005</td>
<td>0.001</td>
<td>0.008</td>
<td>0.0076</td>
</tr>
</tbody>
</table>

The lateral displacement normalised by the applied effective stress at two inclinometer positions (MS24 and MS34) are plotted in Fig. 11 in order to compare two locations with a different loading history. It was clear that...
the lateral displacements were maximum in the upper Holocene shallow clay depths and were insignificant below 10m. From this limited inclinometer data, the membraneless BeauDrain system (MS34) had controlled the lateral displacement more effectively than the surcharge only section (MS24). Excess pore water pressure and settlement predictions and field data for a typical settlement plate (TSP3) are shown in Fig. 12. The predicted settlement curve agreed with the field data. The excess pore water pressures in every section monitored, in spite of the PVDs. From the perspective of stability, the incremental rate of change of the lateral displacement/settlement ratio ($\mu$) with time can be plotted as shown in Fig. 13. This rate of change of $\mu$ can be determined for relatively small time increments where a small and decreasing gradient can be considered to be stable with respect to lateral movement, while a continuously increasing gradient of reflects potential lateral instability. In Fig. 13, the gradient in the non-vacuum area WD3 increased initially, which could be attributed to the final surcharge loading placed quickly, while the clay was still at early stages of consolidation. However, as the PVDs become fully active and settlement increased at a healthy rate, the gradient of decreased, as expected. In general, Figure 13 illustrates that the vacuum pressure provides a relatively unchanging gradient of with time.

![Fig. 11: Comparison of Lateral Displacements in Vacuum and Non-vacuum Areas](image)

![Fig. 12: (a) Settlement; and (b) Excess Pore Water Pressure Predictions and Field Data for a Typical Settlement Plate Location](image)

![Fig. 13: Rate of Change of Lateral Displacement/Settlement Ratio with Time](image)
consolidation (TA8) and membrane type (VC1-5) were also small.

Ballina Bypass
The Pacific Highway linking Sydney and Brisbane is constructed to reduce the high traffic congestion in Ballina. This bypass route has to cross a floodplain consisting of highly compressible and saturated marine clays of up to 40 m thick. A system of vacuum assisted surcharge load in conjunction with PVDs was selected to shorten the consolidation time and stabilise the deeper clay layers. To investigate the effectiveness of this approach, a trial embankment was built north of Ballina, 34mm diameter circular PVD at 1.0m spacing were installed in a square pattern. The vacuum system consisted of PVDs with an air and water tight membrane, horizontal transmission pipes, and a heavy duty vacuum pump. Transmission pipes were laid horizontally beneath the membrane to provide uniform distribution of suction. The boundaries of the membrane were embedded in a peripheral trench filled with soil-bentonite to ensure absolute air tightness. Figure 15 presents the instrument locations, including surface settlement plates, inclinometers and piezometers. The piezometers were placed 1m, 4.5m, and 8m below the ground level, and eight inclinometers were installed at the edges of each embankment. The embankment area was then divided into Section A (no vacuum pressure), and Section B, subject to vacuum pressure and surcharge fill. As the layers of soft clay fluctuated between 7m to 25 m (Table 2), the embankment varied from 4.3m to 9.0m high, to limit the post-construction settlement. A vacuum pressure of 70 kPa was applied at the drain interface and removed after 400 days. The geotechnical parameter of the three subsoil layers obtained from standard oedometer tests are given in Table 3.
Settlement and associated pore pressure recorded by the settlement plates and piezometers are shown in Fig. 17, with the embankment construction schedule. The actual suction varied from -70 kPa to -80 kPa, and no air leaks were encountered. Suction was measured by miniature piezometers embedded inside the drains.

The predictions from 2D and 3D analyses agreed with the measured data, where the rate of settlement increased significantly after a vacuum was applied (Fig. 19c).

![Graph showing embankment stage construction with associated settlements and excess pore pressures.](image1.png)

**Fig. 17:** Embankment Stage Construction with Associated Settlements and Excess Pore Pressures (Indraratna *et al*. 2009)

Lateral displacement at the border of the embankments needed to be examined carefully, particularly the vacuum area where the surcharge loading was raised faster than that at the non-vacuum area. The soil properties and lateral displacement plots before and after vacuum are shown in Fig. 18. Inclinometer I1 was installed at the border of the vacuum area, whereas inclinometers I2-I4 were located at the edge of the vacuum area. In this case, the lateral displacement subjected to vacuum was smaller even though the embankments were higher. In Figure 18b the plot of lateral displacement normalised to embankment height showed that the vacuum pressure undoubtedly reduced lateral displacement.

2D and 3D single drain analyses were used to compute the settlement at location SP-12. The construction history and measured settlement at the settlement plate SP-12 are shown in Fig. 19a and 19b, respectively. Here the clay was assumed to 24m thick, based on the CPT data. The analytical pattern was similar to Indraratna *et al*. (2005a).

![Graph showing 2D and 3D analyses.](image2.png)

**Fig. 19:** Embankment TS1 in Thailand; (a) Stages of Loading, (b) Settlements and (c) Excess Pore Pressure
The Second Bangkok International Airport

The Second Bangkok International Airport is located in Samutprakan Province, Thailand, about 30 km east of capital city Bangkok. The subsoil is relatively uniform, consisting of a top weathered crust (1.5 m thick) overlying a layer of soft clay approximately 12 m thick. A stiff clay layer underlies the soft clay and extends to a depth of 20-24 m below the ground surface. During wet seasons, the area is often flooded, and the soil generally retains very high moisture content.

Three test embankments TS1 was constructed and stabilized with 12 m long PVDs installed in square patterns. More details of these embankments are explained elsewhere by Indraratna and Redana (2000). The embankment load was applied in stages. The Stage 1 loading was equivalent to 18 kPa, which included a sand blanket and surcharge fill having a total height of 1.0 m. The loads in Stages 2, 3 and 4 were 45 kPa, 54 kPa, and 75 kPa, respectively. The details of loading sequences for Bangkok preloading embankments TS1 is shown in Fig 19a. Figure 19b illustrates the comparison of predicted settlements at the centerline of embankments with the field measurements. The settlement predictions for all three embankments agree well with the field data. The analytically predicted and the measured excess pore pressures beneath the embankment centerline at a depth of 8m below ground surface are shown in Fig. 19c. The differences in excess pore water pressure between the predictions and field data can be attributed to the assumption of instantaneous step loading.

7. CONCLUSIONS

It is clear that the application of PVDs combined with vacuum and surcharge preloading has become common practice, and is now considered to be one of the most effective ground improvement techniques. Vacuum assisted consolidation has been successfully used for large scale projects on very soft soils in reclamation areas. The extent of surcharge fill can be decreased to achieve the same amount of settlement via vacuum consolidation. The lateral yield of the soft soil in sensitive zone can be controlled using vacuum pressure.

The lateral displacement and pore pressure dissipation associated with PVDs and vacuum pressure are often difficult to predict accurately. This may be attributed to the complexity of evaluating the true magnitudes of soil parameters inside and outside the smear zone, the correct drain properties, and aspects of the soil-drain interface unsaturation. Therefore, one needs to use the most appropriate laboratory techniques to obtain these parameters, preferably large-scale testing equipment.

From a practical point of view, the surcharge fill can be reduced in height by using vacuum preloading to achieve the same desired rate of consolidation. This system eliminates the need for a high embankment surcharge load, on the condition that air leaks must be prevented.

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