Department of Civil, Mining and Environmental Engineering

Anisotropic Visco-Plastic Behaviour of Soft Soil with Special Reference to Radial Consolidation

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"This thesis is presented as part of the requirements for the award of the Degree of Doctor of Philosophy of the University of Wollongong"

March, 2017
THESIS DECLARATION

I, Pankaj Baral, declare that this thesis, submitted in fulfilment of the requirements for the award of Doctor of Philosophy, in the School of Civil, Mining & Environmental Engineering (CME), Faculty of Engineering and Information Sciences (EIS), University of Wollongong (UOW), is wholly my own work unless otherwise referenced or acknowledged. The document has not been submitted for qualification at any other academic institution.

........................................

Pankaj Baral

March, 2017
Especially Dedicated to my Father
Late Mr. Tika Ram Baral
LIST OF PUBLICATION


ABSTRACT

Rapid population growth and the correspondingly increased demand for infrastructure necessitate effective ground improvement to support surface infrastructure. Due to the highly compressible and problematic soft soil that commonly exists in the coastal areas of Australasia, improving the ground before construction by giving it sufficient time to reach post-consolidation settlement is imperative. But these time constraints must be weighed against the urgency of the project and its projected completion date. Certainly there are various types of ground improvement techniques available for soft soil, but prefabricated vertical drains (PVDs) combined with surcharge and vacuum preloading is one of the most popular, effective, and environmentally friendly techniques. Here, the drains are driven into the ground to a considerable depth (normally the thickness of soft clay) to accelerate the dissipation of pore water pressure by promoting radial consolidation with a shortened drainage path length. The vacuum used in this system also accelerates dissipation by creating suction in the drain which helps the soil reach post-construction settlement much earlier.

Soft soil, possesses time-dependent stress-strain behaviour due to its viscous nature, and this visco-plastic behaviour has an effect on long term settlement and pore water dissipation. A novel mathematical model has been developed to describe the visco-plastic behaviour of soft clay with a non-Darcian flow function; it was developed by coupling the basic radial consolidation equation developed by Barron combined with Bjerrum’s time-equivalent (Bjerrum, 1967) concept that incorporates Yin and Graham (1989b) visco-plastic parameters. The settlement and excess pore water pressure obtained from this model are compared with pre-existing models as a Class C prediction for the Ballina trial embankment. The proposed elastic visco-plastic model gave better results in terms of settlement and pore water pressure with the field data, although the excess pore water pressure that did not dissipate after 1 year
or so is mainly due to the biological and chemical clogging of piezometers in acid sulphate soil (ASS) terrain. These predictions were also compared with the results from finite element method using PLAXIS by performing both plane strain and actual 3D models, respectively.

Similarly, to investigate the elastic visco-plastic behaviour of soft soil more clearly, the concept of strain rate dependency of pre-consolidation pressure was used along with the isotache concept. Since pre-consolidation pressure is a function of the strain rate (which was high in the laboratory experiment but very low in the field), a pressure ratio can be obtained from the strain rate and pre-consolidation pressure plot to convert laboratory pre-consolidation pressure into field pressure. Delayed consolidation could then be discovered using the change in effective stress from a pre-consolidation pressure change which actually retarded the dissipation of excess pore water pressure. This model was then used to validate the different case histories (Pacific Highway, Australia, SBIA, Bangkok and Muar, Malaysia); the results obtained using the isotache model are more promising than the other pre-existing radial consolidation model.

Laboratory tests using samples of remoulded and undisturbed Ballina clay were carried out in UOW laboratory and tested using a Rowe cell and large scale consolidometer. This 350mm diameter large scale consolidometer cum corer was used to extract an undisturbed sample from the field, and then transported into the laboratory and tested with appropriate instrumentation to determine the behaviour of this undisturbed marine soft clay containing random seashells and natural partings. Different consolidation and smear parameters were obtained from the large scale consolidometers, which were then used to predict the exact behaviour in the field, and they differed from the parameters obtained using the remoulded sample.
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CHAPTER 1: INTRODUCTION

1.1 Soft ground improvement

In order to construct safe, stable and serviceable infrastructures, Engineers must consider the ground conditions and decide whether stabilisation techniques are required or not. Building infrastructure without the appropriate ground improvement technique, especially in the coastal regions of Eastern Australia, can cause infrastructure to collapse due to soft soil which can reach significant depths. Soft soils with properties such as high compressibility, low permeability and low bearing capacity can be a challenging geotechnical concern (Indraratna et al., 2010), which is why ground improvement techniques are needed to prevent failure and minimise differential settlement.

Even though there are many ground improvement techniques available to stabilise soft ground, the use of prefabricated vertical drains with preloading is still regarded as one of the most popular and classical techniques. This process involves applying a surcharge load that is equal to or greater than the permanent foundation loading which then accelerates consolidation by allowing pore water pressure to dissipate through the vertical drains. Faster acceleration can also be achieved by applying a vacuum pressure with the surcharge; this not only enhances the rate of consolidation, it also helps to decrease the surcharge height without creating any adverse effects on the stability of the embankment.

The one dimensional consolidation theory proposed by Terzaghi (1943) has been modified by researchers (such as Bjerrum, 1967; Ladd et al., 1977; Mesri and Godlewski, 1977; Leroueil, 1988; Indraratna et al., 1992) to understand the consolidation behaviour of clays. This modification arose because the assumption made on this classical theory regarding the “constant nature of coefficient of volume compressibility and permeability during the whole consolidation
process” is unrealistic when it was found that the void ratio decreased under constant effective stress. Numerous researchers have tried to remove the simplified assumptions which have originated from classical Terzaghi’s theory. For instance, Barron (1948) derived comprehensive solution for radial consolidation based on 1-D vertical consolidation theory proposed by Terzaghi (1943), and then Hansbo (1997a) introduced a non-Darcian flow concept into radial consolidation, while Mohamedelhassan and Shang (2002) modified the consolidation solution for vacuum and surcharge preloading. Similarly, Lekha et al. (2003) modified Terzaghi’s classical theory which includes the variation of soil compressibility and permeability. Although there are many solutions for one dimensional consolidation, the solutions which actually capture creep settlement are strictly limited. In their report Ladd et al. (1977) was asked, “Does creep act as a separate phenomenon while excess pore pressure dissipates during primary consolidation?” Following this, two different hypotheses known as Hypothesis A and Hypothesis B emerged; according to Hypothesis A, creep only occurs after primary consolidation, whilst hypothesis B assumes that some sort of structural viscosity is responsible for creep and it occurs during whole consolidation process (Jamiolkowski et al., 1985).

Existing consolidation theory cannot accurately predict the rate of settlement and pore water pressure for soil exhibiting creep settlement, especially when the surcharge is removed from the ground. Yin and Graham (1994) found that pore water pressure actually increases, even after loading, before decreasing towards zero, whilst Indraratna et al. (1994) noted un-dissipated pore water pressure under an embankment stabilised with vertical drains, which contradicts Terzaghi’s classical theory.
1.2 Application of vertical drain in ground improvement technique
Soft soil with very low permeability and high compressibility may take several years to reach ultimate settlement under certain preloading conditions due to the lengthy drainage path (depending on the depth of soil). However, vertical drains with an appropriate spacing significantly reduce the drainage path which speeds up the consolidation process. Reducing the drainage path leads to a dramatic increase in the dissipation of excess pore water pressure and the rate of consolidation. Figure 1.1 shows how vertical drains with a shorter drainage path enable soft soil to drain radially, unlike conventional consolidation, indeed prefabricated vertical drains have been successfully used and validated in several ground improvement projects around the world.

Figure 1.1 Shortening the drainage path using vertical drains to initiate radial consolidation
1.3 **Objectives of the study**
The main objectives of this study are summarised as follows:

1) Formulating an elastic visco-plastic (EVP) model of soft soil while considering non-Darcian flow, in order to accurately model the settlement and retarded pore water pressure of soft soil with a viscous skeleton.

2) Investigate the strain rate dependency of pre-consolidation pressure and the application of pre-consolidation pressure with a corresponding strain rate to model settlement and the dissipation of retarded pore water.

3) A laboratory investigation and comparison of the behaviour of a large scale undisturbed sample (350mm diameter) with small scale traditional Rowe cell specimen, while obtaining the properties of this soil.

4) *Class A and Class C* predictions of a Ballina embankment using an existing model and a newly developed EVP model.

5) A numerical validation of the laboratory results and application of models with different case histories.

1.4 **Organisation of chapters in this thesis**
This thesis consists of seven chapters; Chapter 1 herein introduced the scope of soft soil improvement with special reference to vertical drains and described the main objectives of the thesis.

Chapter 2 is a literature review of soft ground improvement, radial consolidation, and the time-dependent behaviour of viscous clay, as well as the use of vacuum preloading in ground improvement techniques.
Chapter 3 describes an analytical model of the elastic visco-plastic behaviour of soft clay using two different approaches: one with Bjerrum (1967) coupled with the Yin and Graham concept, while the other is based on the strain rate dependency of pre-consolidation pressure.

Chapter 4 describes the laboratory aspect of this study, and consists of Rowe cell testing (small scale specimens) and large scale consolidometer testing of an undisturbed specimen; it also presents the validation of the experimental results using FEM analysis and analytical models.

Chapter 5 describes the application of EVP model to three case histories for the purpose of validation; one from Ballina, one from the Second Bangkok International Airport (SBIA), and the Muar clay embankment in Malaysia.

Chapter 6 presents Class A and C predictions of the Ballina trial embankment (NFTF) with an existing and newly developed model, including any possible discrepancies of the model.

Chapter 7 concludes this study and makes recommendations for future research.
CHAPTER 2: LITERATURE REVIEW

2.1 Consolidation

2.1.1 Consolidation settlement

Consolidation is the process whereby the bulk volume of soil decreases due to the flow of pore water; according to Terzaghi (1943), consolidation is “any process which involves a decrease in the water content of a saturated soil without replacement by water or air”. On that basis consolidation settlement can be categorised as immediate settlement, primary consolidation settlement, and secondary compression settlement.

Immediate settlement – is due to the elastic deformation of dry, moist, and saturated soil without any change in the moisture content.

Primary Consolidation Settlement – is the result of a change in volume of saturated cohesive soil due to the expulsion of water from the void space.

Secondary compression settlement – is the plastic adjustment of soil fabric which follows primary consolidation settlement under constant effective stress.

2.1.2 Terzaghi’s 1D consolidation settlement

Terzaghi’s theory of one dimensional consolidation is used around the world to determine the compression and excess pore water pressure dissipation rates of soil with a low permeability; Terzaghi (1943) basic assumptions are as follows:

1) Soils are fully saturated and homogeneous, so the compressibility of soil and water is negligible.

2) The coefficient of permeability is assumed to be constant during consolidation.
3) There is a unique linear relationship between the vertical effective stress ($\sigma'$) and the void ratio ($e$) which is independent of the stress history and time.

4) Small strain theory and Darcy’s law are valid.

5) The flow of water is only in one direction (i.e., vertical).

Based on these assumptions, the following relationship can be determined from Terzaghi’s theory of one-dimensional consolidation.

\[ \frac{k_v}{\gamma_w} \frac{\partial^2 u'}{\partial z^2} = \frac{1}{1 + e} \frac{\partial e}{\partial t} \]  \hspace{1cm} (2.1)

Equation 2.1 is more generally written as: (Berry and Poskitt, 1972)

\[ \frac{\partial \varepsilon_v}{\partial t} = \frac{1 + e_0}{\gamma_w} \frac{\partial}{\partial z} (\frac{k_v}{1 + e} \frac{\partial u'}{\partial z}) \]  \hspace{1cm} (2.2)

\[ c_v \frac{\partial^2 u'}{\partial z^2} = \frac{\partial u'}{\partial t} \]  \hspace{1cm} (2.3)

Under appropriate boundary conditions, these equation lead to the final expression which is given by:

\[ U_v = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \exp^{-M^2 T_v} \]  \hspace{1cm} (2.4)

where $m$ is an Integer, \( M = \frac{\pi}{2} (2m + 1) \), $T_v$ is the time Factor for vertical drainage, $T_v = C_v t / H^2$, $C_v$ is the coefficient of consolidation, $t$ is elapsed time, and $H$ is the length of the drainage path.
2.1.3 Theory of vertical consolidation for variable compressibility and permeability

Lekha et al. (2003) modified Terzaghi’s classic theory by incorporating variations in compressibility and permeability using e-log $\sigma'_v$ and e-log k plot.

\[
U_v = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \exp^{-M^2 T_v^*}
\]

\[
T_v^* = 0.5 \left( 1 + \left( 1 + \frac{\Delta p}{\sigma'_1} \right)^{1-C_c/C_k} \right) T_v
\]

where $T_v^* = \text{modified time factor}$.

\[
\frac{\Delta p}{\sigma'_1} = \text{load increment ratio}.
\]

$\sigma'_1 = \text{initial effective stress}$.

$\Delta p = \text{applied preloading pressure}$.

Lekha et al., 2003 concluded that the $C_c/C_k$ and $\Delta p/\sigma'_1$ ratios govern the rate of consolidation, so when $C_c/C_k < 1$ consolidation occurs at higher than conventional rate, and this rate of consolidation increases as the load ratio increases. When $C_c/C_k > 1$, this process occurs at a slow rate, which then decreases with increments in the load ratio, and moreover, this equation resembles Terzaghi’s classic equation when $\frac{\Delta p}{\sigma'_1}$ is unity. Variations in the degree of consolidation with the time factor for different values of $\frac{\Delta p}{\sigma'_1}$ and $C_c/C_k$ are shown in Fig. 2.1.
2.1.4 Coefficient of vertical consolidation ($c_v$)

The linear relationship between the vertical effective stress ($\sigma'$) and void ratio (e), and the constant permeability throughout the consolidation process, are assumed in Terzaghi’s classic theory, which means the coefficient of consolidation which generally controls consolidation can be defined as:

$$c_v = \frac{k_v}{\gamma_w m_v}$$

(2.7)

where $m_v = \text{coefficient of volume change, } = \frac{\Delta \epsilon_v}{\Delta \sigma'_v}$
The value of $c_v$ can be calculated using the above mentioned equation (2.7) directly, by measuring $k_v$ and taking $m_v$ from the end of the primary consolidation (EOP) e Vs $\sigma'$ curve. It is better to consider $c_v$ as a curve fitting parameter rather than a fundamental parameter because it varies considerably with the position of the element and time (Leroueil, 1988).

Many researchers introduced different methods to estimate $c_v$; according to Bjerrum (1967), the typical $\varepsilon_v = \log t$ curve of a clay sample in an oedometer test has either an S-shape or a continuously increasing slope. With help from this S-shape curve, $c_v$ can be derived using the Casagranade method, but when this value is compared with $c_v$ obtained from the Taylor square root method (Taylor, 1948); it is generally smaller (Lambe and Whitman, 1979; Pelletier et al., 1979). Furthermore, Sridharan and Rao (1981) proposed the rectangular hyperbola fitting method to estimate $c_v$, which differs from that obtained from the Casagranade and Taylor methods. The magnitude of $c_v$ is generally higher in the recompression range than in the compression zone, but as the soil passes from the over-consolidated region to the normally consolidated region, there will be a change in $m_v$ and a sudden decrease of $c_v$ close to the preconsolidation pressure. (Terzaghi et al., 1996)

### 2.2 Theory of radial consolidation

#### 2.2.1 Barron’s theory

Based on the theory of consolidation proposed initially by Terzaghi (1943), Barron (1948) derived solutions for the radial consolidation problem facilitated by vertical drains. The assumptions made are as follows:

1) The soil is saturated, and therefore all vertical loads are initially carried by the excess pore water pressure
2) The applied load is assumed to be uniformly distributed and all the compressive strain within the soil occurs in a vertical direction.

3) The zone of influence of the drain is assumed to be circular.

4) The permeability of the drain is infinite compared to the soil.

5) Darcy’s law is valid, and

6) Small strain theory is applicable.

Based on these assumptions, two main cases, a) free strain, and b) equal strain were clearly described by Barron (1948). The three dimensional model of consolidation of radial drainage is given by:

\[
\frac{\partial \bar{u}}{\partial t} = \frac{C_p (\frac{\partial^2 u}{\partial z^2})}{C_h \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right)}
\]  

(2.8)

where \( t \) is the time elapsed after the load is applied, and \( u \) is the excess pore water pressure at radius \( r \) and at depth \( z \). For radial flow only, the above equation becomes:

\[
\frac{\partial \bar{u}}{\partial t} = \frac{C_h \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right)}
\]  

(2.9)

2.2.2 Theory of the approximate equal strain solution (Hansbo, 1981)

Based on the equal strain theory, Hansbo (1981) proposed an approximate solution for vertical drains which includes the smear effect and well resistance. The average degree of consolidation \( U_h \), of a soil cylinder with a vertical drain is given by,

\[
U_h = 1 - \exp\left( -\frac{8T_h}{\mu} \right)
\]

(2.10)

\[
\mu = \ln\left( \frac{n}{s} \right) + \left( \frac{k_h}{k_s} \right) \ln(s) - 0.75 + \pi z (2l - z) \frac{k_h}{q_w} \left\{ 1 - \frac{k_h/k_h'}{(k_h/k_h')(s/n)} \right\}
\]

(2.11)
Alternatively, $\mu$ can be expressed in a simplified form as:

$$
\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right)\ln(s) - 0.75 + \pi z (2l-z) \frac{k_h}{q_w}
$$

(2.12)

Ignoring well resistance, $\mu$ is given by,

$$
\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right)\ln(s) - 0.75
$$

(2.13)

Ignoring the smear effect, $\mu$ is given by,

$$
\mu \approx \ln(n) - 0.75 + \pi z (2l-z) \frac{k_h}{q_w}
$$

(2.14)

If the smear and well resistance are both ignored, the above parameter becomes,

$$
\mu = \ln(n) - 0.75
$$

(2.15)

2.2.3 Numerical modelling using plane strain consolidation

To carry out a multi-drain analysis of vertical drains means establishing an equivalence between the plane strain and axisymmetric conditions; this equivalence can be done in three different ways:

1) Geometric matching approach: Spacing between drains is matched by keeping the permeability constant.
2) Permeability matching approach: coefficient of permeability is matched by keeping the spacing between the drains constant.

3) Combination of permeability and geometric matching approach: plane strain permeability is calculated together with the change of drain spacing.

A vertical drain system (Fig 2.2) was converted into an equivalent parallel drain well by adjusting the permeability of the soil and by assuming the plane strain unit cell has a width of 2B (see Fig 2.3; Indraratna and Redana (1997)). The half width of the drains (bw) and the half width of the smear zone (bs) are matched to the axisymmetric radii rw and rs, respectively.

The average degree of consolidation (DOC) for plane strain proposed by Indraratna and Redana (1997) is given by:

\[
U_{h,ps} = 1 - \frac{\bar{u}}{\Delta P} = 1 - \exp\left( - \frac{8T_{hp}}{\mu_p} \right) \quad (2.16)
\]

where \( \bar{u}_0 \) = initial excess pore pressure, \( \bar{u} \) = average excess pore pressure at time t, \( T_{hp} \) = time factor in plane strain, and

\[
\mu_p = \left[ \alpha + (\beta) \frac{K_{h,ps}}{K'_{h,ps}} + (\theta)(2lz - z^2) \right] \quad (2.17)
\]

where \( K_{h,ps} \) and \( K'_{h,ps} \) are the undisturbed horizontal and relevant smear zone permeability, respectively. The geometric parameters \( \alpha \), \( \beta \) and the flow term \( \theta \) given by,

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

\[ \theta = \frac{2k_{s,ps}}{k_{s,ax} qz B} \left(1 - \frac{b_w}{B}\right) \]

The average degree of consolidation for both axisymmetric \((U_h)\) and equivalent plane strain \((U_{h,ps})\) conditions are made equal at a given stress level and at each time step, hence:

\[ U_h = U_{h,ps} \]

The time factor ratio \(\frac{T_{h,ps}}{T_h}\), can be given by the following equation:

\[ \frac{T_{h,ps}}{T_h} = \frac{k_{h,ps}}{k_h} \frac{R^2}{B^2} \tag{2.18} \]

Figure 2.2 Representation of a soil cylinder using a vertical drain (modified after Hansbo, 1997b)
2.3 Use of PVD in ground improvement

2.3.1 History of vertical drains

The use of vertical drains has been reported by Porter (1936) and Johnson (1970). They are used to accelerate consolidation by reducing the drainage path; this then reduces post construction settlement and increases the shear strength of relatively weak soil foundations. Vertical drains can be classified into three main types: a) prefabricated vertical drains, b) sand drains, and c) fabric encased sand drains. The types of drain and their sub-types are tabulated in Table 2.1.
Table 2.1 Classification of vertical drains (Rixner et al., 1986)

<table>
<thead>
<tr>
<th>Types</th>
<th>Sub-Types</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prefabricated vertical drain</td>
<td>Cardboard type drain</td>
<td>Full displacement of small volume</td>
</tr>
<tr>
<td></td>
<td>Plastic drain without jacket</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fabric covered</td>
<td></td>
</tr>
<tr>
<td>Sand drain</td>
<td>Closed end mandrel</td>
<td>Displacement-maximum</td>
</tr>
<tr>
<td></td>
<td>Continuous flight hollow stem auger</td>
<td>Displacement-limited</td>
</tr>
<tr>
<td></td>
<td>Screw type auger</td>
<td>Experience-limited</td>
</tr>
<tr>
<td></td>
<td>Rotary jet</td>
<td>Non-displacement</td>
</tr>
<tr>
<td></td>
<td>Dutch jet-bailer</td>
<td>Non-displacement</td>
</tr>
<tr>
<td></td>
<td>Internal jetting</td>
<td>Control measure-difficult</td>
</tr>
<tr>
<td>Fabric encased sand drain</td>
<td>Sand wick, Fabric and Pack drain</td>
<td>Full displacement of relatively small volume</td>
</tr>
</tbody>
</table>

The first use of cardboard wick drains was reported in Kjellman (1948) but the problem with them is that the top part decays, which inhibits its drainage capacity and then the dissipation rate of pore water pressure. Late in 1971, Geodrains used a plastic grooved core instead of cardboard, and then it became popular enough to use for commercial purposes (Bergado et al., 1996).

To meet the demand for rapid development and urbanisation in Southeast Asia region during the 1980’s, vertical drains came into general use as a ground improvement technique. In fact during that period Tominaga et al. (1979), Choa et al. (1979), Chou et al. (1980), Akagi (1981),
Balasubramaniam et al. (1980) and Woo et al. (1989) reported the use of sand drains, while Belloni et al. (1979), Choa et al. (1981), Nicholls (1981), Volders (1984), Lee et al. (1989), Bergado et al. (, 1988, 1990a, 1990b &, 1991) reported on the use of prefabricated vertical drains. In recent years, prefabricated vertical drains made from a corrugated plastic core and covered with a geosynthetic filter have become very popular and are frequently used to replace conventional sand drains. Various forms of drains whose properties differ from each other (i.e., their geometric shape, material, and filter) have now been introduced into today’s world; Alidrain, Mebradrain, Colbond, Desol, geodrains and flodrains are the common drains now found in commercial markets.

Circular drains are also available, which is very helpful when a vacuum pressure needs to be applied. In recent years, biodegradable drains made from environmentally friendly geosynthetics have recently been introduced; these drains are made from organic geotextiles (e.g. Jute acts like a filter and coconut coir act like a vertical drainage path).

2.3.2 Equivalent drain diameter

The theory of radial consolidation proposed by Barron (1948) assumed there is a circular drain in the derivation, which means the equivalent diameter of a band-shaped drain can be related to the diameter of a circular drain having the same theoretical radial drainage performance as a band shaped drain. Previous studies endeavoured to come up with an expression for the equivalent diameter; for instance, Kjellman (1948) suggests that the circumference of a band drain is more significant than the cross-sectional area for assessing the discharge capacity, while the finite element analysis by Rixner et al. (1986) and Hansbo (1987) suggest an equivalent diameter of band drain of width a and thickness b (as shown in Fig. 2.4) as:
Figure 2.4 Equivalent drain diameter of vertical drain

**Error! Bookmark not defined.**

\[ d_w = \frac{(a + b)}{2} \]  
(2.19)

Based on the flow nets around the drain influence zone, Pradhan et al. (1993) proposed an expression for the equivalent diameter of a vertical drain as given by:

\[ d_w = d_e - 2\sqrt{\bar{s}^2} + b \]  
(2.20)

\[ \bar{s}^2 = \frac{1}{4} d_e^2 + \frac{1}{12} a^2 - \frac{2a}{\pi^2} d_e \]  
(2.21)

Long and Covo (1994) used an electronic plotter to derive the following expression for a vertical drain with a rectangular cross-section:

\[ d_w = 0.5a + 0.7b \]  
(2.22)
2.3.3 Filters in prefabricated vertical drains

The first attempt to design a filter was made by Terzaghi (1929), although it is regarded as a conservative approach to design and was based on the following assumptions:

1) \( D_{15} \) of filter \( \geq 5 \times D_{15} \) of soil being protected

2) \( D_{15} \) of filter \( < 5 \times D_{85} \) of soil being protected

The criteria for wrapping a geotextile around a vertical drain to act as a filter is normally estimated using its retention, permeability, and resistance to clogging. Several guidelines can be used to determine these factors, for example, Carroll (1983) presented the criteria for assessing the apparent opening size of a geotextile filter as:

\[
\frac{O_{95}}{D_{95}} \leq (2 - 3) \tag{2.23}
\]

The retention ability in his study can be given by;

\[
\frac{O_{50}}{D_{50}} \leq (10 - 12) \tag{2.24}
\]

Holtz et al. (1991) proposed a ratio between the permeability of filter to the soil, which should be at least 10 as a basic guideline. The criteria to satisfy resistance to clogging proposed by Christopher and Holtz (1985) is given by;

\[
\frac{O_{95}}{D_{15}} \geq 3 \tag{2.25}
\]

\[
\frac{O_{15}}{D_{15}} = (2 - 3) \tag{2.26}
\]
2.3.4 Discharge capacity

The performance of a long vertical drain depends mainly on its discharge capacity; its discharge capacity depends on its cross-sectional area, the horizontal earth pressure, the kinking of the drain and the clogging phenomenon. Figure 2.5 shows the discharge capacity of different types of drains with varying confining pressure where, as the confining pressure increases, the discharge capacities decrease due to lateral pressure on the discharge capacity.

![Figure 2.5 Vertical discharge capacity with horizontal confining pressure. (modified after Rixner et al., 1986)](image_url)

2.3.5 Diameter of the influence zone

The time a given project needs to attain a certain degree of consolidation depends on the square of equivalent diameter of the soil cylinder ($d_e^2$). This parameter is a function of the spacing
between two drains and the pattern of installation; that is, drains can be installed in either a square pattern or a triangular pattern (see fig. 2.6). The diameter of the influence zone also plays an important role in an analytical solution of radial consolidation. A square pattern is normally preferred because it is easier to define the layout however, installing in a triangular pattern allows for a more uniform settlement between two individual drains. According to Hansbo (1981), the diameters of the influence zone for square and triangle patterns with spacing S are given by:

\[
D_e = 1.128S \quad \text{for square pattern} \\
D_e = 1.05S \quad \text{for triangular pattern}
\]
2.3.6 Techniques and equipment for installing drains

When thousands of approximately 15-20 m long drains (depending on the extent of soft soil) are to be installed in the field, they must be installed efficiently, which means having the proper equipment. Vertical drains are usually enclosed in a mandrel which is driven by a mechanical rig (commonly called a Stitcher) to the required depth, after which the mandrel is removed. The shoe is an anchor attached to the base of a vertical drain to align and ease the installation process. Rig selection also plays an vital role in the PVD installation process, which means the rig must be selected based on project requirements. A heavier rig might prove to be unstable during installation whereas a rig that is too light rig will be unable to deliver the power needed to drive the mandrel. A sand platform is also needed to stabilise the dead weight of the rig.

Rectangular, rhombic and circular mandrels are on the market but in practice circular mandrels are not very common (Bo et al., 2003). Figure 2.7 shows the machinery needed to install PVDs. A rhombic mandrel creates fewer disturbances in soil but it is prone to buckle due to high lateral pressure; a rectangular mandrel is very common in practice because it is good for stiffer ground, even though it causes much higher disturbances than a rhombic shaped mandrel. Moreover, the anchor selected should enable the mandrel to be extracted after the drain reaches its required depth, and the shoe attached to the mandrel must also prevent the ingress of soil into the drain during installation. Rectangular steel plates and steel bars are the most common types of anchors used while installing PVDs.
2.4 Soil disturbance due to the installation of vertical drains

When a pebble is thrown into a pond with water, the surrounding water is disturbed by the ripple effect; so too is soil disturbed when a vertical drain is installed with a mandrel. Virgin soil can be disturbed when a mandrel is pushed through a layer of soft clay by a mechanical rig. Penetration disturbs the surrounding soil, an effect known as the smear zone, within which the permeability and the rate of radial consolidation are greatly reduced. Unless the smear zone and its reduced permeability are known, it is very difficult to predict the settlement and pore water pressure during consolidation with any degree of accuracy. Barron (1948) and Hansbo (1979) defined two different zones: a smear zone and an undisturbed zone, whereas Onoue et al. (1991) & Rujikiatkamjorn et al. (2013) proposed three different zones based on the level of disturbance caused by drain installation. These zones are defined as, (1) the smear zone adjacent to the drain periphery where the permeability and compressibility of soil is greatly reduced, (2) the transition zone where the permeability and compressibility of soil is moderately reduced, and (3) an
undisturbed zone, where the permeability and compressibility is not affected by drain installation.

Several previous researchers proposed that the extent of the smear zone is based on aspects of permeability; some are based on assumption while others are based on laboratory investigations on different clay. A summary of the extent of the smear effect based on aspects of permeability are listed in Table 2.2.

Table 2.2 Smear extent

<table>
<thead>
<tr>
<th>Source</th>
<th>Extent of smear zone</th>
<th>Permeability aspects</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barron (1948)</td>
<td>( r_s = 1.6r_m )</td>
<td>( \frac{k_h}{k_s} = 3 )</td>
<td>Based on assumption</td>
</tr>
<tr>
<td>Hansbo (1979)</td>
<td>( r_s = 1.5\sim3r_m )</td>
<td>open</td>
<td>Based on literature</td>
</tr>
<tr>
<td>Hansbo (1981)</td>
<td>( r_s = 1.5r_m )</td>
<td>( \frac{k_h}{k_s} = 3 )</td>
<td>Based on assumption</td>
</tr>
<tr>
<td>Bergado et al. (1991)</td>
<td>( r_s = 2r_m )</td>
<td>( \frac{k_h}{k_v} = 1 )</td>
<td>Back analysis for Bangkok clay and experimental investigation</td>
</tr>
<tr>
<td>Onoue et al. (1991)</td>
<td>( r_s = 1.6r_m )</td>
<td>( \frac{k_h}{k_s} = 3 )</td>
<td>Based on test interpretation</td>
</tr>
<tr>
<td>Almeida and Ferreira (1993)</td>
<td>( r_s = 1.5\sim2r_m )</td>
<td>( \frac{k_h}{k_s} = 3\sim6 )</td>
<td>Based on authors experience</td>
</tr>
<tr>
<td>Indraratna and Redana (1998)</td>
<td>( r_s = 4\sim5r_m )</td>
<td>( \frac{k_h}{k_v} = 1.15 )</td>
<td>Based on laboratory investigation of Sydney clay</td>
</tr>
<tr>
<td>Author</td>
<td>Formula</td>
<td>Parameter</td>
<td>Scale Application</td>
</tr>
<tr>
<td>-------------------</td>
<td>---------</td>
<td>-----------</td>
<td>-------------------</td>
</tr>
</tbody>
</table>
| Chai and Miura  
(1999)           | $r_s = 2\sim3r_m$  
$k_h / k_s = \frac{1}{\sqrt[3]{k_s}}$  
$\bar{C}(r_h / r_s)$ | Based on lab and field values variation |
| Hird and Moseley  
(2000)           | $r_s = 1.6r_m$  
$k_h / k_s = 3$ | Design recommendation |
| Sharma and Xiao  
(2000)           | $r_s = 4r_m$  
$k_h / k_s = 1.3$ | Based on laboratory investigation for Kaolin clay |

2.5 **Ground improvement using vacuum preloading**

2.5.1 **Introduction**

Vacuum as a preloading technique was first introduced by Kjellman (1952), and since then it has been practiced in many part of the worlds. A list of projects where vacuum preloading is used to accelerate consolidation is tabulated in Table 2.3. Most projects applied 80 kPa (maximum) of vacuum pressure and if that proved to be inadequate, it can be used together with surcharge preloading. Vacuum preloading enables soil to reach its ultimate settlement earlier by accelerating the dissipation of pore water pressure, so it is mainly used in land reclamation projects where very soft clay dredged from sea bed cannot sustain additional surcharge loads and in areas where lateral deformation is constrained. It is also very applicable to projects where transporting fill material for constructing embankments is too expensive.
Table 2.3 Projects with vacuum preloading

<table>
<thead>
<tr>
<th>Country/Project Name</th>
<th>(Membrane/ Membrane less/)</th>
<th>Soil Type</th>
<th>Vacuum Pressure :kPa</th>
<th>Area of Influence:m²</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thailand/ Subarnabhumi International Airport (SBIA)</td>
<td>Membrane with surcharge</td>
<td>Soft Bangkok Clay</td>
<td>60</td>
<td>Two test embankment ; 1600 each</td>
<td>Bergado et al., 1998</td>
</tr>
<tr>
<td>Australia/ Ballina Bypass</td>
<td>Membrane</td>
<td>Clayey silt, silty clay</td>
<td>70-80</td>
<td>NR</td>
<td>Indraratna et al., 2010b</td>
</tr>
<tr>
<td>Australia/ Port of Brisbane</td>
<td>Membrane</td>
<td>Clay, sand, dredged mud</td>
<td>60-75</td>
<td>Two sections; 4305 and 10500.</td>
<td>Indraratna et al., 2011</td>
</tr>
<tr>
<td>China/Tianjin oil storage station</td>
<td>Membrane with surcharge</td>
<td>Soft clay</td>
<td>80</td>
<td>Two sections; 30000 and 20000</td>
<td>Chu et al., 2000</td>
</tr>
<tr>
<td>China/Tianjin Port storage yard</td>
<td>Membrane with surcharge</td>
<td>Silty clay , muddy clay</td>
<td>80</td>
<td>7433</td>
<td>Yan and Chu, 2005</td>
</tr>
<tr>
<td>China/Guangzhou Port</td>
<td>Membrane</td>
<td>Sandy silt, soft clay, silty clay and sandy clay, hydraulic fill</td>
<td>70</td>
<td>28200</td>
<td>Qiu et al., 2007</td>
</tr>
<tr>
<td>China/Road at Tianjin Port</td>
<td>Membrane</td>
<td>Silt, silty clay</td>
<td>80</td>
<td>18590</td>
<td>Yan and Chu, 2003</td>
</tr>
<tr>
<td>China/Tianjin New Harbour</td>
<td>Membrane</td>
<td>Silty clay</td>
<td>80</td>
<td>1250</td>
<td>Qian et al., 1992</td>
</tr>
<tr>
<td>China/Northeast New Railway</td>
<td></td>
<td>Peat, silty</td>
<td>93</td>
<td>1950</td>
<td>Lou, 1988</td>
</tr>
<tr>
<td>China/Factory at Lianyungang city</td>
<td></td>
<td>Marine clay</td>
<td>86</td>
<td>4000</td>
<td>Guan et al., 2011</td>
</tr>
<tr>
<td>China/land reclamation, Huizhou, Guangdong</td>
<td>Membrane</td>
<td>Gray silt, clayey soil, coarse sand, medium sand</td>
<td>85 initially but average 65-70 kPa</td>
<td>70552</td>
<td>Zheng et al., 2016</td>
</tr>
<tr>
<td>Srilanka/Southern Expressway</td>
<td>Membrane</td>
<td>Amorphous Peat</td>
<td>55 Initially but average 35</td>
<td></td>
<td>Ariyarathna et al., 2010</td>
</tr>
<tr>
<td>Japan/Reclaimed landfill with two test section</td>
<td>Membrane less</td>
<td>Reclaimed layer, clayey soil</td>
<td>80-90</td>
<td>3600 and 3782</td>
<td>Chai et al., 2010</td>
</tr>
<tr>
<td>Japan/ Reclaimed land project,</td>
<td>Membrane less</td>
<td>Reclaimed clayey layer,</td>
<td>65</td>
<td>11100</td>
<td>Chai et al., 2008</td>
</tr>
<tr>
<td>Location</td>
<td>Project Description</td>
<td>Membrane Type</td>
<td>Soil Compositions</td>
<td>Surcharge Method</td>
<td>Pressure</td>
</tr>
<tr>
<td>----------</td>
<td>--------------------------------------</td>
<td>---------------</td>
<td>-------------------</td>
<td>------------------</td>
<td>----------</td>
</tr>
<tr>
<td>Japan/ Test Embankment, Kushiro</td>
<td>Membrane with surcharge</td>
<td>Peat, clay, sand</td>
<td>60</td>
<td>1350</td>
<td>Tran and Mitachi, 2008</td>
</tr>
<tr>
<td>Japan/ Road construction, Saga</td>
<td>Membrane</td>
<td>Clay, silty clay, sandy clay, gravelly sand, fill</td>
<td>60-70</td>
<td>16796</td>
<td>Chai et al., 2006</td>
</tr>
<tr>
<td>Japan/ Embankment construction, Sapporo</td>
<td>Membrane with surcharge</td>
<td>Peat, clayey peat, clay, sand</td>
<td>60 /two vacuum pump</td>
<td>3200</td>
<td>Hayashi et al., 2002</td>
</tr>
<tr>
<td>Japan/ Embankment stabilisation, Ishikari</td>
<td>Membrane with surcharge</td>
<td>Peat, peaty clay, silty clay, silty sand, sand</td>
<td>60-70</td>
<td>20870 (divided into 13 sub units)</td>
<td>Shiona et al., 2001</td>
</tr>
<tr>
<td>France/ Leman tin Airport</td>
<td>Membrane with surcharge</td>
<td>Compressible alluvium</td>
<td>NR</td>
<td>NR</td>
<td>Cognon et al., 1994 &amp; Munfakh, 2003</td>
</tr>
<tr>
<td>France/ Oil depot, Ambes</td>
<td>Membrane with surcharge</td>
<td>Peat, silt, organic clay</td>
<td>NR</td>
<td>NR</td>
<td>Cognon et al., 1994</td>
</tr>
<tr>
<td>France/ Pilot test of highway emb</td>
<td>Membrane with surcharge</td>
<td>Peat, highly organic clay</td>
<td>150 (vacuum plus surcharge)</td>
<td>390</td>
<td>Cognon et al., 1994</td>
</tr>
<tr>
<td>Ireland/ Field test, Ballydermot raised bog</td>
<td>Membrane</td>
<td>Pseudo fibrous peat</td>
<td>80 initially, 71 sustained</td>
<td>100</td>
<td>Osorio-Salas, 2012</td>
</tr>
</tbody>
</table>
2.5.2 Systems of vacuum preloading

At this point in time there are two vacuum preloading systems which are classified on the basis of whether or not they contain a membrane. A membrane system has an airtight membrane placed over the sand drainage layer and its edges are submerged under the peripheral sealing trench (normally filled with bentonite slurry). After that, vacuum pumps are connected to the prefabricated drainage (PVD) system extending from those trenches via horizontal drains inside the sand blanket. The suction head thus generated enables rapid dissipation of pore water pressure in the soil using the principle of radial consolidation. This type of system allows for the propagation of a vacuum head within the sand blanket and down the PVDs, as well as along the surface of the soil within the whole airtight region, however, its efficiency depends entirely on whether the membrane prevents air leaks while a vacuum is being utilised. A typical system with a membrane is shown in Figure 2.8.

![Figure 2.8 Vacuum preloading system with membrane](image-url)
A system without a membrane is called a membrane less vacuum preloading system; it is generally used to prevent a vacuum loss through layers of soil that are close to the ground. Even cut off walls will not solve this problem, so a system of individual tubes are used to connect the vertical drains to the horizontal drainage network and then to the vacuum pump. The efficiency of this system depends on the individual drain and its connection, but it does require a lot of tubing for hundreds of drains and connections, and this can affect the cost and the installation time. A typical membrane less vacuum preloading system is shown in Fig. 2.9.

Figure 2.9 Membrane less vacuum preloading system

2.5.3 Basic principle of vacuum preloading

The total stress remains unchanged when vacuum preloading is applied to a layer of soft clay but the negative pressure from the vacuum pump reduces the pore water pressure by an amount that is equal to the negative pressure transferred to the clay; this also allows for an increase in effective stress as well as helping to stabilise the foundation. Qian et al. (1992) and Indraratna et al. (2005a) point out the main
difference between conventional surcharge preloading and vacuum preloading in Fig. 2.10, which can be summarised as follows:

1. Since consolidation with vacuum preloading is isotropic, the compressive lateral inward movement, especially at the toe of the embankment, should be monitored carefully so as to avoid any damage to the adjacent structures.

2. Using a vacuum within a PVD system enables the vacuum head to be propagated to a bigger depth of underlying subsoil.

3. For the same amount of settlement, the surcharge fill height can be decreased by applying a vacuum; this also helps to accelerate embankment construction and increase its stability.

4. The maximum value of excess pore water pressure (EPWP) generated using vacuum preloading is always less in compared to conventional surcharge preloading due a decrease in the surcharge load to the system.

5. If an unsaturated field condition arose (soil-drain interface) when a conventional surcharge method is being applied, the rate of consolidation decreases, but it can be increased using vacuum preloading and an increasing rate of consolidation.
2.5.4 Theory of vacuum consolidation and analytical methods

Mohamedelhassan and Shang (2002) modified a conventional consolidation apparatus to apply a vacuum pressure and then developed a one dimensional consolidation model with combined vacuum and surcharge. Using the principle of superposition between a vacuum alone 1-D equation and a surcharge alone 1-D equation, a combined model was derived. The average degree of consolidation for this kind of model can be expressed by,

\[ U_{vc} = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \exp^{-M^2T_{vc}} \]  

(2.30)
where $T_{vc} = C_{vc} t / H^2$ is a time factor for combined vacuum and surcharge preloading.

Figure 2.11 is a schematic diagram for vacuum and surcharge preloading, and for surcharge preloading alone and vacuum preloading alone. The major finding of this research is that a vacuum pressure has a similar effect as surcharge preloading alone, and the rate that pore water pressure is dissipated increases due to the combined vacuum and surcharge preloading.

Figure 2.11 Schematic diagram of a vacuum preloading system showing, (a) combined vacuum and surcharge, (b) surcharge alone, and (c) vacuum alone (after Mohamedelhassan and Shang, 2002)
Due to the rapid use of vacuum preloading technology over the past years, a novel and simple model for analysing vacuum preloading was essential. A comprehensive mathematical solution for vacuum preloading with vertical drain proposed by Indraratna et al. (2005c) assumed an equal strain hypothesis (Barron, 1948) and a trapezoidal distribution of vacuum pressure along the length of the vertical drain (see Fig. 2.12). This model presents a mathematical expression in axisymmetric and plane strain conditions, as well as verifying the model with different case histories from around the world.

The average pore water pressure ratio obtained from this analytical model under axisymmetric conditions is given by:

\[
R_u = \left(1 + \frac{(1 + k_1) p_0}{u_0}\right) \exp \left(\frac{-8 k_h}{\mu} \right) - \frac{p_0}{u_0} \frac{(1 + k_1)}{2} \quad (2.31)
\]

and

\[
\mu = \ln \left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right) \ln(s) - \frac{3}{4} + \pi z (2l - z) \frac{k_h}{q_w} \left\{1 - \frac{k_h}{k_h'} - 1\right\} \quad (2.32)
\]

Figure 2.12 Distribution of vacuum pressure along the drain under, (a) Axisymmetric, and (b) Plane strain conditions (after Indraratna et al., 2005c)
where $P_0$ is the vacuum pressure applied at the top, $k_1$ is the factor for the loss of vacuum along the length, $k_h$ & $k_s$ are the horizontal permeability of soil in the undisturbed and smeared zones, respectively and $\bar{u}_0$ is the average initial excess pore water pressure. The value of $\mu$ in Eq. (2.32) can be further simplified as:

$$\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right)\ln(s) - \frac{3}{4} + \pi z(2l - z) \frac{k_h}{q_w}$$

(2.33)

Similarly, the above expressions for a plane strain model can be summarised as;

$$R_{up} = \left(1 + \frac{(1 + k_1)p_{0p}}{2\bar{u}_0}\right) e^{\left(-\frac{8T_{hp}}{\mu p}\right)} - \frac{p_{0p}(1 + k_1)}{\bar{u}_0}$$

(2.34)

and

$$\mu_p = \left[\alpha + \frac{k_{hp}}{k_{sp}}(\beta) + (\theta)(2lz - z^2)\right]$$

(2.35)

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

(2.36)

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

(2.37)

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

(2.38)

2.6 Viscous behaviour of soft clay

2.6.1 Time dependent behaviour of clays

A study of the time-dependent behaviour of soft clay includes the effects of creep, stress relaxation, and the strain rate, effects that are evaluated on the basis of a
standard test (i.e. creep tests, stress relaxation tests, and constant rate of strain tests), respectively.

2.6.2 Hypotheses A and B

Until now, it has been very difficult to mimic in-situ conditions by conducting experiments on small scale specimens; indeed controversies from several researchers are still a matter of debate. However, two hypotheses (A & B) have been proposed (as shown in Fig. 2.13) and an explanation of the behaviour of soil has been defined either with these two hypotheses, or between them and Aboshi (1973).

![Figure 2.13 Hypotheses A and B (modified after Ladd, 1973)](image)

Hypothesis A assumes there is no relationship between the end of primary consolidation (EOP) and the pre-consolidation pressure, and the effective stress is
independent of the thickness of the sample used. Therefore, the strain which corresponds to EOP is unique and equal for the laboratory and in-situ sample, and there is no time-dependent behaviour during the primary consolidation phase, it begins only when the pore water pressure has dissipated. Ladd (1973); Ladd et al. (1977); Mesri and Godlewski (1977); Mesri and Choi (1985a) & Mesri and Rokshar (1974) support the statement in line with Hypothesis A. In a similar way, Hypothesis B assumes that strain at End of primary consolidation (EOP) is not unique to the small laboratory specimens and large in-situ specimens. In other words, the strain at EOP is a function of the thickness of the sample and time-dependent behaviour due to the viscous skeleton of soil because primary consolidation continues throughout the whole consolidation process. (Suklje, 1957, Bjerrum, 1967, Yin and Graham, 1989b and Degago et al., 2011)

2.6.3 Delayed consolidation

It is impossible to demonstrate the compressibility and secondary compression of clay using a single e-logσ′ curve because this process takes time, so to overcome this issue, a series of lines or curves (as shown in Fig. 2.14) known as “time lines” have been established to represent the equivalent void ratio for a given effective stress at a time of sustained loading; each time line has a different pre-consolidation pressure.
Figure 2.15 shows the relationship between the effective stress, time, and void ratio, and suggests that for a given effective stress and void ratio, an equivalent time of sustained loading and a certain rate of delayed compression exists, which is independent of the path the clay had taken to reach these values. The compression part can be clearly divided into two parts: (a) Instant compression which occurred after increasing the effective stress, and (b) delayed compression which occurred at constant loading (i.e. effective stress). These terms are clearly explained with the help of Fig 2.15.
2.6.4 Effect of strain rate in consolidation

The strain rate has a primary role when calculating the settlement and dissipation of pore water pressure because in small scale laboratory specimens the strain rate is too small, unlike in the field. The dependency of the strain rate to effective stress was formulated by Leroueil et al. (1985b) using a rheological model, and can be defined using the following equation:

$$\sigma_p' = f(\epsilon_v')$$

(2.29)

$$\frac{\sigma_v'}{\sigma_p'} = g(\epsilon_v')$$

(2.30)
These two equations can be combined to obtain a rheological model which will represent the stress-strain-strain rate relationship which can be expressed as:

$$\dot{\varepsilon}_v = f^{-1}\left(\frac{\sigma_{v'}}{g(\varepsilon_v)}\right)$$  \hspace{1cm} (2.31)

Figure 2.16 shows the individual relationship between the strain rate versus pre consolidation pressure, as well as the vertical strain versus normalised effective stress, as given by Eqs. (2.30) and (2.31), respectively. These two plots can help to determine the stress-strain-strain rate relationship, which is very beneficial in practice.
Figure 2.16 Effect of the strain rate with an increase in effective stress (modified after Leroueil et al., 1985b)
2.6.5 Elastic visco-plastic modelling of soft clay

Several models have been developed to investigate the time-dependent behaviour of clays, most of which use curve fitting techniques based on laboratory data (Mesri and Godlewski, 1977, Leroueil et al., 1985b and Leroueil, 1988). These models cannot capture phenomenon such as relaxation and multistage loading, so Yin and Graham (1989a), Yin and Graham (1989b) and Yin and Graham (1994) developed a one-dimensional time-dependent model based on the concept of time-equivalence. Further models by Yin and Graham (1996), Yin and Zhu (1999), Yin and Graham (1999) and Yin et al. (2002) also provided a detailed study of the elastic visco-plastic behaviour of soft clay. All of these models are based on the Bjerrum time-equivalent concept and the strain rate approach. The concept of equivalent time in Yin and Graham’s model is used to model the creep phenomenon of normal and over-consolidated clays using parameters such as $\sigma_z', \dot{\sigma}_z', \varepsilon_z$ and $\dot{\varepsilon}_z$. This model also enables normal and over-consolidated clays to be distinguished, and can also model time-relaxation behaviour. The Yin and Graham models contain four primary parameters which can be described as:

Equivalent time ($t_e$): several series of time lines having an equal value of equivalent time have been proposed by Yin and Graham (1989a), but this does not necessarily mean they have equal real time durations. For normally consolidated clay the duration of the load increments and equivalent time are equal, but the values for over-consolidated clay are different due to the OCR effect. Here, the unique strain rate is represented by the equivalent time and a higher value of equivalent time indicates smaller creep strain rates. The value of equivalent time is equal to zero for certain vertical strains under a constant vertical stress.
Reference time line (\(\lambda\)-line): as the time line used to calculate the equivalent time is called the reference time line; it is referred to as reference when the value of the equivalent time is zero. The bounds for reference time lines are between \(0 < t_e < \infty\) and \(-t_0 < t_e < 0\).

Instant time line (\(\kappa\)-line): the instant time line captures the elastic response of a soil skeleton due to a change in the effective stress in the soil. These time lines are assumed to be perfectly elastic and are used to calculate instantaneous strains.

Limit time line: a limit time line can be defined as time lines when the value of an equivalent time reaches infinity and with a zero corresponding creep rate.

Figure 2.17 Definition of various parameters used in Yin and Graham’s model (modified after Yin and Graham, 1994)
All the parameters needed to calculate the settlement and pore water pressure using the elastic viscoplastic models are shown in Fig. 2.17; based on these parameters, Yin and Graham proposed an EVP model which is given by:

\[
\dot{\varepsilon}_z = \kappa \frac{1}{\nu \sigma_z' t_0} \sigma_z' + \frac{\psi}{\nu_t_0} \exp \left[ -(\varepsilon_z - \varepsilon_{z0}) \frac{\nu}{\psi} \left( \frac{\sigma_z'}{\sigma_{z0}'} \right) \right]
\]  

(2.32)

where \( \kappa \) is the material properties (instant time line), \( \psi \) is the creep parameter, \( \lambda \) is the slope of the reference time line, \( \varepsilon_{z0} \) is the strain corresponding to initial effective stress \( \sigma_{z0}' \) and \( \nu \) is the specific volume.

2.7 Use of flow relationship in consolidation model

There are several consolidation models where the relationship between the flow of pore water pressure and the hydraulic gradient is considered as both linear and non-linear. Hansbo (1960) first proposed a radial consolidation model with a non-linear flow relationship, after which most analytical methods developed till now are based on Darcy’s law, which might not be true for all types of soils, especially fine grained soils under a very low hydraulic gradient (Hansbo, 2001, 1960). However, all of these models, despite using Darcian or non-Darcian flow in their formulation, rely completely on the procedure for determining the value of permeability and the coefficient of horizontal consolidation. Due to the phenomenon of grain migration, variations in the hydraulic gradient, grain reorientation, flow channel clogging, alterations in the average viscosity, fluctuations in temperature and seepage due to consolidation, there might be some unique relationship between the flow velocity and gradient that must be established in order to predict accurate flow characteristics.

The formulation proposed by Hansbo (1960) is a combination of the non-linear and
linear parts where the non-linear part is mainly for a low hydraulic gradient and the linear part is for a high hydraulic gradient. The relationship between the pore water flow and hydraulic gradient, as per Hansbo (1960), is shown in Fig 2.18.

\[ v = \kappa i^m \quad \text{when } i \leq i_l \]  

\[ v = k(i - i_0) \quad \text{when } i \geq i_l \]  

\[ i_0 = i_l(m - 1)/m \]  

\[ k = m\kappa i_l^{(m-1)} \]

Kianfar et al. (2013) carried out a Rowe cell experiment on remoulded kaolin clay and established a power law relationship between the pore water flow and velocity gradient. This relationship between the flow velocity \( v \) and hydraulic gradient \( i \)
was plotted for the different vacuum surcharge ratio (VSR); it follows the power law rather than a combination of the non-linear and linear flow velocity relationship. Based on a laboratory investigation of completely remoulded Kaolin clay, a relationship like the following equation was proposed, and the plot is shown in Fig. 2.19.

$$v = \alpha_c i^\beta$$

(2.37)

Where $\alpha_c$ and $\beta$ are two constants, the value of which for remoulded kaolin clay is $3.2 \times 10^{-10} \text{ m/s}$ and 1.3, respectively.

![Figure 2.19 Non-Darcian flow model for remoulded kaolin clay (modified after Kianfar et al., 2013)]
Furthermore, Hansbo (1960) developed a radial consolidation model that captures non-Darcian flow and explained that the hydraulic gradient in the field is much lower than the hydraulic gradient in a small scale laboratory test on radial consolidation. The relationship between the field and laboratory conditions based on hydraulic gradient can be determined by:

\[
(i)_{\text{Lab}} = \frac{D_{\text{field}}}{D_{\text{Lab}}} (i)_{\text{field}}
\]  

(2.38)

Where \( i \) is the hydraulic gradient and \( D \) is the influence zone diameter. The testing on large scale consolidometer of 350 mm diameter (undisturbed sample) has been conducted so as to minimise the error in terms of hydraulic gradient over the 50 mm small scale remoulded sample. In other words, use of 350 mm specimen will decrease the discrepancy by reducing field to laboratory influence diameter ratio.

### 2.8 Summary

The use of vertical drains to allow for radial consolidation is regarded as the most efficient ground improvement technique because the drain is used to shorten the drainage path and accelerate consolidation. The installation method, filter criteria, and discharge capacity, as well as the types of drain, play a significant role in defining the efficiency of vertical drains, therefore it is imperative to select the type of vertical drain best suited for the particular ground improvement project, whilst maintaining adequate spacing between them and an installation method with fewer disturbances. The use of a vacuum tandem with PVD and a negative pore pressure developed from suction can further accelerate consolidation, as has been shown in several projects around the world.
Several analytical models have also been proposed by previous researchers to predict settlement and the dissipation of excess pore water pressure, but they still cannot capture the viscous behaviour of soft soil. There are two primary hypotheses, Hypothesis “A” and “B” which explain the creep behaviour of soft soil. Hypothesis A assumes that creep occurs only after primary consolidation whereas hypothesis B assumes some sort of structural viscosity is responsible for the creep that occurs during consolidation. However, most of the analytical models developed with vertical drains and vacuum preloading are more in line with Hypothesis A.

A small scale laboratory specimen with a higher strain rate can finish consolidation in several hours, whereas consolidation in the field takes a long time. This disparity is due to variations in the strain rate between the laboratory and the field. There is a unique relationship for individual soil between the strain rate and pre-consolidation pressure which is called the strain rate dependency of pre-consolidation pressure. In order to accurately predict the dissipation of excess pore water and a settlement graph, converting the laboratory pre-consolidation pressure into the field is essentially based on the strain rate.
CHAPTER 3: ANALYTICAL MODELLING

3.1 Introduction

The use of vertical drains on soft soil saves a huge amount of time as well as money because they accelerate the dissipation of pore water and reduce time needed to reach post consolidation settlement. This dissipation of excess pore water pressure (EPWP) and the rate of settlement are heavily influenced by the visco-plastic behaviour of soft clay despite the lack of an appropriate elastic visco-plastic model for radial consolidation which can describe the real behaviour of clay. This chapter describes two analytical models to describe the visco-plastic behaviour of soft clay. The first model deals with the elastic visco-plastic behaviour of soft soil into a consolidation equation derived for the \( r \) and \( z \) directions facilitated with a vertical drain; it also considers the non-Darcian flow of fluid parameters during radial consolidation. An alternating direction implicit (ADI) finite difference (FD) method called the Peaceman-Rachford (P-R) method is applied to solve the derived complex partial differential equation and also to calculate the settlement and excess pore water pressure dissipation rate. The second analytical model mainly deals with the isotache concept for radial consolidation to model settlement and excess pore water pressure considering the strain rate dependency of pre-consolidation pressure. The details of both above mentioned models are described herein:

3.2 Elastic visco-plastic (EVP) model with special reference to radial consolidation considering non-Darcian flow using the FD method

3.2.1 Assumptions

The governing equation for radial consolidation with elastic visco-plastic behaviour was derived based on non-Darcian flow of fluid and visco-plastic constitutive relationship by Yin and Graham (1989a) by capturing the Bjerrum time equivalent
constant. The finite difference (FD) technique is then applied to solve the complex non-linear partial differential equation; the following assumptions are made:

1) The soil is fully saturated and homogeneous.
2) The applied load is uniformly distributed and all the compressive strain within the soil element occurs in the radial and vertical directions.
3) The zone of influence of the drain is assumed to be circular and axisymmetric.
4) The permeability of the drain is infinite compared to the soil.
5) The flow of pore water through the soil follows the non-Darcian flow law.

3.2.2 Derivation

A differential soil element around a vertical drain in a cylindrical co-ordinate system (as shown in Fig. 3.1) is considered where the movement of pore water is only allowed in the radial and vertical directions (r- and z- directions), and there is no flow in the Ω-direction. In order to satisfy the flow continuity equation, the rate of volume change must be equal to the net flow rate, where ∂V/∂t is given by,
Figure 3.1 Co-ordinates in the $r$ and $z$-directions

\[
\frac{\partial V}{\partial t} = \Delta q = \frac{\partial q_r}{\partial r} dr + \frac{\partial q_z}{\partial z} dz
\]  

(3.1)

\[q_r = v_r(rd\Omega dz)\]  

(3.2)

\[q_z = v_z(rd\Omega dr)\]  

(3.3)

where $q_r$ and $q_z$ are the net flow rate in the $r$ and $z$ directions and $v_r$ and $v_z$ are the flow velocities in the $r$- and $z$ directions.

The incremental volume change of the soil element $\frac{\partial V}{\partial t}$ is given by,

\[
\frac{\partial V}{\partial t} = -(\varepsilon_r + \varepsilon_\Omega + \varepsilon_z)(rd\Omega drdz)
\]  

(3.4)

where $\varepsilon_r$, $\varepsilon_\Omega$, and $\varepsilon_z$ are strain of the element in the $r$-, $\Omega$- and $z$-direction, respectively.

Substituting (3.2), (3.3) and (3.4) into Eq. (3.1),

\[
\frac{\partial \varepsilon_v}{\partial t} + \frac{\partial v_r}{\partial r} + \frac{v_r}{r} + \frac{\partial v_z}{\partial z} = 0
\]  

(3.5)
The non-Darcian flow relationship proposed by Hansbo (1960) consists of exponential plus linear parts, but for simplicity, the power law for the curve is used in this analysis and is given by:

\[ v = \alpha_c i^\beta \]  \hspace{1cm} (3.6)

where \( v \) = the flow velocity, 
\( i \) = the hydraulic gradient and, 
\( \alpha_c \) and \( \beta \) = flow constants depending on the type of soil. The \( r \)- and \( z \)- components of velocity are given below:

\[ v_r = \alpha_c |i_r|^\beta = \alpha_c \frac{1}{\gamma_w} \frac{\partial u}{\partial r} |^\beta \]  \hspace{1cm} (3.7)

\[ v_z = \alpha_c |i_z|^\beta = \alpha_c \frac{1}{\gamma_w} \frac{\partial u}{\partial z} |^\beta \]  \hspace{1cm} (3.8)

Substituting (3.7) and (3.8) into (3.5),

\[ \frac{\partial \varepsilon_v}{\partial t} + \alpha_c \frac{\partial}{\partial r} |\frac{1}{\gamma_w} \frac{\partial u}{\partial r}|^\beta + \alpha_c \frac{1}{\gamma_w} \frac{\partial u}{\partial r}|^\beta + \alpha_c \frac{\partial}{\partial z} |\frac{1}{\gamma_w} \frac{\partial u}{\partial z}|^\beta = 0 \]

\[ \frac{\partial \varepsilon_v}{\partial t} + \frac{1}{\gamma_w} \left[ \alpha_c \frac{\partial}{\partial r} |\frac{\partial u}{\partial r}|^\beta + \alpha_c \frac{\partial}{\partial r}|^\beta + \alpha_c \frac{\partial}{\partial z} |\frac{\partial u}{\partial z}|^\beta \right] = 0 \]

\[ \frac{\partial \varepsilon_v}{\partial t} + \frac{\alpha_c \beta}{\gamma_w} \left[ |\frac{\partial u}{\partial r}|^{\beta-1} \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) + |\frac{\partial u}{\partial z}|^{\beta-1} \frac{\partial^2 u}{\partial z^2} \right] = 0 \]  \hspace{1cm} (3.9)

Eq (3.9) is governing equation for radial consolidation.

According to the Yin-Graham EVP model (Yin and Graham, 1989b), strain at any given effective stress is:

\[ \varepsilon_z = \varepsilon_{z0} + \frac{\lambda}{V} \ln \frac{\sigma_x'}{\sigma_{z0}'} + \frac{\psi}{V} \ln \left( \frac{t_0 + t_e}{t_0} \right) \]  \hspace{1cm} (3.10)
where $\epsilon_{z0}^{ep}$ is the strain at the reference point, $(\psi/V)$ is determined from the slope of creep strain plotted against $\ln(t_e)$ and $t_e$ is equivalent time.

From equation (3.10), the equivalent time can be calculated by:

$$t_e = -t_0 + t_0 \exp \left[ (\epsilon_z - \epsilon_{z0}^{ep}) \frac{V}{\psi} \left( \frac{\sigma_z'}{\sigma_{z0}'} \right) \right]$$  \hspace{1cm} (3.11)

According to the creep model given by (Bjerrum, 1967; see fig. 3.2):

![Diagram of Bjerrum time equivalent concept](after Bjerrum, 1967)

Figure 3.2 Bjerrum time equivalent concept (after Bjerrum, 1967)

The incremental strain rate $\frac{\partial \epsilon_z}{\partial t}$ based on this figure can be written as:

$$\frac{\partial \epsilon_z}{\partial t} = \frac{\kappa}{V \sigma_z'} \frac{\partial \sigma_z'}{\partial t} + \frac{\psi}{\nu \ t_0 + t_e} \frac{1}{\nu \ t_0 + t_e}$$  \hspace{1cm} (3.12)

Substituting Eq. (3.11) into Eq.(3.12), the elastic visco-plastic model can be obtained as:
Equation (3.13) can be rewritten in terms of excess pore pressure by:

\[
\frac{\partial \varepsilon_z}{\partial t} = \frac{\kappa}{V \sigma_z} \frac{\partial (\sigma_z - u)}{\partial t} + \frac{\psi}{t_0 V} \exp \left[ -(\varepsilon_z - \varepsilon_{z0}^{ep}) \frac{V}{\psi} \frac{\left( \frac{\sigma_x}{\sigma_{x0}} \right)^{\lambda}}{\psi} \right] \tag{3.14}
\]

where \(\sigma_z\) is the total vertical stress.

If the total vertical stress is constant and \(\varepsilon_{z0}^{ep}\) is assumed to be zero, Equation (3.14) becomes:

\[
\frac{\partial \varepsilon_z}{\partial t} = -m_v \frac{\partial u}{\partial t} + g(u, \varepsilon_z) \tag{3.15}
\]

In the above expression,

\[
m_v = \frac{\kappa}{V (\sigma - u)}
\]

\[
g(u, \varepsilon_z) = \frac{\psi}{t_0 V} \exp \left[ -(\varepsilon_z) \frac{V}{\psi} \left( \frac{\sigma_x - u}{\sigma_{x0}} \right)^{\lambda} \right]
\]

Combining (9) and (15), we get:

\[
\frac{\partial \varepsilon_z}{\partial t} + \frac{\alpha_c \beta}{\gamma_w^{\beta}} \left( \frac{\partial u}{\partial r} \right)^{\beta-1} \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) + \left| \frac{\partial u}{\partial z} \right|^{\beta-1} \frac{\partial^2 u}{\partial z^2} = 0
\]

\[
m_v \frac{\partial u}{\partial t} - g(u, \varepsilon_z) = \frac{\alpha_c \beta}{\gamma_w^{\beta}} \left( \frac{\partial u}{\partial r} \right)^{\beta-1} \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) + \left| \frac{\partial u}{\partial z} \right|^{\beta-1} \frac{\partial^2 u}{\partial z^2} \tag{3.16}
\]

3.2.3 FD solution of axisymmetric equation by Peaceman-Rachford (P-R) ADI approach

The Peaceman-Rachford alternating direction implicit (ADI) (Murray and Lynn, 1965) is a two-step finite difference method applied to solve equation (3.16). In the
first step, applying the implicit differentiation in the $r$ direction and explicit differentiation in the $z$ direction yielded the predictor, secondly, applying the implicit differentiation in the $z$ direction and explicit differentiation in the $r$ direction resulted the corrector. An intermediate time step $t+\Delta t/2$, was defined for the predictor and corrector, and it lies in between $t$ and $t+\Delta t$ (Fig. 3.3a).

The finite difference grid for this solution is shown in Fig. 3.3b, where $i$ is the variable along x-direction representing $r$ coordinates which varies from $i-1$, $i$ and $i+1$. Similarly, $j$ is the variable along the y-direction representing $z$ coordinates which vary from $j-1$, $j$, and $j+1$. The pore water pressure will be calculated at each node of the grid using the P-R ADI scheme.

To simplify Eq (3.16), we define $\left( \frac{\partial u}{\partial r} \right)^{\beta-1}$ and $\left( \frac{\partial u}{\partial z} \right)^{\beta-1}$ at the last time step and continue the iteration process along with the Peaceman-Rachford ADI scheme.

$$\omega_{i,j}^t = \left( \frac{u_{i+1,j}^t - u_{i,j}^t}{2\Delta r} \right)^{\beta-1} \quad (3.17)$$

$$\varphi_{i,j}^t = \left( \frac{u_{i,j+1}^t - u_{i,j}^t}{2\Delta z} \right)^{\beta-1} \quad (3.18)$$
Equation (3.16) derived to incorporate the elastic visco-plastic behaviour for soft soil consists of a linear and non-linear part of \( \frac{\partial u}{\partial r} \) and \( \frac{\partial u}{\partial z} \). This combination of a linear and non-linear part in a partial differential equation makes it impossible to solve the equation in one step so the method of iteration for the non-linear part has been applied for the portion consisting non-linear flow in order to solve the equation correctly.

The form of equation that should undergo FD-ADI scheme is;

\[
\frac{\partial u}{\partial t} = \frac{\alpha \beta}{\gamma_w \beta m_v} \left[ \frac{\partial u}{\partial r} \right]^{\beta-1} \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) + \frac{|| \partial u }{\partial z ||} \left( \beta-1 \right) \frac{\partial^2 u}{\partial z^2} \right] + \frac{1}{m_v} g(u, \varepsilon_z) \tag{3.19}
\]

Applying Finite difference techniques on Eq. (3.19):

\[
\frac{u_{i,j}^{t+\Delta t} - u_{i,j}^t}{\Delta t} = \frac{\alpha \beta}{\gamma_w \beta m_v} \left[ \omega_{i,j}^{t} \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) + \right] + \frac{\phi_{i,j}^{t} \beta}{m_v} \frac{\partial^2 u}{\partial z^2} \right] + \frac{1}{m_v} g(u, \varepsilon_z) \tag{3.20}
\]
A simple explicit finite difference differentiation on the right hand side (R.H.S.) of Eq. (3.20) yields,

\[
\frac{u_{i,j}^{t+\Delta t} - u_{i,j}^{t}}{\Delta t} = \frac{\alpha_c \beta \gamma}{\nu_m} \left[ \omega_{i,j} \left\{ \frac{\partial_r^2 u_{r}^{t+\Delta t} + \partial_r^2 u_{r}^{t}}{2(\Delta r)^2} + \frac{1}{r} \frac{\partial_r u_{r}^{t+\Delta t} + \partial_r u_{r}^{t}}{2(\Delta r)} \right\} \right] \\
+ \frac{\varphi_{i,j}^t}{\nu_m} \frac{\partial_z^2 u_{z}^{t+\Delta t}}{2(\Delta z)^2} + \frac{1}{m_v} g(u, \varepsilon_z)
\]

A simple implicit finite difference differentiation on the right hand side (R.H.S) of Eq. (3.20) yields,

\[
\frac{u_{i,j}^{t+\Delta t} - u_{i,j}^{t}}{\Delta t} = \frac{\alpha_c \beta \gamma}{\nu_m} \left[ \omega_{i,j} \left\{ \frac{\partial_r^2 u_{r}^{t+\Delta t} + \partial_r^2 u_{r}^{t}}{2(\Delta r)^2} + \frac{1}{r} \frac{\partial_r u_{r}^{t+\Delta t} + \partial_r u_{r}^{t}}{2(\Delta r)} \right\} \right] \\
+ \frac{\varphi_{i,j}^t}{\nu_m} \frac{\partial_z^2 u_{z}^{t+\Delta t}}{2(\Delta z)^2} + \frac{1}{m_v} g(u, \varepsilon_z)
\]

The average of simple Implicit and simple explicit function on R.H.S. yields the Crank-Nicholson ADI scheme:

\[
\frac{u_{i,j}^{t+\Delta t} - u_{i,j}^{t}}{\Delta t} = \frac{\alpha_c \beta \omega_{i,j}^{t+\Delta t}}{\nu_m} \left[ \frac{\partial_r^2 u_{r}^{t+\Delta t} + \partial_r^2 u_{r}^{t}}{2(\Delta r)^2} + \frac{1}{r} \frac{\partial_r u_{r}^{t+\Delta t} + \partial_r u_{r}^{t}}{2(\Delta r)} \right] \\
+ \frac{\alpha_c \beta \varphi_{i,j}^{t+\Delta t}}{\nu_m} \left[ \frac{\partial_z^2 u_{z}^{t+\Delta t} + \partial_z^2 u_{z}^{t}}{2(\Delta z)^2} \right] + \frac{1}{m_v} g(u, \varepsilon_z)
\]

\[
\frac{u_{i,j}^{t+\Delta t} - u_{i,j}^{t}}{\Delta t} = \frac{\alpha_c \beta \omega_{i,j}^{t+\Delta t}}{\nu_m} \left[ \frac{\partial_r^2 u_{r}^{t+\Delta t} + \partial_r^2 u_{r}^{t}}{2(\Delta r)^2} + \frac{1}{r} \frac{\partial_r u_{r}^{t+\Delta t} + \partial_r u_{r}^{t}}{2(\Delta r)} \right] \\
+ \frac{\alpha_c \beta \varphi_{i,j}^{t+\Delta t}}{\nu_m} \left[ \frac{\partial_z^2 u_{z}^{t+\Delta t} + \partial_z^2 u_{z}^{t}}{2(\Delta z)^2} \right] + \frac{\Delta T_r}{m_v} g(u, \varepsilon_z)
\]

\[
u_m = \frac{\alpha_c \beta \omega_{i,j}^{t+\Delta t}}{2 \nu_m (\Delta r)^2}
\]
\[
\Delta T_v = \frac{\alpha c \beta \phi_{i,j} \Delta t}{2 \gamma_w \beta m_v (\Delta z)^2}
\]

\[
u_{i,j}^{t+\Delta t} - \Delta T_r \frac{\partial}{\partial r} u_{i,j}^{t+\Delta t} - \Delta T_r \frac{\Delta r}{r} \frac{\partial}{\partial r} u_{i,j}^{t+\Delta t} - \Delta T_v \frac{\partial^2}{\partial z^2} u_{i,j}^{t+\Delta t}
\]

\[
= \Delta T_r \frac{\partial}{\partial r} u_t + \Delta T_r \frac{\Delta r}{r} \frac{\partial}{\partial r} u_t + \Delta T_v \frac{\partial^2}{\partial z^2} u_t + \frac{\Delta t}{m_v} g(u, \varepsilon_z) + u_{i,j}^t
\]

Step I of the Peaceman -Rachford (P-R) ADI scheme consists of solving the equation implicitly in the \( r \)-direction and explicitly in the \( z \)-direction.

\[
u_{i,j}^{t+\Delta t} - \Delta T_r \frac{\partial}{\partial r} u_{i,j}^{t+\Delta t} - \Delta T_r \frac{\Delta r}{r} \frac{\partial}{\partial r} u_{i,j}^{t+\Delta t} = u_{i,j}^t + \Delta T_v \frac{\partial^2}{\partial z^2} u_t + \frac{\Delta t}{m_v} g(u, \varepsilon_z)
\] (3.21)

Step II of the Peaceman –Rachford (P-R) ADI scheme consists of solving the equation implicitly in the \( z \)-direction and explicitly in the \( r \)-direction.

\[
u_{i,j}^{t+\Delta t} - \Delta T_v \frac{\partial^2}{\partial z^2} u_{i,j}^{t+\Delta t} = u_{i,j}^t + \Delta T_v \frac{\partial^2}{\partial z^2} u_t + \Delta T_v \frac{\Delta r}{r} \frac{\partial}{\partial r} u_{i,j}^{t+\Delta t} + \frac{\Delta t}{m_v} g(u, \varepsilon_z)
\] (3.22)

A combination of Steps I & II (Eq. 3.21 & 3.22) is called the Peaceman –Rachford ADI Scheme. The first step is defined as a predictor which evaluates the parameter at time \( t + \Delta t / 2 \) based on the value at time \( t \), whereas Step II is called the corrector which corrects the parameter at time \( t + \Delta t \) based on the value predicted by the predictor. If we differentiate the predictor and corrector in a different form of tri-diagonal matrix, then:

The predictor term is:

\[
u_{i,j}^{t+\Delta t / 2} = C^I + C^{II} u_{i+1,j}^{t+\Delta t / 2} + C^{III} u_{i-1,j}^{t+\Delta t / 2}
\]

\( C^I, C^{II} \) & \( C^{III} \) are constant.

The corrector term is:
\[ u_{i,j}^{t+\Delta t} = C' + C'' \left( u_{i+1,j}^{t+\Delta t} + C''' \left( u_{i,j-1}^{t+\Delta t} \right) \right) \]

\( C', C'' \) and \( C''' \) are constants.

The predictor term is:

\[
\frac{t+\Delta t}{r} u_{i,j}^{t+\Delta t} = \frac{\Delta r}{r} \frac{\Delta r}{r} u_{i,j}^{t+\Delta t} - \Delta T_r \cdot \frac{\Delta r}{r} u_{i,j}^{t+\Delta t} = u_{i,j}^{t+\Delta t} + \Delta T_r \cdot \Delta T_v \cdot \frac{u_{i+1,j}^{t+\Delta t} + u_{i,j-1}^{t+\Delta t}}{2} \]

Further simplification leads to,

\[
\frac{t+\Delta t}{2} u_{i,j}^{t+\Delta t} \left( 1 - \Delta T_r \cdot \frac{\Delta r}{r} - \Delta T_r \cdot \frac{\Delta r}{r} \right) = u_{i,j}^{t+\Delta t} + \Delta T_v \cdot \frac{u_{i+1,j}^{t+\Delta t} + u_{i,j-1}^{t+\Delta t}}{2} \]

\[
\frac{t+\Delta t}{2} u_{i,j}^{t+\Delta t} = \Delta T_r \cdot \frac{u_{i+1,j}^{t+\Delta t} - u_{i,j}^{t+\Delta t}}{2} \]

The matrix form:

\[
\frac{t+\Delta t}{2} u_{i,j}^{t+\Delta t} = C' + C'' u_{i+1,j}^{t+\Delta t} + C''' u_{i,j-1}^{t+\Delta t} \]

For each \( j \),
\[
\begin{bmatrix}
1 & -C_{i,j}^n & 0 \\
-C_{i,j}^n & 1 & -C_{i,j}^n \\
0 & -C_{i,j}^n & 1 \\
\vdots & \ddots & \ddots \\
-C_{N-3,i}^n & \cdots & 1 & -C_{N-3,i}^n \\
(-C_{N-3,i}^n - C_{N-3,i}^n) & 1 & \vdots & \vdots \\
\end{bmatrix}
\begin{bmatrix}
u_{i,j}^t \vspace{1em} \\
u_{i+1,j}^t \vspace{1em} \\
u_{i,j+1}^t \vspace{1em} \\
u_{i,j-1}^t \vspace{1em} \\
u_{N-3,i}^t \vspace{1em} \\
u_{N-3,i}^t \\
\end{bmatrix}
= \begin{bmatrix}
C_{i,j}^t + C_{i,j}^n u_{i,j+1}^t + \Delta t \\
C_{i,j}^t \\
C_{i,j}^t \\
C_{i,j}^t \\
C_{N-3,i}^t \\
C_{N-3,i}^t \\
\end{bmatrix}
\begin{bmatrix}
u_{i,j}^t \vspace{1em} \\
u_{i+1,j}^t \vspace{1em} \\
u_{i,j+1}^t \vspace{1em} \\
u_{i,j-1}^t \\
u_{N-3,i}^t \\
u_{N-3,i}^t \\
\end{bmatrix}
\]

Where,
\[
C' = \frac{(1 - 2\Delta T_v)}{(1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r})} u_{i,j}^t + \Delta T_v \left( \frac{\Delta r}{r} \right)\frac{\Delta t}{1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r}} (u_{i,j+1}^t + u_{i,j-1}^t)
\]
\[
+ \frac{\Delta t}{1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r}} g(u, \epsilon_z)_{i,j}
\]
\[
C'' = \frac{\Delta T_r \cdot \left(1 + \frac{\Delta r}{r}\right)}{(1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r})}
\]
\[
C''' = \frac{\Delta T_r}{(1 + 2\Delta T_r + \Delta T_r \cdot \frac{\Delta r}{r})}
\]

The corrector term is:
\[
u_{i,j}^{t+\Delta t} - \Delta T_v \partial_r^2 u^{t+\Delta t} = u_{i,j}^{t+\Delta t} + \Delta T_r \partial_r^2 u_{i,j}^{t+\Delta t} + \Delta T_r \cdot \frac{\Delta r}{r} \partial_r u_{i,j}^{t+\Delta t} \frac{\Delta t}{m_v} + \frac{\Delta t}{m_v} g(u, \epsilon_z)
\]

Further simplification leads to,
\[
u_{i,j}^{t+\Delta t} - \Delta T_v \{u_{i,j+1}^{t+\Delta t} - 2u_{i,j}^{t+\Delta t} + u_{i,j-1}^{t+\Delta t}\}
\]
\[
= u_{i,j}^{t+\Delta t} + \Delta T_r \left\{u_{i,j+1}^{t+\Delta t} - 2u_{i,j}^{t+\Delta t} + u_{i,j-1}^{t+\Delta t}\right\} + \Delta T_r \cdot \frac{\Delta r}{r} \left\{u_{i+1,j}^{t+\Delta t} - u_{i,j}^{t+\Delta t}\right\}
\]
\[
+ \frac{\Delta t}{m_v} g(u, \epsilon_z)
\]
\[ u_{i,j}^{t+\Delta t} (1 + 2\Delta T_v) \]

\[ = \Delta T_v u_{i,j+1}^{t+\Delta t} + \Delta T_v u_{i,j-1}^{t+\Delta t} + u_{i,j}^{t+\Delta t} \left( 1 - 2\Delta T_r \frac{\Delta r}{r} \right) \]

\[ + u_{i+1,j}^{t+\Delta t} \Delta T_r \left( 1 + \frac{\Delta r}{r} \right) + \Delta T_r u_{i-1,j}^{t+\Delta t} + \frac{\Delta t}{m_v} g(u, \varepsilon_z) \]

The matrix form is:

\[ u_{i,j}^{t+\Delta t} = C^l + C^{III} u_{i,j+1}^{t+\Delta t} + C^{III} u_{i,j-1}^{t+\Delta t} \]

For each i

\[
\begin{bmatrix}
1 & -C_{i,1}^{ll} & 0 \\
-C_{i,2}^{ll} & 1 & -C_{i,2}^{ll} \\
0 & -C_{i,3}^{ll} & 1 \\
\vdots & \vdots & \vdots \\
-C_{i,M-1}^{ll} & 1 & -C_{i,M-1}^{ll} \\
-C_{i,M}^{ll} - C_{i,M}^{ll} & 1 & \end{bmatrix}
\begin{bmatrix}
u_{i,1}^{t+\Delta t} \\
u_{i,2}^{t+\Delta t} \\
u_{i,3}^{t+\Delta t} \\
\vdots \\
u_{i,M-1}^{t+\Delta t} \\
u_{i,M}^{t+\Delta t} \\
\end{bmatrix} =
\begin{bmatrix}
C_{i,1}^{l} + C_{i,1}^{ll} u_{i+1,j}^{t+\Delta t} \\
C_{i,2}^{l} \\
C_{i,3}^{l} \\
\vdots \\
C_{i,M-1}^{l} \\
C_{i,M}^{l} \\
\end{bmatrix}
\]

Where

\[ C^l = \frac{\left( 1 - 2\Delta T_r - \Delta T_r \frac{\Delta r}{r} \right) u_{i,j}^{t+\Delta t}}{(1 + 2\Delta T_v)} \]

\[ + \frac{\Delta t}{m_v (1 + 2\Delta T_v)} g(u, \varepsilon_z)_{i,j} \]

\[ C^{III} = \frac{\Delta T_r}{(1 + 2\Delta T_v)} \]

With the help of the pore water pressure at a different time at each node, the vertical strain at each node can be calculated using following equation:

\[ (\varepsilon_z)_{i,j,t+\Delta t} = (\varepsilon_z)_{i,j} - m_{vi,j} (u_{i,j,t+\Delta t} - u_{i,j,t}) + \Delta t g(u, \varepsilon_z)_{i,j} \quad (3.23) \]
3.2.4 Development of a computer Program

This entire computation has been performed using a finite-difference technique and by adopting forward, backward, and central differences techniques. For this purpose, a user friendly program has been written in MATLAB in a tri-diagonal matrix form and executed with settlement and pore water pressures as an output. Details of this program written in MATLAB are presented in Appendix A.

3.3 Radial consolidation analysis using an isotache approach

3.3.1 Background

Many coastal regions of Australia and Southeast Asia contain very soft clays (estuarine or marine), which have adverse geotechnical properties such as low bearing capacity and excessive long term (creep) settlement. This presents challenges in design and construction, especially for minimising long term deformation during the lifetime of the infrastructure. Construction, design and field stability problems are of economic significance in Australia, particularly along the East Coast where millions of dollars are spent annually on road maintenance alone. Every improvement in the geotechnical properties of saturated soft clays at significant depths (say 20 m) and accurate predictions of long term deformation can offer significant benefits to regional communities for infrastructure development.

The preloading method of stabilising soft clays by surcharge fill embankments (facilitated by prefabricated vertical drains, PVDs) is generally a low-cost solution (Hansbo, 1981, Bergado et al., 1991, Indraratna and Redana, 2000), but in sites with thick soft soil there can be a significant delay in consolidation due to very low soil permeability and lack of efficient drainage. This is why the installation of PVDs followed by the application of vacuum pressure (suction) as a preload (i.e. prior to
construction of the main structure) would facilitate the rapid dissipation of pore water pressure (Kjellman, 1952; Bergado et al., 2002; Bo et al., 2003; Geng et al., 2010).

The application of PVDs to improve the stability of soft ground has the potential to significantly reduce construction and maintenance costs and enhance the performance of infrastructure, through better drainage, greater load bearing capacity, and reduced long term settlement of the improved soil (Cascone and Biondi, 2013).

Analytical solutions for radial consolidation have been developed to consider aspects such as the smear zone, stratified soils, and vacuum preloading (Barron, 1948; Tang and Onitsuka, 2001, Indraratna et al., 2005; Walker and Indraratna, 2009). However, only limited efforts have been made to analytically incorporate the effect of soil viscosity for radial consolidation (Yang et al., 2016).

Evaluating the time-dependent behaviour of soft soils is important for predicting settlement and excess pore water. There are two different methods (Hypotheses A and B, see Fig. 3.4) for evaluating long term time-dependent settlements, and they can be categorised as follows:

**Hypothesis A**: The ratio between the coefficient of secondary consolidation ($C_a$) and ($C_v$) is assumed to be constant, and creep deformation occurs after excess pore water has been completely dissipated. This shows that the primary consolidation strains arising from transferring pore water pressure into the effective stress of the soil, and the secondary consolidation strains which cause viscous deformation (creep) of soils, are separated (Ladd, 1973; Mesri and Castro, 1987). The coefficient of secondary consolidation is used to estimate the resultant viscous deformation. Although
Hypothesis A is simple and is often used due to the reliability of $C_a$ and $C_c$, that approach can result in continuous long term settlement even at an infinite time.

**Hypothesis B:** The concept of isotache (Suklje, 1957, Barden, 1969; Bjerrum, 1967) assumes that some structural viscosity is responsible for creep, and it begins during the primary consolidation phase the excess pore water pressure has been dissipated (Fig. 3.4). As a result, strain at the end of primary consolidation increases with sample thickness, hence the experimental results obtained using thin samples do not represent the actual in-situ behaviour of thick clay fields due to the different time scale. In this hypothesis, a series of compression curves can be used to show the relationship between the strain rate and pre-consolidation pressure. The creep ratio can decrease with the strain rate which decreases with time. Various studies have been carried out on Hypothesis B (Leroueil, 1988, Yin et al., 1994, Adachi et al., 1996, Kim and Leroueil, 2001, Hawlader et al., 2003, Imai et al., 2005, Tanaka et al., 2006, Watabe et al., 2012, Qu et al., 2010, Degago et al., 2011).
Several studies have been carried out on the time-dependent behaviour of soft soil using the Isotache concept (Leroueil et al., 1985, Yin et al., 1994, Kim and Leroueil, 2001, Watabe et al., 2012, Qu et al., 2010, Degago et al., 2011) and the strain rate dependency of pre-consolidation pressure (Watabe and Leroueil, 2012, Watabe et al., 2012, Watabe et al., 2008, Tanaka, 2005). All of these isotaches are characterised by a constant reference time line obtained from a CRS test and a series of long term consolidation tests. However, Tsutsumi and Tanaka (2011) designed a special CRS test and performed an experiment on a single sample with a multiple strain rate applied in different stages. Despite this, there is still no application of the strain rate dependency of pre-consolidation pressure in a radial consolidation model to mimic the exact field condition, but this can be done by performing a constant rate of strain test along with a long term consolidation test. The isotaches obtained from these two
tests, and the strain rate dependency of pre-consolidation pressure, can be used to model the undissipated pore water pressure.

3.3.2 Modelling based on the Isotache concept

Based on the unique relationship between strain and consolidation pressure corresponding to the strain rate proposed by Watabe and Leroueil (2015), a strain rate dependency relationship between the strain rate and pre-consolidation pressure can be established using long term consolidation (LT) and a constant rate of strain (CRS) test (Fig. 3.5). The upper bound of the isotache represents the compression curve in the laboratory experiment with higher strain rate, whereas the other isotache lines resemble the compression curves at a lower strain rate. Watabe and Leroueil (2015) proposed a relationship between the pre-consolidation pressure and strain rate as:

\[ \sigma_p' = f(\epsilon_v') \]  

(3.18)

Based on long term consolidation (LT) and constant rate of strain (CRS) test the exponential relationship between the pre-consolidation pressure and the strain rate can be determined from:

\[ \ln \sigma_p' = a_1 + a_2 \ln \epsilon_v' \]  

(3.19)

\[ \sigma_p' = \sigma_{pL}' + b_1 \exp(b_2 \ln \epsilon_v') \]  

(3.20)

\[ \ln \frac{\sigma_p' - \sigma_{pL}'}{\sigma_{pL}'} = c_1 + c_2 \ln \epsilon_v' \]  

(3.21)

\[ \epsilon_v' = \left( \frac{\sigma_p' - \sigma_{pL}'}{\sigma_{pL}'} \right)^{\frac{1}{c_2}} \exp^{\frac{c_1}{c_2}} \]  

(3.22)

\[ \epsilon_v' = c_3 \left( \frac{\sigma_p' - \sigma_{pL}'}{\sigma_{pL}'} \right)^{c_4} \]  

(3.23)
where \( \sigma_p' = \text{preconsolidation pressure at a given strain rate} \),
\[ \sigma_{pl}' = \text{lower limit of preconsolidation pressure} \]
\[ \sigma_{p0}' = \text{preconsolidation pressure when } \dot{\varepsilon}_v = 1 \times 10^{-7} \text{s}^{-1} \]
\( \dot{\varepsilon}_v = \text{axial strain rate} \)
\[ c_3 = \exp \frac{c_1}{c_2} \text{ and } c_4 = \frac{1}{c_2} \text{ are two constants used in the model} \]
\[ c_1 = \ln \frac{\sigma_p' - \sigma_{pl}'}{\sigma_p'} \text{ at } \dot{\varepsilon}_v = 1 \text{ and } c_2 = \frac{1}{\ln \dot{\varepsilon}_v} \left[ \ln \frac{\sigma_p' - \sigma_{pl}'}{\sigma_p} - c_1 \right] \]

The lower limit of preconsolidation pressure (\( \sigma_{pl}' \)) can be calculated based on Eq. (3.23). An illustration of this procedure to evaluate the dependency of the strain rate on pre-consolidation pressure (\( \sigma_p' \)) from the CRS and LT test is shown in Fig. 3.5.

The analysis was carried out using clay samples obtained from Ballina NSW.

Figure 3.5 Illustration of the method to determine the strain rate dependency of pre-consolidation pressure using Constant rate of strain (CRS) and Long term (LT) consolidation test, (a) Reference time line, (b) Isotaches for Ballina clay, (c) Strain rate dependency of pre-consolidation pressure.

The relationship between \( \log \left( \frac{\sigma_p'}{\sigma_{p0}} \right) \)-log strain rate (\( \dot{\varepsilon}_v \)) for all worldwide clays has been examined and provided by Watabe et al. (2008), Watabe et al. (2012) and
Leroueil (1988). Samples from Ballina was examined in laboratory (both CRS and LT test) to find strain rate dependency of pre-consolidation pressure and plotted in Fig. 3.6 together with worldwide clays. Here the ratio $\frac{\sigma_{pL}'}{\sigma_{po}'}$ for Ballina clay is 0.86 and the ratio $\frac{\sigma_{p}'}{\sigma_{po}'}$ approaches unity at $\dot{\varepsilon} = 1 \times 10^{-07} s^{-1}$. For Ballina clay the value of $c_1$ and $c_2$ are 0.887 and 0.158, respectively, and by using these parameters, the curve fitting equation for Ballina clay has been established and is stated in Figure 3.6.

\[
\dot{\varepsilon}_v = 3.63 \times 10^{-03} \left( \frac{\sigma_{p}'}{\sigma_{po}'} - \frac{\sigma_{pL}'}{\sigma_{pL}'} \right)^{6.33}
\]

Figure 3.6 Log $\left( \frac{\sigma_{p}'}{\sigma_{po}'} \right)$-log strain rate ($\dot{\varepsilon}_v$) curve for all worldwide clays by Watabe and Leroueil (2015), compared with Ballina and Southeast Asian clay.
Figure 3.7 Conceptual drawing behind the model

A laboratory sample of soil having pre-consolidation pressure $\sigma_p'$ is loaded from initial effective stress of $\sigma_{v0}'$ to the final effective stress $\sigma_{vf}'$ and the corresponding strain rate for the sample is $\dot{\varepsilon}_1$. Based on Fig. 3.7, for a given preloading ($\Delta\sigma_t$), the change in effective stress at a 100% degree of consolidation based on pore pressure is calculated by:

$$\Delta\sigma_t = \Delta\sigma_c + \Delta\sigma_d$$

(3.24)

The first term $\Delta\sigma_c$ is an increase in effective stress due to the dissipation of excess pore-water pressure whereas the second term ($\Delta\sigma_d$) is an increase in effective stress.
due to delayed consolidation caused by the viscosity of clay. In order to estimate $\Delta \sigma_d$, the strain rate dependency of pre-consolidation pressure can be used and calculated using the following relationship:

$$\Delta \sigma_d = \sigma_{p_0}' - \sigma_{p(\dot{\varepsilon})}'$$  \hspace{1cm} (3.25)

Where $\sigma_{p_0}'$ corresponds to the laboratory pre-consolidation pressure corresponding to the strain rate of $1 \times 10^{-07} \text{s}^{-1}$ and $\sigma_{p(\dot{\varepsilon})}'$ is the pre-consolidation pressure at a given strain rate ($\dot{\varepsilon}$). Note that the strain rate can affect the location of $\sigma_{p(\dot{\varepsilon})}'$ in isotaches; the strain rate can be estimated using the following formulation:

$$\dot{\varepsilon} = \frac{\varepsilon_{U100}}{\Delta t_{U100}}$$  \hspace{1cm} (3.26)

where $\varepsilon_{U100}$ and $\Delta t_{U100}$ are the strain and time at the degree of consolidation based on a pore pressure equal to 100% ($U_{100}$), respectively. These parameters can be calculated based on formulations proposed by Indraratna et al. (2005a). Once the strain rate ($\dot{\varepsilon}$) is known, $\sigma_{p(\dot{\varepsilon})}'$ can be calculated based on Eq (3.23).

After estimating $\Delta \sigma_d$ using Eq. (3.25), the term $\sigma'_{vc}$ can be calculated using the following equation:

$$\sigma'_{vc} = \sigma'_{vf} - \Delta \sigma_d$$  \hspace{1cm} (3.27)

With the calculation of $\Delta \sigma_d$, the dissipation of excess pore water pressure and consolidation settlement ($\rho_c$) can be found using following formulations:

According to Indraratna et al. (2005a), the excess pore pressure at radial distance $r$ from the centre of a drain at any time $t$, $(u_r)$ by considering the linear variation of soil permeability in the smear zone can be calculated by:

$$u_r = \frac{1}{\mu r_e^2} \left[ r_e^2 \ln \left( \frac{r}{r_w} \right) - \frac{(r^2 - r_w^2)}{2} \right] \exp \left( -\frac{8F_ha^*}{\mu} \right) \Delta \sigma_t;$$  \hspace{1cm} (3.28a)
\[ \sigma_v' \leq \sigma_{p(\dot{\varepsilon})}' \] and \( t \leq t_i \)

\[ u_r = \frac{1}{\mu r_e^2} \left[ \frac{r_e^2}{r_w} \ln \left( \frac{r}{r_w} \right) - \frac{(r^2 - r_w^2)}{2} \right] \exp \left( -\frac{8\tau_{hi}}{\mu} \right) \left( \sigma_{v0}' + \Delta \sigma_t - \sigma_{p(\dot{\varepsilon})}' \right) \] \hspace{1cm} (3.28b)

\[ u_t = \sigma_{v0}' + \Delta \sigma_t - \sigma_{p(\dot{\varepsilon})}' \] at \( T_{hi} = 0 \) and \( t = t_i \), for \( \sigma_v' > \sigma_{p(\dot{\varepsilon})}' \) and \( t > t_i \),

where \( \mu = \ln \left( \frac{n}{s} \right) - \frac{3}{4} + \frac{\kappa(s-1)}{s-\kappa} \ln \left( \frac{s}{s-\kappa} \right) \), \( n = \frac{r_e}{r_w} \) and \( s = \frac{r_s}{r_w} \)

\( r_w \) is the radius of the drain, and \( r_e \) is the equivalent diameter of a soil cylinder which is the function of drain spacing. Similarly, \( r_s \) is the radius of smear zone, and \( \kappa \) is the permeability index.

The above mentioned modified time factor, \( T_{hi}^* \) for radial consolidation with vertical drain considering smear effect can be expressed as:

when \( \sigma_v' \leq \sigma_{p(\dot{\varepsilon})}' \) and \( t \leq t_i \)

\[ T_{h0}^* = P_{av,0} T_{h0} = 0.5 \left[ \left( \frac{\sigma_{p(\dot{\varepsilon})}'}{\sigma_{v0}'} \right)^{1-(c_s/c_k)} + 1 \right] T_{h0} \] \hspace{1cm} (3.29a)

\[ P_{av,0} = 0.5 \left[ \left( \frac{\sigma_{p(\dot{\varepsilon})}'}{\sigma_{v0}'} \right)^{1-(c_s/c_k)} + 1 \right] \] \hspace{1cm} (3.29b)

When, \( \sigma_v' > \sigma_{p(\dot{\varepsilon})}' \) and \( t > t_i \)

\[ T_{hi}^* = P_{av,i} T_{hi} = 0.5 \left[ \left( \frac{\sigma_{v0}' + \Delta \sigma_t'}{\sigma_{p(\dot{\varepsilon})}'} \right)^{1-(c_s/c_k)} + 1 \right] T_{hi} \] \hspace{1cm} (3.29c)

\[ P_{av,i} = 0.5 \left[ \left( \frac{\sigma_{v0}' + \Delta \sigma_t'}{\sigma_{p(\dot{\varepsilon})}'} \right)^{1-(c_s/c_k)} + 1 \right] \] \hspace{1cm} (3.29d)

The term \( t_i \) can be calculated from Eq. (3.29) when the effective pressure \( \sigma_v' \) is equal to the pre-consolidation pressure(\( \sigma_{p'} \)).

Now, the excess pore pressure dissipation ratio \( (R_u) \) at a distance \( r \) from the centre of drain and at a given time \( t \), can be calculated as:

70
\[ R_u = \frac{u_r - \Delta \sigma_d}{\Delta \sigma_t - \Delta \sigma_d} \]  

(3.30)

Similarly the consolidation settlement \( (\rho_c) \) can be calculated using the formulation given below:

\[ \rho_c = \frac{H c_s}{(1 + e_0)} \log \left( \frac{\sigma_v'}{\sigma_{v0}} \right) \quad \text{for} \quad \sigma_v' \leq \sigma_{p(i)}' \]  

(3.31a)

\[ \rho_c = \frac{H}{(1 + e_0)} \left[ c_s \log \left( \frac{\sigma_{p(i)}'}{\sigma_{v0}} \right) + c_c \log \left( \frac{\sigma_{vf}'}{\sigma_{p(i)}'} \right) \right] \quad \text{for} \quad \sigma_v' > \sigma_{p(i)}' \]  

(3.31b)

where \( c_c \) is a compression index, \( c_s \) is a recompression index and \( H \) is a compressible soil thickness.

The total settlement \( (\rho_t) \) comprises of consolidation settlement due to excess pore pressure dissipation \( (\rho_c) \) and additional settlement \( (\rho_a) \) due to delayed consolidation \( (\Delta \sigma_d) \).

\[ \rho_t = \rho_c + \rho_a \]  

(3.32)

The slope \( (\alpha) \) of the \( \log \left( \frac{\sigma_v'}{\sigma_{p0}} \right) - \log (\dot{\varepsilon}) \) at a given strain rate is equal to the secondary compression index \( (C_{\alpha e}) \) to the compression index \( (C_c) \) and is given by the following relationship:

\[ \alpha = \frac{\Delta \log \left( \frac{\sigma_v'}{\sigma_{p0}} \right)}{\Delta \log (\dot{\varepsilon})} = \frac{C_{\alpha e}}{C_c} \]  

(3.33)

The additional settlement then calculated using the following equation:

\[ \rho_a = d \dot{\varepsilon} \times dt \times H \]  

(3.34)
Mesri and Castro (1987) proposed a constant ratio \( C_{ae}/C_c \) for clay and Mesri et al. (1995) reported this value as 0.04±0.01 for inorganic clays. However, this value is not the constant parameter because the ratio of \( C_{ae}/C_c \) decreases with decrease in the strain rate. Figure 3.8 (a) shows the strain rate dependency of pre-consolidation pressure for Ballina clay with a constant slope (i.e. 0.047) whereas Figure 3-8 (b) shows that the ratio \( \alpha(=C_{ae}/C_c) \) decreases with a decrease in the strain rate. The value proposed by Mesri et al. (1995) is typically valid for laboratory strain rates between \( 1 \times 10^{-4} \) s\(^{-1} \) to \( 1 \times 10^{-6} \) s\(^{-1} \) (see Fig. 3.8b). This statement is more in line with Leroueil (2006) for Canadian and Swedish clays as well as with Watabe et al. (2012) for worldwide clays. Based on the variation of the \( C_{ae}/C_c \) ratio with the strain rate, it is very rough to take one constant value of \( C_{ae}/C_c \) based on laboratory strain rate. Due to this fact, an additional settlement can be calculated using Fig 3.8(b) with the help of Eq. (3.34).
CHAPTER 4: LABORATORY EXPERIMENT PROGRAM AND VALIDATION

4.1 Rowe cell testing

4.1.1 Background and history

The oedometer apparatus developed by Terzaghi (1925) has been used extensively to determine consolidation parameters but its limitations (i.e. 1-D vertical consolidation) imply it cannot correctly evaluate the parameters for radial consolidation or capture the lateral distribution of pore water pressure. The limitations of the oedometer tests are as follows:

1) The drainage path is only in a vertical direction and the sample size is very small.

2) The pore water pressures cannot be measured with a conventional oedometer due to the size of the sample.

3) The rigid disc used for loading purposes means that only equal strain can be simulated.

4) The type of loading in the oedometer is constant loading only.

To overcome these limitations, Rowe and Barden (1966) developed standard Rowe cells at Manchester University, as shown in Fig. 4.1. Rowe cells utilise a hydraulic loading system where water pressure is applied through a flexible diaphragm; this is completely different to a conventional oedometer with a mechanical lever system. Moreover, unlike an oedometer, larger samples can be tested in Rowe cell and large deformations can be measured with this loading arrangement. A schematic diagram of the very first Rowe cell is shown in Fig. 4.2.
Further advancements in the Rowe cell occurred in 1954, 1966, and 1967. Rowe (1954) modified the Rowe cell by introducing a diaphragm loading system which works with air pressure to carry out compression tests on sand, and later on, Rowe cells with 3 inch, 6 inch, 10 inch, and 20 inch diameters were developed.
Rowe cells are now available in the market with three different diameters (tabulated in Table 4.1). The pore water pressure transducer in these Rowe cells depends upon the nominal diameter of the base plate; in 3 inch and 6 inch Rowe cells, two pore water pressure transducers are usually fitted to measure the lateral flow of pore water, whereas in a 10 inch Rowe cell, four pore water pressure transducers are fitted to the base plate. These pore water pressure transducers are connected to the base plate through small porous discs and are located in a radial direction; one in the centre and others 0.55 R, 0.1R, and 0.9R away from the centre.

<table>
<thead>
<tr>
<th>Sample Diameter</th>
<th>Nominal Diameter</th>
<th>3 inch (75 mm)</th>
<th>6 inch (150 mm)</th>
<th>10 inch (250 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exact equivalent</td>
<td>mm</td>
<td>76.2</td>
<td>152.4</td>
<td>254</td>
</tr>
<tr>
<td>New Series</td>
<td>mm</td>
<td>75.7</td>
<td>151.4</td>
<td>252.3</td>
</tr>
<tr>
<td>Sample Area</td>
<td>mm$^2$</td>
<td>4560</td>
<td>4500</td>
<td>18241</td>
</tr>
<tr>
<td>Sample Height</td>
<td>mm</td>
<td>30</td>
<td>50</td>
<td>90</td>
</tr>
<tr>
<td>Sample Volume</td>
<td>cm$^3$</td>
<td>136.8</td>
<td>135</td>
<td>912.2</td>
</tr>
</tbody>
</table>

4.2 Soil classification
Samples of soft marine clay from a marine environment were obtained from a trial site along the Pacific Highway at Ballina, south of Brisbane. A sample of disturbed soil was extracted from a depth of 1.5 m, and the sub-soil within a 1.3 m to 2.2 m depth can be classified as highly compressible marine clay with very low permeability and high plasticity (CH). The basic properties obtained from this remoulded sample are tabulated in Table 4.2.
<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit, LL</td>
<td>94-102%</td>
</tr>
<tr>
<td>Plastic limit, PL</td>
<td>28-36%</td>
</tr>
<tr>
<td>Plasticity index, PI</td>
<td>58-74</td>
</tr>
<tr>
<td>Specific gravity, G&lt;sub&gt;s&lt;/sub&gt;</td>
<td>2.56-2.60</td>
</tr>
<tr>
<td>Water content: %</td>
<td>90.7-98.7</td>
</tr>
<tr>
<td>Void ratio, e</td>
<td>2.32-2.56</td>
</tr>
<tr>
<td>Bulk unit weight: kN/m&lt;sup&gt;3&lt;/sup&gt;</td>
<td>14.40-16.5</td>
</tr>
<tr>
<td>Undrained shear strength, s&lt;sub&gt;u&lt;/sub&gt;: kPa</td>
<td>9.4-12.3</td>
</tr>
</tbody>
</table>

### 4.3 Test apparatus and procedure

To prepare a sample of remoulded soil, the soil excavated from Ballina was taken to the laboratory where, after removing larger particles and shells it was mixed with water to 1.2 to 1.4 times its liquid limit to form a slurry which was then poured into air tight plastic bags and kept in a humidity room for 48 hours. The sample is then subjected to pre-consolidation in the same cell under 20 kPa in equal strain conditions and then a 6.9 mm diameter sand drain was inserted into the sample with minimum disturbance. The sample with 75mm diameter by 20 mm thick was then subjected to radial consolidation inside a Rowe cell (Fig. 4.3) developed by Rowe and Barden (1966). A pressure of 80 kPa was applied to the sample inside Rowe cell using a GDS pressure volume controller via a surcharge, vacuum, or a combination of both. The top drainage valve remained closed and a surcharge pressure was applied from the pressure/volume controller; this surcharge continued until the pore water pressure reached 90 % of the applied pressure, and then a vacuum pressure was applied via a vacuum pump. The corresponding settlement and pore water pressure
were recorded using linear vertical differential transducer (LVDT) and pore water pressure transducer, respectively. Five different tests were carried out using different surcharge and vacuum pressures. Details of the experimental procedure are summarised in Table 4.3.

Table 4.3 Details of experiments with varying VSR in a Rowe cell

<table>
<thead>
<tr>
<th>No.</th>
<th>Diameter of sample: mm</th>
<th>Diameter of drain: mm</th>
<th>n</th>
<th>Surcharge pressure: (kPa)</th>
<th>Vacuum pressure: (kPa)</th>
<th>Vacuum-Surcharge Ratio (VSR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>75</td>
<td>6.9</td>
<td>10.86</td>
<td>80</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>60</td>
<td>20</td>
<td>0.25</td>
<td>60</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>40</td>
<td>40</td>
<td>0.5</td>
<td>40</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>20</td>
<td>60</td>
<td>0.75</td>
<td>20</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>80</td>
<td>1</td>
<td>0</td>
<td>80</td>
<td></td>
</tr>
</tbody>
</table>

[Note: Vacuum Surcharge Ratio (VSR) = Vacuum Pressure (VP)/ (Vacuum Pressure + Surcharge Pressure (SP))]  
To capture lateral deformation under vacuum preloading, a peripheral rubber membrane was inserted between the wall of the Rowe cell and the soil sample. The rubber membrane fitted the Rowe cell properly and the ends were sealed at the top and bottom of the consolidation cell with wax. A layer of saturated filter paper was placed between cell wall and rubber membrane before fitting the rubber membrane, and the filter paper is in direct contact with the rim drain. Since there is some inward movement during vacuum consolidation, it was simulated in the Rowe cell by calculating the volume of water collected in the gap between cell wall and peripheral rubber membrane. The rim drain is connected to the GDS pressure/volume controller.
and the volume of water filling the gap under controlled lateral stresses can be determined by using the discharged amount of water from the controller.

![Diagram of Rowe cell](image)

Figure 4.3 (a) 75mm Rowe cell with drain, (b) plan view of base plate

### 4.4 Test results and discussion

The readings obtained from the linear variable differential transformer (LVDT) fitted to the top of Rowe cell are the value of axial settlement over a period of time because the LVDT readings increase from zero to settlement at any time during consolidation, whereas changes in the volume of water collected between cell wall and membrane are used to calculate lateral deformation. The vertical and lateral deformation readings were converted into strain and plotted with time. The variations
of axial and lateral strain with varying VSRs are shown in Figs. 4.4 and 4.5. Note that the axial strain decreases as the VSR increases, whereas this trend is in reverse for lateral strain. Here it increases with an increase in VSR as more inward deformation occurs as the vacuum increases in magnitude. Although there is a variation between the axial strain and lateral strain with VSR, the volumetric strain for the set of VSRs is the same; the ultimate volumetric strain is 25.5%.

As with settlement, the pore water pressures for all the experiments with different VSRs were recorded and plotted in Fig. 4.6. Based on the settlement and pore water pressure readings, two different plots for the Degree of Consolidation (DOC) were plotted; one of which was DOC based on settlement, while the other is DOC based on PWP (as shown in Fig. 4.7 and 4.8). Note that the degree of consolidation (DOC) based on settlement at any time always over-predicted DOC based on PWP which is in line with the conclusion made by Davis and Raymond (1965). There are other factors (such as permeability of porous stones, side friction, temperature, vibration, compressibility of pore pressure gauge, electro-chemical differences in pore fluid and fluid in pore pressure gauge), which influence the discrepancy between the two DOC during the laboratory experiment. In addition, stress increment ratio (final to initial effective pressure) plays an important role when determining DOC based on pore water pressure, however, this effect is negligible for evaluating DOC based on settlement. In practice, DOC based on settlement is adopted so as to calibrate the design parameters. However, there is still some debate regarding the choice of DOC when calibrating the soil parameters.
Figure 4.4 Variation of axial strain with time with different VSR

Figure 4.5 Variation of lateral strain with time with different VSR
Figure 4.6 Excess pore water pressure with different VSR

Figure 4.7 DOC based on settlement with different VSR
4.5 **Large scale consolidometer**

4.5.1 **Background**

Reconstituted small samples are often used to assess the performance of radial consolidation due to PVDs, but the permeability and compressibility of samples of undisturbed soil often differ from remoulded ones. This problem seems to be more complex in a marine environment due to the presence of random coarse particles such as gravels, shells, and natural partings. Small scale laboratory experiments using reconstituted samples, especially in a marine environment, cannot predict the
exact behaviour of soil in the field, so a large scale consolidometer (350 mm dia.) with an undisturbed specimen was used in an experimental program at a UOW laboratory to determine the exact consolidation properties; these specimens were obtained from soil taken from a site along the Pacific Highway, north of Sydney. Moreover, this test also gives some idea of the smear effect factor which is very useful when converting laboratory properties into real field properties.

4.5.2 Apparatus

A schematic diagram of the combined two-in-one large-scale corer and consolidometer is shown in Fig. 4.9. It consists of, (a) a cylindrical corer, (b) a loading rig platform and rig, and (c) a pneumatic air pressure chamber.

1) Cylindrical corer: it is manufactured by rolling 5 mm thick steel plate into a 350 mm diameter by 700 mm long cylinder and then cutting it longitudinally into two halves. Teflon spray is applied to the inside walls to reduce friction, and two pore pressure transducers are located beneath the bottom lid; one in the centre and one 96.25 mm from the centre. A custom made piston with two O-ring grooves transfers the load from a pneumatic air pressure chamber on top of the sample; O-rings are fitted into the grooves to prevent air or water leakage.

2) The platform and loading rig: The bottom part of the corer is attached to the base of the platform while the upper part, including the piston and air pressure chamber, are attached to the loading rig.

3) The pneumatic air pressure chamber: This chamber provides a load onto the sample during testing, and a load cell is used to measure the applied load.
Figure 4.9 Schematic diagram of a large scale consolidometer (modified after Indraratna and Redana, 1998)

Three large scale consolidometers have been developed at the High bay lab, University of Wollongong Australia (UOW) to examine large scale specimens of undisturbed clay extracted from Ballina, NSW. The first consolidometer (LSC1) is set up to investigate the effect of undisturbed Ballina clay without PVD and vacuum preloading, the second one (LSC2) investigates the effect PVD under surcharge preloading, while the third investigates the effect of PVD under combined surcharge and vacuum preloading.
Large scale consolidometer1 (LSC1): Undisturbed Ballina clay + 80 kPa (Staged loading)

Large scale consolidometer2 (LSC2): Undisturbed Ballina clay + PVD + 80 kPa (Staged loading)

Large scale consolidometer3 (LSC3): Undisturbed Ballina clay + PVD + 20 kPa + Vacuum (60 kPa)

4.5.3 Extraction of samples

Samples of soft clay in a marine environment were obtained from a trial site along the Pacific Highway at Ballina, south of Brisbane. A sample of undisturbed soil was extracted from a depth of 1.5 m, where sub-soil from 1.3 m to 2.2 m deep can be classified as highly compressible marine clay with very low permeability and high plasticity (CH). The basic properties obtained from this sample are tabulated in Table 4.4.

Table 4.4 Basic properties of Ballina clay (1.3m to 2.2m depth)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit, LL</td>
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</tr>
<tr>
<td>Undrained shear strength, s_u: kPa</td>
<td>9.4-12.3</td>
</tr>
</tbody>
</table>
To extract this sample of undisturbed soil, a 350 mm diameter by 700 mm long cylindrical corer is pushed into the soil by a light excavator to a depth of 1.5 m-2.2 m; this static push method prevents the corer from rotating, thus ensuring a quality sample (Andresen and Kolstad, 1979). Furthermore, the quality of the sample in relation to the degree of disturbance was confirmed by calculating the area ratio for a 5 mm thick wall and a 345 mm internal diameter; it turned out to be 2.9%, which confirmed that the degree of disturbance during the retrieval stage was less than 10% (Hvorslev, 1949). The corer also has a top cap to stop any loss of moisture when it is pushed into the ground. After the corer is filled with soil, the surrounding soil is excavated and the corer is then removed from the bottom of the pit. The sample is trimmed and sealed at both ends with wax, and both end caps are re-attached and tightened so there is no air gap at either end of the sample; this technique also helps to prevent any stress relief. The samples are transferred to the laboratory and stored in a room with controlled humidity. The overall process of sample extraction is shown in Fig.4.10.

![Figure showing (a) excavation of trench (b) Insertion of corer cum cylinder (c) Extracted undisturbed Ballina clay sample (d) transportation of samples to UOW laboratory.](image-url)
4.5.4 Laboratory setup of samples

A traditional band-shaped drain (100 mm × 3 mm), based on the equivalent diameter and spacing in the field, was scaled down for the purpose of laboratory simulation, hence a model prefabricated vertical drain (PVD) of 25 mm × 3 mm (i.e. equivalent wick drain radius, \( r_w = 7 \) mm) was used. A hollow rectangular mandrel (30 mm × 5 mm) was used to insert the model PVD into the laboratory sample. This hollow rectangular mandrel had an equivalent mandrel diameter (\( d_m \)) of 13.8 mm. The model PVD was installed by pushing the mandrel at a constant speed along a vertical guide to the required depth inside the corer containing the undisturbed sample, and then withdrawing it immediately after it reached the base of the sample. A linear variable differential transformer (LVDT) is connected to the top of the piston to measure the vertical displacement of the sample. Two pore water pressure transducers are connected to the bottom base at different locations, and then the undisturbed sample is compressed under an axial pressure of 80 kPa in four different phases (20 kPa each time).

The consolidometer has been modified to capture the lateral deformation of sample (LSC3); this consisted of wrapping a 1.5mm diameter porous plastic sheet around the inner wall of the corer and then placing a 1.5mm thick rubber membrane between the sample and the porous plastic sheet; a rim drain is then fitted around the piston and connected to a pressure volume controller. There is a gap between the rubber and the wall of the corer as the soil moves towards the drain due to vacuum consolidation; these gaps are filled with water from the valve connected to the rim drain with respect to time. Based on the volume of water flow in the gap between the wall of the cell and the rubber membrane with time, the lateral deformation of the
sample can be calculated. A surcharge load of 20 kPa and a vacuum pressure 60 kPa (VSR=0.75) was applied to the consolidometer (LSC3).

4.5.5 Non-Darcian flow relationship with reference to radial consolidation

The flow of pore water with the radial distance can be estimated by the pore water pressure transducers fitted to the base of consolidometer. For this purpose, a graph between the velocity of pore water flow and hydraulic gradient is plotted and it was found that the flow is non-Darcian rather than bilinear, as proposed by Hansbo (2001). Based on this plot, a relationship was found between the velocity of flow and hydraulic gradient which satisfied the power law, and where these non-Darcian flow constants are used for modelling purposes. The plot for non-Darcian flow is shown in Fig. 4.11 and indicates the clear deviation from the straight line following the power law rather than following linear flow.

Figure 4.11 Non-Darcian flow for radial consolidation
4.5.6 Test results and discussion

Three consolidometers were set up in the High bay lab at the University of Wollongong Australia (UOW) to compare the effect of a vertical drain and a vacuum in soft soil. Long term consolidation was observed for all three samples (LSC1, LSC2 & LSC3); the first consolidometer took almost 3.5 years to complete consolidation, and this experiment still continues because there is still some pore water pressure which needs to dissipate. The second consolidometer with a scaled down prefabricated vertical drain took almost a year to complete the consolidation process, and in a similar pattern, the third consolidometer (LSC3) took almost a month to complete the consolidation process. Two consolidometers (LSC1 and LSC2) were under a load of 80 kPa whereas the third one (LSC3) was a combination of surcharge (20 kPa) and vacuum (60 kPa) with a VSR=0.75. This shows the difference between a ground improvement technique using prefabricated vertical drain with vacuum preloading and the classical consolidation approach. The settlement, excess pore water pressure (EPWP) for consolidometer LSC1, LSC2 and LSC3, are shown in Figs.4.12 to 4.14, respectively.

Following a similar trend for excess PWP, the dissipation rate for the consolidometer under surcharge and vacuum was the fastest, followed by the large scale consolidometer with PVD and without PVD. In addition, the lateral strain due to the application of a vacuum was evaluated for the large scale consolidometer and combined with axial strain in order to obtain the volumetric strain; this volumetric strain was then compared to all three consolidometers and plotted as a time-strain curve for all three samples. Based on the time-strain and excess pore water pressure dissipation plot, the coefficient of horizontal consolidation for all these experiments
were found, and from which it was concluded that a vacuum can increase the coefficient of horizontal consolidation ($C_h$) by 20-30%. The coefficient of vertical consolidation in the first large scale consolidometer was at a minimum compared to the other two large scale consolidometers.
Figure 4.12 (a) Loading stages (b) time-strain plot (c) excess PWP dissipation plot of Large scale consolidometer (LSC1) consisting undisturbed Ballina clay under surcharge preloading.
Figure 4.13 (a) Loading stages (b) time-strain plot (c) excess PWP dissipation plot of Large scale consolidometer (LSC2) consisting undisturbed Ballina clay with scaled down PVD under surcharge preloading.
Figure 4.14 (a) Loading stages (b) time-strain plot (c) excess PWP dissipation plot of Large scale consolidometer (LSC3) consisting undisturbed Ballina clay with scaled down PVD under combined surcharge and vacuum preloading (VSR=0.75).
4.5.7 Evaluation of smear effect for large scale undisturbed sample

It was inevitable that the soil surrounding the mandrel for the large sample would be disturbed while installing the vertical drain; this results in a smear zone. To evaluate the extent of this smear zone, a graph showing the normalised reduction in water content \(((W_{\text{max}}-W)/W_{\text{max}})\) and the ratio of the equivalent radius to the radius of the mandrel \((r/r_m)\) is plotted as shown in Fig. 4.15. The smear zone obtained from the larger sample (LSC2) was then compared with the in-situ smear zone determined experimentally for single and multi-drain cases (Indraratna et al., 2015), and with the experimental approach by Sathananthan and Indraratna (2006). It is found that the radius of the smear zone can be almost 5 times the radius of the mandrel, \((r/r_m=5)\) for the large undisturbed sample and the field study (both single and multi-drain), but this ratio was twice that obtained from the small laboratory specimens \((r/r_m=2.5)\). In this respect, soil in the region with the equivalent diameter can be divided into three different zones: (a) the smear zone \((0 \leq r/r_m \leq 5)\) (b) the marginally disturbed zone \((5 < r/r_m \leq 6.5)\), (c) the undisturbed zone \((6.5 < r/r_m \leq r_v)\). The smear zone obtained from the 350 mm-diameter specimen with a scaled-down mandrel and drain is similar to the in-situ smear zone, indicating that the use of large undisturbed samples is better able to represent the field behaviour for this particular marine clay where natural partings and relics of the marine environment (as shown in Fig.4.16) may influence the consolidation behaviour.
Figure 4.15 Determining the smear zone in large specimen using the water content approach

Figure 4.16 Soil sample showing the presence of large shells and natural partings
4.6 Validation

4.6.1 Validation using the EVP model

An elastic visco-plastic (EVP) model developed in Chapter 3 was utilised to model the large scale consolidometer, while the settlement and excess pore water pressure were plotted with the laboratory data. Table 4.5 lists all the properties required to model the consolidometers according to EVP model while considering non-Darcian flow.

Table 4.5 Soil and drain properties used for EVP model

<table>
<thead>
<tr>
<th>Soil Parameter(s)</th>
<th>(\kappa/V)</th>
<th>(\lambda/V)</th>
<th>(\psi/V)</th>
<th>(e_0)</th>
<th>(\alpha_c) (m/s)</th>
<th>(\gamma_s) (kN/m(^3))</th>
<th>(p_c')</th>
</tr>
</thead>
<tbody>
<tr>
<td>value</td>
<td>0.034</td>
<td>0.148</td>
<td>0.003</td>
<td>3.1</td>
<td>5.28\times10^{-10}</td>
<td>14.5</td>
<td>46</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Drain Parameter(s)</th>
<th>(d_e) (mm)</th>
<th>(d_w) (mm)</th>
<th>(d_s) (mm)</th>
<th>(n)</th>
<th>(q_w) (m(^3)/s)</th>
<th>(s)</th>
<th>(\beta)</th>
</tr>
</thead>
<tbody>
<tr>
<td>value</td>
<td>350</td>
<td>14</td>
<td>108.78</td>
<td>25</td>
<td>8\times10^{-5}</td>
<td>7.77</td>
<td>1.28</td>
</tr>
</tbody>
</table>

4.6.2 Validation using Isotache model

The isotache model developed in Chapter 3 was also used to model the consolidometer. The strain rate corresponding to the small Rowe cell sample is 1\times10^{-07} s\(^{-1}\) whereas the strain rate for 350 mm diameter undisturbed sample is 1.97\times10^{-08} s\(^{-1}\). Based on the change in strain rate, a conversion ratio of 0.92 was determined by using the strain rate dependency of pre-consolidation pressure. This ratio converts the laboratory pre-consolidation pressure to the pre-consolidation pressure which corresponds to the strain rate of the 350 mm diameter specimens. Table 4.6 shows
the properties of the soil and drain needed to carry out the analysis using the isotache model; the results obtained from it are plotted with the laboratory data.

### Table 4.6 Soil and drain properties used for the isotache model

<table>
<thead>
<tr>
<th><strong>Drain Parameter(s)</strong></th>
<th>(d_u) (mm)</th>
<th>(d_d) (mm)</th>
<th>(d_c) (mm)</th>
<th>(s)</th>
<th>(n)</th>
<th>(k_h/k_s)</th>
<th>(\mu)</th>
<th>(\sigma_{PL}'/\sigma_{P0}')</th>
<th>(c_1)</th>
<th>(c_2)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>value</strong></td>
<td>14</td>
<td>108.8</td>
<td>350</td>
<td>7.77</td>
<td>25</td>
<td>3</td>
<td>3.602</td>
<td>0.86</td>
<td>0.887</td>
<td>0.158</td>
</tr>
<tr>
<td><strong>Soil Parameter(s)</strong></td>
<td>(c_0)</td>
<td>(C_v/(1+e_0))</td>
<td>(C_v/(1+e_0))</td>
<td>(\gamma_s)</td>
<td>(p_c')</td>
<td>(k_h) (m/s)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>value</strong></td>
<td>3.1</td>
<td>0.42</td>
<td>0.02</td>
<td>14.5</td>
<td>46</td>
<td>3.078×10^{-9}</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 4.6.3 Validation using numerical simulation

Finite element modelling (FEM) is utilised using PLAXIS 2D (Brinkgreve et al., 2015), which uses linear strain triangular elements with 6 nodes and 3 integration (stress) points for the validation purpose. A unit cell is used as an input in finite element software, and the top, bottom, and outer boundaries are fixed and set to be impermeable. A load is then applied onto the unit cell as a uniformly distributed load function and a drain is positioned at the centre of the unit cell. Three zones are found by evaluating the smear effect on this large scale test (refer section 4.5.7), all of which are modelled for LSC2 and LSC3, whereas only one zone (undisturbed zone) is modelled for consolidometer without a vertical drain (LSC1) because there will be no smear zone effect because there is no drain installation process. The corresponding permeability \(k_h\) values for the marginally disturbed zone and smeared zone are 1.5 and 3.5 times less than the undisturbed zone, respectively.
Since modelling in PLAXIS with vacuum preloading is relatively new and there is some information in the manual regarding modelling with vacuum preloading, the following are summarised as the key points when modelling under vacuum preloading:

1) A deformation is only allowed in the Y-max direction so the other boundaries are fixed for Large scale consolidometers (LSC1 and LSC2), but both Y-max and X-max were kept open for the large scale consolidometer (LSC3) because there will be inward lateral deformation when a vacuum is applied.

2) All of the boundaries except the top (Y-max) were made impermeable for large scale consolidometer and allowed pore water pressure to dissipate from the top for large scale consolidometer whereas for large scale consolidometer (LSC2), the bottom (Y-min) and outer (X-max) were kept impermeable. The leftmost boundary (X-min) is now permeable due to the prefabricated vertical drain which enables radial consolidation. Similar arrangements in terms of permeable and impermeable boundaries are defined for the third large scale consolidometer (LSC3).

3) The drain element in LSC2 is modelled under normal behaviour but the behaviour of the drain has been changed into a vacuum with an appropriate head (as under pressure) for LSC3 with vacuum consolidation. Meanwhile, the suction option in PLAXIS 2D was de-selected under vacuum consolidation.

4) Since there is a reduction in the groundwater head, the soil close to the vacuum area acts as if it is unsaturated. Therefore it is imperative to set the saturated and unsaturated unit weight of soil to the same value (under
material data set). Moreover, the hydraulic model in groundwater tab sheet must be set to “Saturated” in order to model vacuum preloading accurately.

5) The type of calculation under a vacuum should be changed to a fully coupled flow deformation analysis or simple consolidation type analysis.

All of these unit cell models are analysed using the soft soil creep model (Vermeer and Neher, 1999). The permeability values used to simulate all three consolidometers are tabulated in Table 4.7, while the soil parameters required to simulate a soft soil creep model are tabulated in Table 4.8. The mesh discretisation for all these simulations along with element type is shown in Fig 4.17.

Figure 4.17 FEM analysis (a) element used, mesh discretisation used in Ballina clay (b) without PVD (c) with PVD (d) with PVD and vacuum
Table 4.7 Permeability value used in FEM analysis

<table>
<thead>
<tr>
<th>Consolidometer</th>
<th>Permeabilities (m/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Smeared zone (0-34.5 mm)</td>
</tr>
<tr>
<td>LSC1</td>
<td>k_h</td>
</tr>
<tr>
<td></td>
<td>k_v</td>
</tr>
<tr>
<td>LSC2</td>
<td>k_h</td>
</tr>
<tr>
<td></td>
<td>k_v</td>
</tr>
<tr>
<td>LSC3</td>
<td>k_h</td>
</tr>
<tr>
<td></td>
<td>k_v</td>
</tr>
</tbody>
</table>

Table 4.8 Soil parameters used in soft soil creep model

<table>
<thead>
<tr>
<th>Parameter(s)</th>
<th>Unit weight ((\gamma_{sat}))</th>
<th>Initial void ratio ((e_0))</th>
<th>Modified compression index ((\lambda^*))</th>
<th>Modified swelling index ((\kappa^*))</th>
<th>Cohesion (c)</th>
<th>Over-consolidation ratio (OCR)</th>
<th>Creep coefficient ((\mu^*))</th>
</tr>
</thead>
<tbody>
<tr>
<td>value</td>
<td>14.5</td>
<td>3.1</td>
<td>0.148</td>
<td>0.034</td>
<td>5.0</td>
<td>2.1</td>
<td>0.003</td>
</tr>
</tbody>
</table>

4.7 Results and discussions

With the parameters as discussed in section 4.6, the strain and dissipation of excess pore water pressure for every consolidometer obtained from the EVP model, the isotache model, and the FEM model (using soft soil creep) are plotted with laboratory results and shown in Figs. 4.18 to 4.20 for LSC1, LSC2, & LSC3, respectively.
The results (Figs. 4.18 to 4.20) reveal that these developed analytical models are very useful for predicting the behaviour of soft soil with a viscous skeleton. The flow parameters obtained from testing a large undisturbed specimen (i.e. non-Darcian flow for fluid) when used with elastic visco-plastic model, can accurately predict the settlement and the dissipation of excess pore water pressure. Similarly, the isotache model (based on the strain rate dependency of pre-consolidation pressure) can also accurately predict soil behaviour in terms of settlement and pore water pressure with retardation in EPWP as well. In this particular case, because the strain rate changed from $1 \times 10^{-07}$ s$^{-1}$ to $1.97 \times 10^{-08}$ s$^{-1}$, a ratio of 0.92 was found out as the ratio of laboratory pre-consolidation pressure to pre-consolidation pressure corresponded to the 350 mm diameter the undisturbed specimens. In this case the delay in effective pressure is less than 2.00 kPa, which is very low for this laboratory case, however, the EPWP dissipation from isotache model is much slower than the other results. Moreover, the results obtained from the numerical simulation using the soft soil creep model agree with the laboratory results. Since the input in the numerical simulation is based solely on testing a large scale undisturbed sample (350mm diameter) in the laboratory with a smear and marginally disturbed zone, the output agrees well with the laboratory result. This confirms that the analytical model and numerical simulation are more accurate when using laboratory data obtained from testing of large undisturbed specimen.
Figure 4.18 (a) Loading stages (b) time-strain plot (c) excess PWP dissipation plot of Large scale consolidometer (LSC1) consisting of undisturbed Ballina clay under surcharge preloading compared to the numerical simulation.
Figure 4.19 (a) Loading stages (b) time-strain plot (c) excess PWP dissipation plot of Large scale consolidometer (LSC2) consisting undisturbed Ballina clay with scaled down PVD under surcharge preloading compared with numerical simulation.
Figure 4.20 (a) Loading stages (b) time-strain plot (c) excess PWP dissipation plot of Large scale consolidometer (LSC3) consisting undisturbed Ballina clay with scaled down PVD under combined surcharge and vacuum preloading (VSR=0.75) compared with numerical simulation.

[NOTE: For a large scale consolidometer (LSC3), the volumetric strain is the function of axial strain plus twice the lateral strain obtained from the numerical simulation, which is similar to the field results.]
CHAPTER 5: APPLICATION OF ISOTACHE MODEL TO CASE HISTORIES

5.1 Pacific Highway, Ballina Bypass

This selected site had a trial embankment built on its uniform layers of soft to firm estuarine and alluvial clays above residual soil and bedrock; the soft clay under the embankment was 25 m thick. The basic soil properties (water content, density, void ratio and soil profile) are shown in Figure 5.1 (Indraratna et al., 2012). The groundwater table appeared at 0.2 m below the ground surface overlying a 10 m thick layer of soft silty clay with undrained shear strength between 5-15 kPa, followed by medium silty clay to a depth of 25 m depth where the maximum undrained shear strength is around 48 kPa.

According to Kelly and Wong (2009), the total surcharge thickness of up to 11.2m, depending on the thickness of clay, was selected to limit the post-construction settlement to 50 mm within 2 years; the bulk density of this fill is 20 kN/m$^2$. To accelerate consolidation, 34mm diameter vertical drains were installed in a square grid with a spacing of 1m. Of the two sections of this embankment: Section B was
consolidated with vacuum and surcharge preloading, which further reduced the construction time, whereas Section A was consolidated with conventional surcharge. The surface settlement obtained from the settlement plate (SP1) and the excess pore pressure obtained from the piezometer (P1) installed 3.3 m deep at the centreline of the embankment in Section A were used for comparative purposes. The embankment was raised to a height of 2 m within 150 days and to its final height of 4 m after 250 days. Table 5.1 tabulates the properties of the drain with the diameter of the smeared zone as well as the ratio of pre-consolidation pressure ($\frac{\sigma_{pl}'}{\sigma_{p0}'}$) with 2 other parameters ($c_1 \& c_2$), as defined in Chapter 3. The properties of Ballina clay with pre-consolidation pressures are presented in Table 5.2 (Indraratna et al., 2012).

Table 5.1 Drain properties and strain rate dependency parameters for Ballina

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$d_w$</th>
<th>$d_s$</th>
<th>$d_c$</th>
<th>$s$</th>
<th>$n$</th>
<th>$k_h/k_s$</th>
<th>$\mu$</th>
<th>$\frac{\sigma_{pl}'}{\sigma_{p0}'}$</th>
<th>$c_1$</th>
<th>$c_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>value</td>
<td>51.5 mm</td>
<td>287 mm</td>
<td>1356 mm</td>
<td>5.57</td>
<td>26.33</td>
<td>3</td>
<td>6.620</td>
<td>0.86</td>
<td>0.887</td>
<td>0.158</td>
</tr>
</tbody>
</table>

Table 5.2 Soil parameters for Ballina Bypass (Indraratna et al., 2012)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$e_0$</th>
<th>$C_r/(1+e_0)$</th>
<th>$C_r/(1+e_0)$</th>
<th>$\gamma_s$ (kN/m$^3$)</th>
<th>$p_{c(lab)}$ (kPa)</th>
<th>$k_s$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>2.9</td>
<td>0.32</td>
<td>0.02</td>
<td>14.5</td>
<td>67</td>
<td>$1 \times 10^{-9}$</td>
</tr>
<tr>
<td>2-10</td>
<td>2.9</td>
<td>0.36</td>
<td>0.06</td>
<td>14.5</td>
<td>46</td>
<td>$1 \times 10^{-9}$</td>
</tr>
<tr>
<td>10-25</td>
<td>2.6</td>
<td>0.40</td>
<td>0.04</td>
<td>15.0</td>
<td>56</td>
<td>$3.3 \times 10^{-10}$</td>
</tr>
</tbody>
</table>
Figure 5.2 Settlement and excess PWP dissipation of embankment using Isotache model
The staged loading, field time-settlement and excess pore water dissipation (EPWP) curves with predictions using the current model proposed by Indraratna et al. (2005), and finite element analysis model by Indraratna and Redana (2000) are plotted in Fig 5.2. A strain rate of $8.54 \times 10^{-11} \text{ s}^{-1}$ was observed in the field which was much lower than the laboratory strain rate ($1 \times 10^{-07} \text{ s}^{-1}$). The settlement curve obtained from the current model always over-predicted the other curves, because, it accounts for creep capturing the variation of the coefficient of secondary consolidation with a decreasing strain rate. However, the settlement predictions using all three models agree with the field data, while the EPWP dissipation curve based on the current isotache model is the closest to the field data, compared to the models of Indraratna et al. (2005) and Indraratna and Redana (2000). The current model shows that the value of EPWP remained at 32 kPa even after 445 days, and then decreased to 20 and 12 kPa when using the models by Indraratna et al. (2005) and Indraratna and Redana (2000), respectively. The dissipation of EPWP in the field was delayed, so the pore water pressure only began to dissipate in the field after 100 days, even though settlement was occurring; this could be due to an error in the instrument not recording the data, properly.

5.2 Second Bangkok International Airport (SBIA), Bangkok
The Second Bangkok International Airport was constructed to cater for the high demand of air-traffic; it was built on swampy land (marine deposits) previously used as agricultural land. The ground level was located at around 1m above the mean sea level. The site consisted of 1.5 m of weathered crust above a 12m thick layer of soft clay followed by an 8-10 m thick layer of stiff clay. There was a uniform layer of dense sand below the layer of stiff clay.
Figure 5.3 shows the water content (w %), Atterberg limits (LL, PL, PI), unit weight (γ), and specific gravity (Gₕ), along with layer classification with depth. The shear strength of clay at different depths and other compressibility parameters are shown in Figure 5.4 where the shear strength for the upper weathered crust is between 30-40 kPa, but only 5-10 kPa for the underlying soft soil to a depth of 9 m.

Prefabricated vertical drains (PVDs) were chosen as the appropriate ground improvement technique because it would increase the shear strength of the soil and
reduce long term deformation. The installation depth of PVD was fixed to 12 m. A cross section of the embankment with the layout of PVDs for Section TS2 is shown in Fig. 5.5. The PVDs (94 mm by 3 mm) with grooved channels (made up of Polyolefin) were installed with the help of a mandrel having dimensions of 125 mm by 45 mm. The drain was installed at 1.2 m spacing in a square pattern. The fill was 4.2 m high and was completed in four different stages within 275 days (see Figure 5.5).

Table 5.3 provides the drain properties and strain dependency parameters for soils at the site of SBIA. The field strain rate was $1.22 \times 10^{-12}$ s$^{-1}$. A pre-consolidation pressure ratio of 0.82 and the necessary parameters ($c_1$ & $c_2$) were determined based on the strain rate and pre-consolidation plot and as shown in Table 5.3, while Table 5.4 shows the soil properties corresponding to different layers together with their pre-consolidation pressures.
Figure 5.5 Embankment cross-sections with sub-soil profile, SBIA (modified after AIT, 1995)

Table 5.3 Drain properties and strain rate dependency parameters for SBIA

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$d_w$</th>
<th>$d_s$</th>
<th>$d_c$</th>
<th>$s$</th>
<th>$n$</th>
<th>$k_i/k_s$</th>
<th>$\mu$</th>
<th>$\frac{\sigma_{pl}}{\sigma_{p0}}$</th>
<th>$c_1$</th>
<th>$c_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>value</td>
<td>48.5</td>
<td>270</td>
<td>1356</td>
<td>5.57</td>
<td>26.33</td>
<td>3</td>
<td>6.0144</td>
<td>0.82</td>
<td>0.887</td>
<td>0.158</td>
</tr>
</tbody>
</table>
Table 5.4 Soil properties for SBIA (AIT, 1995)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$e_0$</th>
<th>$C_e/(1+e_0)$</th>
<th>$C_r/(1+e_0)$</th>
<th>$\gamma_s$ (kN/m$^3$)</th>
<th>$p_{c(lab)}$’ (kPa)</th>
<th>$k_h$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>2.80</td>
<td>0.38</td>
<td>0.04</td>
<td>16</td>
<td>70</td>
<td>$2.04 \times 10^{-9}$</td>
</tr>
<tr>
<td>2-8</td>
<td>5.90</td>
<td>0.35</td>
<td>0.035</td>
<td>14</td>
<td>46</td>
<td>$4.67 \times 10^{-10}$</td>
</tr>
<tr>
<td>8-12</td>
<td>4.00</td>
<td>0.43</td>
<td>0.042</td>
<td>15</td>
<td>58</td>
<td>$2.05 \times 10^{-11}$</td>
</tr>
</tbody>
</table>

The stage loading with time settlement and excess pore pressure curves are plotted together with the predictions in Fig. 5.6. Similarly to the previous case history, predicting settlement using the current model over-predicts the other two predictions, because, creep settlement was used in this analysis. However, the settlement predictions using all three models are generally within an acceptable range with the field data. The remaining excess PWP to be dissipated at the end of 420 days are 4.40, 2.60 & 1.30 kPa for the current model, Indraratna et al. (2005) and Indraratna and Redana (2000), respectively. The results of excess pore water pressure from the proposed model agree with the field observation, unlike those predicted by Indraratna et al. (2005) and Indraratna and Redana (2000).
5.3 Muar clay, Malaysia
The Malaysian Highway Authority used different ground improvement techniques to the North South Expressway at Muar plain, Malaysia, to avoid excessive differential
settlement on the ground. The techniques applied in the field consisted of electro
osmosis, chemical injection, micro piles, sand compaction piles, pre-stressed spun
piles, well point preloading, vacuum preloading, sand wick installation, and
preloading with drains. These methods were tested at 14 different sites on Muar clay
floodplain. Most locations had marine and deltaic origins and contained thick
deposits of compressible soft soil. The section with conventional surcharge
preloading was used for comparison purposes.

The sub soil profile of Muar clay consisted of 2 m of weathered crust above a 16m
thick layer of soft to very soft silty clay, followed by a 1m layer of organic clay. It
also consists of medium dense to dense clayey silty sand below the organic clay,
which extends up to 24 m from the ground surface. The weathered clay was slightly
over-consolidated compared to normally consolidated soft clay. The water content
along with the Atterberg limits of soil, the over-consolidation ratio (OCR), and the
compressibility parameters with layer classification are shown in Figure 5.7. The unit
weight of soft clay was almost uniform (15-16 kN/m\(^3\)) except for the top layer of
weathered clay which was 17 kN/m\(^3\). The undrained shear strength of the sample
located at a depth of 3 m was 8 kPa, a value that increased linearly with depth.
Several in-situ and laboratory experiments were carried out to obtain the soil
parameters required for the analysis.
Figure 5.7 Sub-soil properties of Muar clay (modified after Ratnayake, 1991)

A 4.74m high embankment was constructed above the soft soil in two stages. The first construction stage consisted of raising the embankment to a height of 2.57 m and then adding fill until it reached 4.74 m. Band drains ($r_w=0.035$ m) were installed in a triangular pattern at 1.3 m spacing and drains were installed to a depth of 18 m. Instruments such as settlement plates, piezometers and inclinometers were installed to monitor the behaviour of the embankment. A cross section of this embankment stabilised with PVDs with the layout of the instrumentation is shown in Figure 5.8.
Table 5.5 Drain properties and strain rate dependency parameters for Muar clay

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$d_w$</th>
<th>$d_s$</th>
<th>$d_c$</th>
<th>$s$</th>
<th>$n$</th>
<th>$k_h/k_s$</th>
<th>$\mu$</th>
<th>$\frac{\sigma_{pl}}{\sigma_{p0}}$</th>
<th>$c_1$</th>
<th>$c_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>value</td>
<td>70.0</td>
<td>390</td>
<td>1365</td>
<td>5.57</td>
<td>19.5</td>
<td>3</td>
<td>5.649</td>
<td>0.79</td>
<td>0.887</td>
<td>0.158</td>
</tr>
</tbody>
</table>

Table 5.6 Soil parameters for Muar embankment (Ratnayake, 1991)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$e_0$</th>
<th>$C_v/(1+e_0)$</th>
<th>$C_p/(1+e_0)$</th>
<th>$\gamma_s$ (kN/m$^3$)</th>
<th>$p_{c(lab)}$’ (kPa)</th>
<th>$k_h$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2.0</td>
<td>3.10</td>
<td>0.38</td>
<td>0.038</td>
<td>16.5</td>
<td>95</td>
<td>$1.7 \times 10^{10}$</td>
</tr>
<tr>
<td>2-5.50</td>
<td>3.10</td>
<td>0.40</td>
<td>0.04</td>
<td>15.0</td>
<td>44</td>
<td>$1.6 \times 10^{10}$</td>
</tr>
<tr>
<td>5.50-8.0</td>
<td>3.06</td>
<td>0.28</td>
<td>0.028</td>
<td>15.5</td>
<td>60</td>
<td>$7.9 \times 10^{11}$</td>
</tr>
<tr>
<td>8.0-18.0</td>
<td>1.61</td>
<td>0.2</td>
<td>0.02</td>
<td>16.0</td>
<td>65</td>
<td>$3.4 \times 10^{11}$</td>
</tr>
</tbody>
</table>

The field strain rate was calculated as $6.78 \times 10^{-11}$ s$^{-1}$, from which a ratio of 0.79 (field pre consolidation pressure to laboratory pre consolidation ratio) was proposed.
and used for the current model analysis. Table 5.5 provides the drain and strain rate dependency ratio for converting the laboratory pre-consolidation pressure into field pre-consolidation pressure, while Table 5.6 presents the soil parameters for the sub-soil layer with the corresponding pre-consolidation pressures. Figure 5.9 shows the stages of construction, the time settlement curve, and the EPWP dissipation pattern of Muar clay comparing the current isotache model to those of Indraratna et al. (2005) and Indraratna and Redana (2000). As expected, the settlement predicted from the current model predicts the field data more accurately because it captures the creep settlement. The field data seems to be at odds for PWP dissipation in that the pattern of excess PWP dissipation is similar to an embankment without drains. The EPWP which remained after 300 days seems to be 65 kPa, which is a very uncommon reading from a piezometer. However, after converting the pre-consolidation pressure from the laboratory using the ratio 0.79, the EPWP dissipation curve using the current model provides more realistic results than Indraratna et al. (2005) and Indraratna and Redana (2000). The rate of dissipation is fastest for Indraratna and Redana (2000) followed by Indraratna et al. (2005). The current model predicts the slowest dissipation rate with 42 kPa of undissipated EPWP after 400 days.
Figure 5.9 Settlement and excess PWP dissipation of Muar embankment using Isotache model
5.4 Conclusion
Carrying out small scale specimens under laboratory conditions can predict settlement very well, but it is difficult to match the EPWP dissipation curve, especially for the given soft soils. The viscous behaviour of soil skeleton, which is also referred to as creep, inhibits EPWP dissipation due to the change in effective stress generated by delayed consolidation. In order to quantify EPWP closely, a radial consolidation model with isotaches could be used, because, it adopted the strain rate dependency of pre-consolidation pressure. This strain rate dependency can be quantified with the aid of a constant rate of strain (CRS) plus a long term consolidation laboratory test. Once the pre-consolidation pressure corresponding to the field strain rate is known, the change in effective stress due to delayed consolidation ($\Delta \sigma_d$) can be determined and the exact rate of EPWP dissipation could be evaluated for the corresponding strain rate. Moreover, additional settlement could also be found using variations of coefficient of secondary compression along with the strain rate.

The above method of determining settlement and EPWP dissipation was validated by three different case histories (Ballina bypass, SBIA, Bangkok and Muar clay, Malaysia). Conversion ratios of 0.86, 0.82, and 0.79 were used to convert the laboratory pre-consolidation pressure to field consolidation pressure for Ballina, SBIA, and Muar clay, respectively; the result obtained from this model indicates there is a good agreement with the field value. The result obtained from the current model was also compared with two other models (Indraratna et al. (2005) & Indraratna and Redana (2000)); the improvement in prediction could be seen using the current model. At the Ballina site on the Pacific highway, PWP began to dissipate after 100 days, even though settlement was still occurring. This could partly be due
to some instrumentation error, whereas with Muar clay, it seems likely that the piezometer tip was clogged so the EPWP dissipation trend was very gradual (similar to a case without drains). However, the use of an isotache model somehow improved the dissipation rate compared to pre-existing models following radial consolidation.
CHAPTER 6: CLASS A AND CLASS C PREDICTIONS OF EMBANKMENT BEHAVIOUR

[NOTE: Disclaimer: Part of this Chapter on Class A prediction has been published in the proceeding of Embankment Prediction Symposium (EPS) as “Indraratna, B., Baral, P., Rujikiatkamjorn, C. & Perera, D. (2016). Predictions Using (a) Industry Standard Soil Testing and, (b) Unconventional Large Diameter Specimens: A Designer’s Perspective of the Trial Embankment at Ballina. Embankment Prediction Symposium, Newcastle, Australia. 12-13 Sept, 2016.”, whereas additional Class C prediction has recently been submitted for the publication in Computer and Geotechnics as “Indraratna, B., Baral, P, Rujikiatkamjorn, C. & Perera, D. (2017). Class A and C Radial Consolidation Predictions Using Industry Soil Testing and, Large Diameter Specimens for Ballina Trial Embankment.” The analysis in this thesis has been performed independently from the paper using only publicly available data. There may be possibilities of having some discrepancies in actual field data and field data plotted in this chapter, which is due to the errors, arose from digitisation of the slides (Kelly et al., 2016).]

6.1 Introduction
Consolidation facilitated with prefabricated vertical drains (PVDs) has been regarded as one of the popular and economical ground improvement techniques promoting radial consolidation by reducing drainage path. Once soil is consolidated, the shear strength of soil is increased with reduced post-construction deformation. To study the various aspects affecting the soil consolidation behaviour, several analytical solutions based on the unit cell approach have been initially proposed. Barron (1948) derived classical axisymmetric solutions for radial consolidation considering
a constant permeability in the disturbed region of the soil adjoining the vertical drain (smear zone). This solution was derived based on the following assumptions:

(a) The soil is fully saturated.

(b) Only vertical compressive strain within the soil occurs uniformly.

(c) Permeability of the drain is significantly higher than that of the soil.

(d) The outer boundary of the unit cell is assumed to be circular.

(e) Linear Darcy’s law is valid and the small strain theory is adequate.

Afterward, Yoshikuni and Nakanodo (1974), Holtz et al. (1991), Hansbo (1981), Zeng and Xie (1989), Chai et al. (1997), Zhu and Yin (2004), Leo (2004), Indraratna et al. (2005), Walker and Indraratna (2009), Lu et al. (2011), Kianfar et al. (2013), and Lei et al. (2015) proposed various other solutions incorporating different assumptions along with boundary conditions. In addition, several researchers have conducted both 2D and 3D finite element modelling (e.g. Chai et al., 2001, Indraratna et al., 2005, Rujikiatkamjorn et al., 2008, Ye et al., 2012) to analyse an embankment facilitated with vertical drains. The results showed that both techniques can provide reasonable predictions depending on the embankment geometry, though 3D modelling used significantly computational effort compared to 2D plane strain modelling. In both cases, a fine mesh discretisation is usually required to generate individual vertical drain and adjacent smear zones representing a detailed multi-drain analysis.

6.2 Embankment characteristics and soil profiles at Ballina site

A 3m high embankment with a crest having dimensions of 80 m long × 15 m wide was constructed with a side slope 1.5H: 1V. Working platform of 95 m long × 25 m wide ×1 m high was placed on the existing ground surface to provide top drainage
and facilitate the drain installation. The embankment consists of three different sections: two sections, each having 30 m long consists of conventional PVDs (i.e. wick drain) and biodegradable drains (i.e. Jute drain); the third section having 20 m length consists of conventional PVD and with a geotextile layer instead of sand drainage layer. A suite of instruments including inclinometers, magnetic extensometers, settlement plates, vibrating wire piezometers and hydraulic profile gauges were installed to monitor the embankment behaviour. The staged construction of embankment was completed in 60 days. The compacted bulk density of the sand drainage layer (0.6 - 1m thick) was 15.9kN/m$^3$ whereas the compacted density of the above fill was 20.6kN/m$^3$, resulting in a total surcharge load of 59.8kN/m$^3$. Vertical drains were installed to 14-15m deep using a rectangular mandrel having cross-section of 120 mm x 60 mm via a static approach from an 80t drain stitcher. Rectangular plates (190 mm × 90 mm) were attached the tip of madrigal as drain anchors while installing the drains in a square pattern at 1.2m spacing. The layout of the Ballina embankment is shown in Fig. 6.1

![Figure 6.1 Layout of embankment](image)

According to the geological survey (1:250,000 Moreton Map, NSW), the Ballina flood plain is composed of Holocene sediments of low strength and high
compressible silty clay with sea shell and natural sand partings, which is a typical characteristics of estuarine deposits.

Figure 6.2 Basic soil properties of Ballina clay (modified after Pineda et al., 2016)

Figure 6.2 summarizes the consolidation and basic soil properties at the site, as provided by Pineda et al. (2016). The sub-soil profile consisted of an approximately 0.2m thick layer of organic material (decomposing sugar cane plants), below which a layer of sandy clayey silt of 1m thick was found, followed by highly plastic silty clay of 8.8 m thick. There was a 4m thick transition zone under this layer which consists of an increasing content of sand, followed by a 5m thick layer of fine sand. This deposit can be classified as highly compressible marine clay with very low permeability and high plasticity (CH). At most depths of the upper Holocene layer, the natural water content was very close to their liquid limits.
6.3 Prediction exercise

Predicting embankment behaviour is very important exercise in practice in order to confirm and correct the design parameters. Prof Lambe (Lambe, 1973) divided the types of prediction based on the availability of the data.

a) **Class A**: This prediction is performed before the construction with the help of site investigation data and soil properties.

b) **Class B**: This prediction is conducted during the construction and may be influenced by initial field data.

c) **Class C**: This prediction is made after the construction event when field data are accessible. It may include back calculation of field results using curve matching techniques.

In this Chapter, both **Class A** and **Class C** consolidation analyses are performed incorporating the soil properties obtained from:

(a) laboratory testing using large diameter samples (Indraratna et al., 2016),

(b) field data obtained for characterising the smear zone (Indraratna et al., 2015) and,

(c) previous Pacific Highway embankment works in Ballina where the field data have been used to interpret soft soil embankment behaviour (Kelly, 2008, RTA, 2009 and Indraratna et al., 2012).

It is important to note that the original laboratory and site investigation data by industry standards alone are not sufficient to obtain the most accurate predictions, because in the field, the mandrel-driven PVDs cause considerable smear and soil destructuration which require further investigation to characterise the alterations to the original soil properties especially the permeability and compressibility parameters.

Six different cases (Cases A-F) were performed including both **Class A and Class C** prediction of embankment and the details of these predictions are given below:
CASE A (Class A prediction): This method of Class A prediction was based on the radial consolidation properties obtained from large diameter samples of undisturbed Ballina soil specimen using a large-scale consolidation chamber (350mm diameter x 700mm high) under an applied surcharge of 80kPa (Fig 6.3). Both the prefabricated vertical drain (PVD) and the mandrel were scaled down for the appropriate unit cell simulated herein in relation to the field drain spacing of 1.2m to mimic exact field condition. The scaled-down synthetic drain (25 mm in width) was pushed through the centre of the soil sample with the aid of the steel mandrel.

![Large-scale consolidometer](modified after Indraratna and Redana, 1998)

Two pore water pressure transducers (one at the centre and another at a distance of 96mm from the centre) were installed at the base of consolidometer to capture the
change in pore water pressure and to measure the coefficient of horizontal consolidation \((c_h)\). Based on the pore water pressure measurements in between two transducers, a relationship of the seepage velocity \((v)\) against the hydraulic gradient \((i)\) was plotted and shown in Fig. 6.4. A non-linear deviation from the conventional linear Darcy’s law, was observed (also reported by Kianfar et al., 2013), the two specific power law constants \(\alpha_c\) and \(\beta\) were evaluated as \(5.3 \times 10^{-10}\) m/s and 1.28, respectively. The extent of the smear zone radius was found to be about 7.5 times the equivalent radius of PVD, which is significantly greater than what were observed from past laboratory studies for fully remoulded clays. The corresponding soil parameters used for this case are shown in Table 6.1.

Figure 6.4 Radial flow characteristics using large-scale consolidometer
Table 6.1 Soil properties based on large specimen incorporating non-Darcian consolidation (Case A)

<table>
<thead>
<tr>
<th>Thickness (m)</th>
<th>$m_v$ (m$^2$/kN)</th>
<th>$\alpha_v \times 10^{-10}$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.7</td>
<td>0.00225</td>
<td>5.28</td>
</tr>
<tr>
<td>3</td>
<td>0.00212</td>
<td>4.95</td>
</tr>
<tr>
<td>3</td>
<td>0.00038</td>
<td>4.86</td>
</tr>
<tr>
<td>3</td>
<td>0.00163</td>
<td>5.45</td>
</tr>
<tr>
<td>3.3</td>
<td>0.00186</td>
<td>6.16</td>
</tr>
</tbody>
</table>

**CASE B (Class A prediction):** The effect of soil disturbance caused by mandrel driving with variations of permeability and compressibility within the smear zone (soil de-structuration) was captured in this model (Perera et al., 2016). The available industry data in Ballina obtained from previous Pacific Highway embankment report (e.g., Kelly, 2008, RTA, 2009 and Indraratna et al., 2012) were used. The corresponding soil properties are shown in Table 6.2. The details formulation of this model can be found in Appendix B.

Table 6.2 Industry standard soil properties for Case B.

<table>
<thead>
<tr>
<th>Thickness (m)</th>
<th>Pav</th>
<th>Recompression</th>
<th>Compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.7</td>
<td>9.3</td>
<td>58.6</td>
<td>40.9</td>
</tr>
<tr>
<td>3</td>
<td>67.9</td>
<td>52.4</td>
<td>23.9</td>
</tr>
<tr>
<td>3</td>
<td>36.7</td>
<td>46.4</td>
<td>38.7</td>
</tr>
<tr>
<td>3</td>
<td>50.8</td>
<td>39.3</td>
<td>44.6</td>
</tr>
<tr>
<td>3.3</td>
<td>65.6</td>
<td>57.6</td>
<td>57.6</td>
</tr>
</tbody>
</table>
**CASE C (Class C prediction):** An elastic visco-plastic model (EVP) using finite difference method (FDM), developed for radial consolidation, was used in this prediction analysis so as to compute the settlements and excess pore water pressures dissipation, the details of which have been described in Chapter 3. The corresponding soil parameters are shown in Table 6.3.

<table>
<thead>
<tr>
<th>Thickness (m)</th>
<th>κ/ν</th>
<th>λ/ ν</th>
<th>ψ/ ν</th>
<th>e₀</th>
<th>αₑ (m/s)</th>
<th>pₑ'</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.7</td>
<td>0.034</td>
<td>0.148</td>
<td>0.003</td>
<td>3.1</td>
<td>5.28×10⁻¹⁰</td>
<td>46.3</td>
</tr>
<tr>
<td>3.0</td>
<td>0.062</td>
<td>0.156</td>
<td>0.004</td>
<td>2.8</td>
<td>4.95×10⁻¹⁰</td>
<td>27.2</td>
</tr>
<tr>
<td>3.0</td>
<td>0.098</td>
<td>0.192</td>
<td>0.004</td>
<td>2.8</td>
<td>4.86×10⁻¹⁰</td>
<td>44.1</td>
</tr>
<tr>
<td>3.0</td>
<td>0.107</td>
<td>0.138</td>
<td>0.004</td>
<td>2.8</td>
<td>5.45×10⁻¹⁰</td>
<td>50.8</td>
</tr>
<tr>
<td>3.3</td>
<td>0.047</td>
<td>0.180</td>
<td>0.003</td>
<td>2.7</td>
<td>6.16×10⁻¹⁰</td>
<td>65.6</td>
</tr>
</tbody>
</table>

**CASE D (Class A prediction):** This case was based on 2D plane strain multi drain analysis using Soft Soil Model (Vermeer and Neher, 1999). The equivalent permeability proposed by Indraratna and Redana (1997) was adopted. The water table was assumed to be at the ground surface. PVDs of appropriate length (i.e. 15 m) were modelled using a drain element (Brinkgreve et al., 2015). A 6-noded 2D triangular element was used in the plane strain analysis. The total number of elements and nodes were found to be 11096 and 17096, respectively. Mesh discretisation for this prediction approach is shown in Fig. 6.5. Settlement, excess pore water pressure as well as lateral deformation was planned as output and briefly discussed in field result section.
CASE E (Class A prediction): This case was based on 3D modelling of Ballina embankment using Soft Soil Model (Vermeer and Neher, 1999). Similar to the plane strain analysis, the water table was assumed to be at the ground surface. PVDs of appropriate length were modelled using a drain element (Brinkgreve et al., 2015). A 10-noded tetrahedron 3D soil element was used in this analysis. A total of 170432 elements and 229533 nodes formed the mesh discretisation for the 3D simulations. A mesh discretisation for 3D analysis is shown in Fig. 6.6 Similar to the plane strain analysis, settlement, excess pore water pressure as well as lateral deformation was planned as output and briefly discussed in field result section.
**CASE F (Class C prediction):** This case involves the analysis of Ballina embankment using 2D plane strain with Soft Soil Creep (SSC) Model (Vermeer and Neher, 1999). The detail of finite element modelling is similar to Case D. The material properties used for all these multi-drain analysis are tabulated in Table 6.4
Table 6.4 Material Parameters used for multi drain analysis

<table>
<thead>
<tr>
<th>Parameter(s)</th>
<th>Embankment</th>
<th>Sand blanket</th>
<th>Layer1</th>
<th>Layer2</th>
<th>Layer3</th>
<th>Layer4</th>
<th>Layer5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer thickness (m)</td>
<td></td>
<td></td>
<td>2.7</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.3</td>
</tr>
<tr>
<td>Material model</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\gamma_{\text{unsat}}$ (kN/m$^3$)</td>
<td></td>
<td></td>
<td>20.6</td>
<td>15.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$\gamma'_{\text{sat}}$ (kN/m$^3$)</td>
<td></td>
<td></td>
<td>20.6</td>
<td>15.9</td>
<td>14.5</td>
<td>13.7</td>
<td>14.2</td>
</tr>
<tr>
<td>$e_{\text{init}}$</td>
<td></td>
<td></td>
<td>0.5</td>
<td>0.5</td>
<td>3.1</td>
<td>2.8</td>
<td>2.8</td>
</tr>
<tr>
<td>$E_{50}^\text{ref}$ (kN/m$^2$)</td>
<td></td>
<td></td>
<td>5000</td>
<td>6000</td>
<td>1000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_{\text{oed}}^\text{ref}$ (kN/m$^2$)</td>
<td></td>
<td></td>
<td>3435</td>
<td>4328</td>
<td>434.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_{\text{ur}}^\text{ref}$ (kN/m$^2$)</td>
<td></td>
<td></td>
<td>15000</td>
<td>18000</td>
<td>3000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\lambda^*$</td>
<td></td>
<td></td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\kappa'$</td>
<td></td>
<td></td>
<td>0.148</td>
<td>0.156</td>
<td>0.192</td>
<td>0.138</td>
<td>0.180</td>
</tr>
<tr>
<td>$e_{r}$ (kN/m$^3$)</td>
<td>1</td>
<td>0.025</td>
<td>2.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>4.0</td>
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<tr>
<td>$\phi'$ (kN/m$^3$)</td>
<td>32.0</td>
<td>35.0</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>$k_{x,}$ (m/day)</td>
<td>0.65</td>
<td>1.30</td>
<td>1.24x10^{-4}</td>
<td>1.04x10^{-4}</td>
<td>1.04x10^{-4}</td>
<td>1.04x10^{-4}</td>
<td>1.28x10^{-4}</td>
</tr>
<tr>
<td>$k_{z,}$ (m/day)</td>
<td>0.65</td>
<td>1.30</td>
<td>0.62x10^{-4}</td>
<td>0.52x10^{-4}</td>
<td>0.52x10^{-4}</td>
<td>0.52x10^{-4}</td>
<td>0.64x10^{-4}</td>
</tr>
<tr>
<td>OCR</td>
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<td></td>
<td>5.0</td>
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<td>1.2</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$k_{h,ps}$ (m/day)</td>
<td></td>
<td></td>
<td>3.29x10^{-5}</td>
<td>2.76x10^{-5}</td>
<td>2.76x10^{-5}</td>
<td>2.76x10^{-5}</td>
<td>3.40x10^{-5}</td>
</tr>
<tr>
<td>$\mu^*$</td>
<td></td>
<td></td>
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<td>0.004</td>
<td>0.004</td>
<td>0.004</td>
<td>0.003</td>
</tr>
</tbody>
</table>

6.4 Lateral deformation

Several analytical approaches and finite element method analysis were used to predict the behaviour of lateral displacement. *Class A* predictions on lateral deformation were performed on the available formulations by past literatures as well as performing 2D and 3D finite element analysis. Improvement on *Class A* predictions were carried out by applying soft soil creep model in finite element
analysis using plane strain model as *Class C* prediction. All the methods of prediction are described herein:

**Stability Factor Method:** Indraratna et al. (1997b) reported three different stability factors \((\alpha, \beta_1 \& \beta_2)\) based on 4.75 m high Muar clay embankment in Malaysia and recommended their values as 0.123, 0.034 and 0.274 respectively. where,

\[\alpha\] is a ratio of maximum lateral displacement at the toe to the maximum settlement at the centreline \((\beta_1/\beta_2)\);

\[\beta_1\] is a ratio of maximum lateral displacement to the corresponding fill height;

\[\beta_2\] is a ratio of maximum settlement to the corresponding fill height.

**Lower and Upper bound method:** Chai et al. (2013) introduced the term NLD (normalized lateral displacement) and RLS (ratio of index pressure to representative shear strength) in order to predict the lateral displacement associated with embankment loading. Subsequently, Xu and Chai (2014) established a co-relationship between NLD and RLS based on different embankments (case histories) as:

\[
NLD = 0.067 \times RLS + 0.11 \quad (0.05 < RLS < 3.0) \pm 0.05
\]

Based on the above relationship, lower bound and upper bound for the lateral deformation of Ballina embankment were found out and plotted in Fig. The distribution of vertical stress followed the method of Osterberg (1957).

**Tavenas approach:** Tavenas et al. (1979) proposed a relationship between the normalized depth of soft soil \((Z=z/D)\) and the normalized lateral deformation \((Y=y/y_m)\) in order to predict the lateral deformation and the proposed equation is stated as:
\[ Y = 1.78Z^3 - 4.7Z^2 + 2.21Z + 0.71 \]  \hspace{1cm} (6.2)

In the above,

D = depth of soft soil and,

\( y_m \) = the maximum lateral deformation, which is the function of maximum settlement at the centreline of the embankment.

6.5 Field results and discussion

In this section, the measured settlement, excess pore water pressure and lateral deformation (extracted from Slides, Kelly et al., 2016) are compared with different prediction approaches. The staged construction, surface settlement and excess pore water pressure at 6m depth from the ground surface close to the centreline of embankment is shown in Fig. 6.7. The predicted settlements by all methods (Case A-F) are acceptably close to the field result. Class C prediction performed using an EVP-FDM model developed in Chapter 3 (i.e. Case C) and multi-drain analysis of embankment using 2D plane strain FEM with soft soil creep (SSC) model yields higher long term settlement (Fig. 6.7b). Compared to other prediction approaches, Case C and Case F account for creep effect but it is absence in other predictions due to the absence of creep related data in past industry reports. Similarly, Figure 6.7c presents the excess pore water pressure dissipation curve along with the field data. All of the Class A predictions are unable to predict the retarded excess pore water dissipation as it under-predicts the field data whereas both of the Class C predictions (Case C and Case F) can produce better match to the field data. However, the use of EVP-FDM model (Case C) seems to give better prediction compared to others although the dissipation rate is much higher than the field observation. In overall, the
inclusion of viscous effect (Case C and Case F) has significantly improved the accuracy of the excess pore water pressure dissipation.

In terms of lateral displacement, Figure 6.8 compares lateral deformation of the embankment after 3 years using various analytical and numerical approaches including stability factor method (Indraratna et al., 1997a), Upper and lower bound method (Xu and Chai, 2014), Tavenas approach (Tavenas et al., 1979) and finite element analyses (Case D, E & F). It can be seen that the observed lateral displacement is maximum at a depth of 5-6 m where the softest upper Holocene layer is located. All approaches are unable to match the lateral deformation in the upper layer of soil approaching the surface. Furthermore, it can be found that there still exist some variation in between field data and prediction approaches in the deeper region of clay layer and uppermost crust which cannot be interpreted at this stage. In relation to the Class A predictions, the method using the stability factor method and lower bound of Xu and Chai (2014) under predict the lateral deformation in upper clay layers whereas better match is found for deeper clay layers. Similarly, Class C prediction in Case F (plane strain analysis with soft soil creep model) are very close to the field measurement. Comparing FEM analysis in terms of lateral deformation, it is found that the lateral deformation obtained using Case D FEM plane strain model is at least 40 % greater at 2-6 m deep clay layers than that predicted from Case E 3D modelling. This is not unexpected, because in 2D plane strain model, a zero strain is prescribed in the longitudinal direction enabling an increased strain in transverse direction.
Figure 6.7 Prediction analyses: (a) stage construction (b) settlements and (c) excess pore pressures.
A prefabricated vertical drain combined with surcharge preloading is one of the effective ground improvement methods for accelerating soft soil consolidation. This chapter elaborates the prediction methods (both Class A and Class C) used in predicting the behaviour of Ballina trial embankment situated on soft estuarine clay.
foundation. The prediction methods mainly consist of available analytical method (including current EVP-FDM model as described in Chapter 3) and numerical tools (PLAXIS 2D and 3D). Both unit cell approaches as well as multi-drain analyses were performed using the geotechnical properties available from past published work (Industry standard testing reports and well-controlled experimental testing on both conventional and large scale undisturbed soil specimens). The smear zone was adequately characterized using the undisturbed specimen at various radial distances from vertical drain from the Ballina site.

Six different cases on predictions based on unit cell analysis as well as multi drain analysis were considered in making these Class A and Class C predictions. The unit cell analysis consisted of non-Darcian flow using large scale consolidometer (Case A), non-linear variation of soil compressibility and permeability including de-structuration effect (Case B) and, currently developed elastic, visco-plastic (EVP) – finite difference method (Case C), whereas the multi-drain analyses consisted of 2D plane strain FEM (Case D), 3D FEM (Case E) and, 2D plane strain model with soft soil creep (Case F). It was observed that the centreline settlement predictions agreed generally well with the field data (Class A), but the ultimate settlement could only be matched after the inclusion of visco-plastic (creep) behaviour in the analytical and numerical methods. Similarly In case of excess pore water pressure, all of the Class A predictions were unable to predict the field measurement. However, the use of visco-plastic behaviour in Class C prediction improved the prediction significantly albeit the measured excess pore pressures remaining at significantly higher levels than the predicted values, especially after 1-1.5 years.
The comparisons show that the empirical formulations, analytical models and numerical analyses presented here still require further refinement to accurately predict the lateral displacement profile, especially nearing the surface. Not surprisingly, the maximum lateral displacement was observed at a depth of 5-6 m in the softest clay layer. In general, the observed lateral displacements tend to agree reasonably well with the computed values based on the 2D FEM plane strain analysis with soft soil creep (Class C-Case F) in the upper soft clay (0-5m). It was also found that accurate estimation of lateral deformation profile introduced by Xu and Chai (2014) could be used in confidence using NLD-RLS relationship.
CHAPTER 7- CONCLUSIONS AND RECOMMENDATIONS

7.1 General
The main aim of this study was to investigate the elastic visco-plastic behaviour of soft soil through analytical modelling with special reference to radial consolidation. A comprehensive literature study and introduction to the topic along, with scope and objectives were presented in Chapters 1 and 2. The development of analytical, numerical, and experimental modelling through time was described briefly in Chapter 2. Chapter 3 presented analytical modelling for the visco-plastic behaviour of soft soil based on two different approaches: 1) Using the time-equivalent concept combined with Yin and Graham’s visco-plastic model and non-Darcian flow and, 2) Using the strain rate dependency of pre-consolidation pressure through the isotache concept. Extensive laboratory work which including a small scale Rowe cell specimen and the long-term consolidation of a 350 mm specimen of undisturbed Ballina clay in a large scale consolidometer was presented and validated in Chapter 4. Chapter 5 included the validation of a novel model (Isotache concept) with respect to three different case histories (Ballina Bypass, Suwarnabhumi International Airport, Bangkok and North-South Expressway, Malaysia (Muar)). Similarly, Chapter 6 validated the EVP model (time equivalent concept combined with Yin and Graham’s visco-plastic model) using the current Ballina trial embankment (CGSE) as a Class C prediction approach.

7.2 Salient features of analytical model
There are two analytical methods for radial consolidation discussed in Chapter 3; one is based on Bjerrum’s time-equivalent concept adopting Yin and Graham’s elastic visco-plastic parameters, whereas other model is based on the isotache based radial
consolidation model which considers the strain rate dependency of pre-consolidation pressure.

- The elastic visco-plastic (EVP) model captured the non-Darcian fluid flow in radial consolidation along with vacuum preloading based on Yin and Graham’s EVP parameters and Bjerrum’s time equivalent concept.
- A two-step finite difference method known as Peaceman-Rachford (P-R) was applied to the governing equations of radial consolidation to make the solution more accurate by performing the calculations consecutively using predictor and corrector.
- A constant rate of strain (CRS) and long term consolidation tests were needed to obtain the strain rate dependency of pre-consolidation pressure, and these dependency parameters for several types of clay all around the world were plotted along with the case studies taken from Ballina, Bangkok and Malaysia.
- The isotache model based on strain rate dependency of pre-consolidation pressure can model delayed consolidation, as well as the retardation of excess pore water pressure in the field, when considering radial consolidation by PVD.
- Creep settlement in the isotache model can be determined by considering variations of the coefficient of secondary consolidation ($C_{ao}$) with the relevant strain rate, by plotting the integrated fitting curve for individual clay in strain rate dependency of pre-consolidation plot.
7.3 Experimental work
Small scale specimens using a remoulded sample were tested using the Rowe cell, and large scale consolidometers were used to test a large scale undisturbed specimens. Both samples were extracted from Ballina, NSW and the following conclusions can be drawn from this experimental work:

- Small scale Rowe cell specimens obtained from Ballina were tested using a modified Rowe cell for different vacuum-surcharge ratio (VSR), so that it could capture the lateral deformation when vacuum pressure is applied. The axial strain obtained from linear variable differential transformer (LVDT) and lateral strain obtained from the modified Rowe cell (via a membrane inserted between the wall of the cell and soil sample) were combined to determine the volumetric strain; it is the same for all the VSR. The ultimate volumetric strain for Ballina clay was 25.5%.

- The degree of consolidation (DOC) based on settlement and pore water pressure was plotted for all VSR and it was found that the DOC obtained from settlement always over predicts the DOC obtained from pore water pressure readings.

- Large scale undisturbed specimens (350 mm diameter) were extracted from the field site with a corer with minimum disturbance, and then a large scale consolidometer test was carried out in the UOW laboratory to model the exact behaviour of soil in the field, especially considering the fact that soils in a marine environment contain random coarse particles such as gravel and shells, as well as natural partings.

- The smear effect factor, which is very useful in converting laboratory properties into real field properties, was obtained from the large scale
consolidometer using a graphical plot between normalised reduction in the water content \( ((W_{\text{max}} - W)/W_{\text{max}}) \) and the ratio of the equivalent radius to the mandrel radius \( (r/r_m) \). Here the radius of the smear zone can be almost 5 times larger than the radius of the mandrel, \( (r/r_m=5) \) for large undisturbed samples and for the field case (both single and multi-drain), but this ratio was twice that obtained for the small laboratory specimens \( (r/r_m=2.5) \). In this respect, soil within the equivalent diameter region can be divided into three different zones: (a) the smear zone \( (0 \leq r/r_m \leq 5) \) (b) the marginally disturbed zone \( (5 < r/r_m \leq 6.5) \) (c) the undisturbed zone \( ((6.5 < r/r_m \leq r_e)) \).

- Three large scale consolidometer tests were carried out to evaluate the efficiency of PVD and vacuum preloading over conventional one-dimensional vertical consolidation; it was found that the radial consolidation with a vacuum is the fastest, followed by radial consolidation facilitated with vertical drains, and then 1-D vertical consolidation. These responses were noted by analysing the settlement and the excess pore water pressure dissipation curve.

7.4 Validation of analytical models
A Class C prediction for the Ballina trial embankment (CGSE) was carried out using an Elastic visco-plastic model with non-Darcian flow; the details were described in Chapter 6. The following conclusions can be drawn from these case studies:

- For the unit cell (single drain) analysis, approaches such as non-Darcian flow, non-linear variation of soil compressibility and soil permeability and elastic, visco-plastic properties were considered in making these Class A and Class C predictions. Note that the centreline settlement predictions generally agreed
well with the field data (*Class A*), but the ultimate settlement could only be matched correctly when visco-plastic (creep) behaviour was captured in the analytical and numerical methods.

- The build-up of excess pore water pressure after construction loading, including its peak and initial dissipation trend, could be predicted reasonably well once the visco-plastic nature of the soft clay was incorporated (Case A and Case F).

- For multi-drain analyses, both 2D plane strain with permeability conversion and true 3D analyses were carried out. The settlement predictions agreed well with the field data for both cases, albeit the excess pore pressures remained at much higher levels than the predicted values, especially after 1-1.5 years (Case F).

- The comparisons on lateral deformation showed that the empirical formulations, analytical models and numerical analyses presented used in this study still require further refinement to accurately predict the lateral displacement profile, especially nearing the surface. Not surprisingly, the maximum lateral displacement was observed at a depth of 5-6 m in the softest clay layer. In general, the observed lateral displacements tend to agree reasonably well with the computed values based on the 2D FEM plane strain analysis with soft soil creep (*Class C*-Case F) in the upper soft clay (0-5m). It was also found that the empirical NLD-RLS relationship introduced by Xu and Chai (2014) could be used in confidence to accurately estimate the lateral deformation profile.
• In a similar way, case studies of the Ballina bypass, Pacific highway; the Second Bangkok International Airport, Thailand; and the North-South Expressway, Muar plain, Malaysia were used to validate the results obtained from the analytical model derived using isotaches with the strain rate dependency of pre-consolidation pressure. All of these case histories where the isotache model was used for validation had a better match in the field in terms of settlement and excess pore water pressure. These results were then compared with the pre-existing model and they indicated that the isotache model could describe the retarded excess PWP dissipation better than the others. For the Muar embankment, the trend of excess PWP dissipation was similar to the EPWP dissipation of the embankment without drains (possibly the drain was completely clogged). Furthermore, the EPWP dissipation curve from isotache model was better than the other models in terms of settlement and pore pressure prediction.

7.5 Numerical simulation
Various numerical simulations were carried out in this study using Finite element packages PLAXIS 2D & 3D. PLAXIS 2D with the soft soil creep model and unit cell approach were used to compare the laboratory results, whereas the 2D plane strain and 3D model were used to model the Ballina trial embankment using multi-drain analysis. The following conclusions can be drawn from the numerical simulation:

• The behaviour of test specimens in the large scale consolidometers was validated using the PLAXIS 2D finite element analysis with a unit cell approach. Three different zones (smeared, marginally disturbed, and undisturbed) were defined using the normalised water content approach, and
their corresponding values of permeability were then used as input for the FEM simulation. The soft soil creep model was used to model the unit cell and the results in terms of pore water pressure and settlement agree with the laboratory data. Furthermore, the validation of two different analytical models described in Chapter 3 were used and there exist good agreement of analytical model and numerical simulation with laboratory data.

- PLAXIS 2D and 3D with soft soil model and soft soil creep model were used to model the Ballina trial embankment as a Class A and Class C prediction approach; it was found that 2D plane strain analysis always over-predicts settlement compared to the 3D model, and the effect of retarded pore water pressure can be modelled readily using the soft soil creep model.

7.6 Recommendation for future researchers
The study of the elastic visco-plastic behaviour of soft soil with reference to radial consolidation via analytical and numerical modelling, laboratory experiments and case history validation has resulted in a novel isotache method that can predict the behaviour of soft soil in terms of settlement and excess pore water pressure. However, that following recommendations can be considered by future researchers on the basis of this study:

- The current research is completely based on an analytical model and laboratory studies. It would be appropriate to model the Rowe cell and the large scale consolidometers numerically in order to compare the results with analytical and laboratory afterwards.
- Samples should be extracted from different depths to investigate the smear effects and compare the results with the current study and available literature.
Most of the analytical models proposed until now do not consider lateral strains in a radial consolidation model; it would be more realistic to consider analysing a radial consolidation model with lateral strains.
Note

Appendix A is under permanent embargo and has been removed from the thesis
APPENDIX B SUMMARY OF MATHEMATICAL FUNCTIONS
CASE A: Kianfar et al. (2013) presented a radial consolidation model to capture non-linear relationship between the flow velocity and the hydraulic gradient (non-Darcian law). They also consider the non-linear relationship of soil compressibility and permeability with the void ratio. The average excess pore water pressure ($\bar{u}$) can then be determined as:

$$\bar{u} = \left(1 - \beta\right) \left(-\frac{2\alpha c}{m_v} \left[\frac{n^2-1}{2n^2} \eta y_w\right] \right)^\beta t + (\bar{u}_0)^{(1-\beta)} \right)^{\frac{1}{1-\beta}}, \quad \text{(A.1)}$$

and,

$$\varepsilon = -m_v \left(1 - \beta\right) \left(-\frac{2\alpha c}{m_v} \left[\frac{n^2-1}{2n^2} \eta y_w\right] \right)^\beta t + (\bar{u}_0)^{(1-\beta)} \right) - (\bar{u}_0) \right)^{\frac{1}{1-\beta}}, \quad \text{(A.2)}$$

$$n = \frac{\bar{u}}{r_w} \quad \text{(A.3)}$$

$$\eta_n = \sum_{i=0}^{\infty} \left(\frac{1}{\beta} \right) \left(\frac{1}{i}\right)^i \left(-1\right)^i \left(\frac{1}{\beta-1}\right) \left[r^j + (c_a - 1) r s^j - c a r s^j \right] \quad \text{(A.4)}$$

$$c_a = \frac{k}{k_s} \quad \text{(A.5)}$$

where $\bar{u}$ = average excess pore water pressure in the unit cell,

$r_w$ and $r_s$ = the radii of the drain and smear zone, respectively ,

$\bar{u}_0$ = initial average excess pore water pressure in the unit cell,

$m_v$ = coefficient of the soil volume compressibility,

$k_s'$ and $\beta$ = constants which depend on the type of soil and flow relationship in the smear zone and,

$\gamma_w$ = unit weight of water.

CASE B: Perera et al. (2016) captured the effect of soil disturbance caused by mandrel driving including the variations of permeability and compressibility in the smear zone, and the corresponding role of void ratio, thus improving on the original
derivations of Indraratna and Redana (1997) & Indraratna and Redana (2000). The excess pore water pressure ratio at any time \( t \) at depth \( z \) can be determined as follows:

\[
R_u = \left( \frac{u_0}{\Delta \sigma'} \right) \times \exp \left\{ \left( \frac{\sigma'_y}{\sigma_0'} \right)^{1-\frac{c_c}{c_s}} + 1 \right\} \frac{4T_{ho}}{\mu}, \quad \bar{\sigma}' \leq \bar{\sigma}'_{ho} , \quad t \leq t_i \tag{A.6}
\]

\[
R_u = \left( \frac{(\sigma'_y + u_0 - \sigma'_y)}{\Delta \sigma'} \right) \times \exp \left\{ \left( \frac{\sigma'_y + \Delta \sigma'}{\sigma'_y} \right)^{1-\frac{c_c}{c_s}} + 1 \right\} \frac{4T_{hi}}{\mu}; \quad \bar{\sigma}' > \bar{\sigma}'_{ho}; \quad t > t_i \tag{A.7}
\]

\[
t_i = \frac{\mu d^2}{4c_ho} \left[ \frac{\sigma'_y^{1-\frac{c_c}{c_s}}}{\sigma_0'} + 1 \right] \ln \left( \frac{\frac{u_0}{\sigma_0' + u_0 - \sigma'_y}}{\frac{u_0}{\sigma'_y}} \right) \tag{A.8}
\]

\[
\mu = \ln \left( \frac{a}{s} \right) - \frac{3}{4} + \frac{k(s-1)}{s-k} \ln \left( \frac{s}{\sigma} \right) \tag{A.9}
\]

\[
T_{ho} = P_{av,0}T_{ho} = 0.5 \left( \frac{\sigma'_y}{\sigma_0'} \right)^{1-\frac{c_c}{c_s}} + 1 \right\} T_{ho}; \quad \bar{\sigma}' \leq \bar{\sigma}'_{ho} \tag{A.10}
\]

\[
P_{av,0} = 0.5 \left( \frac{\sigma'_y}{\sigma_0'} \right)^{1-\frac{c_c}{c_s}} + 1 \right\}; \quad \bar{\sigma}' \leq \bar{\sigma}'_{ho} \tag{A.11}
\]

\[
T_{hi} = P_{av,i}T_{hi} = 0.5 \left( \frac{\sigma'_y + \Delta \sigma'}{\sigma'_y} \right)^{1-\frac{c_c}{c_s}} + 1 \right\} T_{hi}; \quad \bar{\sigma}' > \bar{\sigma}'_{ho} \tag{A.12}
\]

\[
P = P_{av,i} = 0.5 \left( \frac{\sigma'_y + \Delta \sigma'}{\sigma'_y} \right)^{1-\frac{c_c}{c_s}} + 1 \right\}; \quad \bar{\sigma}' > \bar{\sigma}'_{ho} \tag{A.13}
\]

where \( R_u \) is the excess pore water pressure ratio, \( c_c \) is the average compression index for a given stress range in a normally consolidated region, and \( c_s \) is the recompression index in the over-consolidation region, \( c_k \) is the permeability index, \( \sigma'_y \) is pre-consolidation stress (yield stress) of the average curve, \( \sigma'_0 \) is effective vertical stress at initial stage, \( u_0 \) is excess pore water pressure, \( \Delta \sigma' \) is total effective stress change, \( t_i \) is the time required for soil to change from an over-consolidated state into a normally-consolidated state.
APPENDIX C: LIST OF NOTATION
\( \bar{C}_c \) average compression index

\( C_c \) Compressibility index

\( C_h \) coefficient of radial consolidation

\( C_k \) permeability index

\( C_r \) Re-compression index

\( C_s \) recompression index

\( C_v \) coefficient of vertical consolidation

\( e_0 \) initial void ratio

\( d_w \) diameter of drain

\( d_s \) diameter of smeared zone

\( D_e \) equivalent diameter of the influence zone

\( \bar{\varepsilon}_0 \) average initial void ratio

\( k_x \) permeability in smear zone

\( k_h \) horizontal coefficient of permeability for axisymmetry in undisturbed zone

\( k_h' \) horizontal coefficient of permeability for axisymmetry in smear zone

\( k_{h,ps} \) horizontal coefficient of permeability for plane strain in undisturbed zone

\( k_{h,ps}' \) horizontal coefficient of permeability for plane strain in smear zone

\( R \) radius of axisymmetric unit cell

\( r_s \) radius of smear zone

\( r_w \) radius of vertical drain

\( n \) ratio of equivalent diameter of soil cylinder to drain diameter

\( s \) ratio of smeared diameter to drain diameter

\( l \) length of vertical drain

\( LL \) liquid limit

\( PL \) plastic limit

\( w_n \) natural water content

\( PSD \) particle size distribution

\( m_v \) coefficient of volume compressibility
\( \gamma \)  unit weight of soil

\( P_{ov} \)  constant as a function of over consolidation ratio

\( P, P_0 \)  vacuum pressure (negative)

\( Q \)  fill surcharge loading

\( q_w \)  discharge capacity

\( R_u \)  Excess pore pressure ratio

\( t_i \)  time (when \( \sigma' = \bar{\sigma}_{vt} \))

\( T_{ho}, T_{bo} \)  dimension-less time factor

\( u_{t_i,j} \)  pore water pressure at i,j co-ordinate at time t.

\( C^I, C^{III} \)  parameters used in P-R FDI analysis.

\( \Delta T_r \)  constant used in EVP analysis in r-direction

\( \Delta T_v \)  constant used in EVP analysis in v-direction

\( V \)  specific volume

\( \lambda \)  slope of reference time line

\( \kappa \)  slope of instant time line

\( \psi \)  slope of fitted creep curve

\( g(u, \varepsilon_z) \)  parameter used during EVP analysis

\( U_i \)  degree of consolidation at time t

\( \bar{u}_0 \)  average pore water pressure

\( \alpha_c \)  non-Darcian flow parameter

\( \alpha_c / \alpha'_c \)  ratio of smeared zone in non-Darcian flow

\( \beta \)  non-Darcian flow parameter

\( \eta \)  constant for non-Darcian flow

\( \alpha \)  geometric parameter representing smear in plane strain

\( \beta \)  geometric parameter representing smear in plane strain

\( \varepsilon \)  strain

\( \Delta \sigma \)  applied preloading pressure
\( \Delta p \) increase in vertical effective stress

\( \gamma_w \) unit weight of water

\( \sigma'_i \) initial surcharge load

\( \bar{\sigma}_0 \) initial average vertical stress

\( \bar{\sigma}_f \) final average vertical stress at the end of consolidation

\( \bar{\sigma}'_i \) initial effective stress

\( \bar{\sigma}_{vy,D} \) average yield stress of the partially disturbed soil

\( \sigma'_{vp}, P_c \) pre-consolidation stress

\( v \) velocity of flow

\( i \) hydraulic gradient
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