SOFT CLAY FOUNDATION IMPROVEMENT VIA
PREFABRICATED VERTICAL DRAINS AND VACUUM
PRELOADING

A thesis submitted in fulfilment of the requirements for the award of the degree

Doctor of Philosophy

from

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by

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CERTIFICATION

I, Chamari Inomika Bamunawita, declare that this thesis, submitted in fulfilment of the requirements for the award of Doctor of Philosophy, in the Department of Civil Engineering, University of Wollongong, is wholly my own work unless otherwise referenced or acknowledged. The document has not been submitted for qualifications at any other academic institution.

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Chamari Inomika Bamunawita

23 February 2004
ABSTRACT

This thesis includes the numerical modelling of prefabricated vertical drain (PVD) subjected to vacuum loading in a 2-D plane strain finite element model employing the modified Cam-clay theory, and the experimental evaluation of effectiveness of combined vacuum and surcharge preloading using a large scale, radial drainage consolidometer. The original axisymmetric analysis and plane strain analysis of vertical drains including the effect of smear and well resistance have been well documented in the past for surcharge preloading. In this study, the existing axisymmetric and plane strain theories of a unit cell are modified to incorporate the vacuum pressure application. Unsaturation of drain soil boundary owing to the vacuum pressure is also considered in the numerical modeling. Thereafter, a multi-drain, plane strain analysis is conducted to study the performance of the entire embankment stabilised with vertical drains subjected to vacuum preloading, for two case histories taken from Thailand.

A laboratory technique of evaluating the effectiveness of combined vacuum and surcharge preloading is elaborated. In this approach, a central vertical drain was installed in soil specimens placed in a large stainless steel cell (450 mm in diameter and 950 mm in height) using a specially designed mandrel, and then the vacuum and surcharge loads were applied using the two different loading systems. The results clearly show the effectiveness of vacuum preloading. Following initial laboratory simulation in the large-scale radial drainage consolidometer, a different approach to conventional analysis is adopted to analyse the vacuum assisted consolidation around vertical drains. It is assumed here that a linear variation of negative pore pressure along
the drain length and a constant (maximum) suction head at the ground surface are realistic and sufficient. The observed retardation of pore pressure dissipation is explained through a series of finite element models, which consider the effect of unsaturation at the drain-soil interface. The results indicate that the introduction of an unsaturated soil layer adjacent to a PVD improves the accuracy of numerical predictions.

The knowledge gained from the modeling of large-scale consolidometer cell is applied to study the behaviour of two embankments built on soft clay, stabilised with vertical drains subjected to vacuum loading. A multi-drain analysis is conducted and the field measurements are compared with a series of numerical model predictions. The best predictions of settlement, lateral displacements and pore pressures are obtained when the numerical analysis included the time and depth dependent changes in vacuum pressure, in addition to having an unsaturated layer of elements along the external boundary of the PVD. Finally, a comprehensive multi-drain analysis is used to predict the failure height of embankment, considering various parameters such as embankment geometry, construction method, sub soil properties and soil improvement techniques.
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LIST OF SYMBOLS

\( a \)  
Width of band drain

\( a_v \)  
Coefficient of compressibility

\( b \)  
Thickness of band drain

\( b_s \)  
Equivalent width of smear zone in plane strain

\( b_w \)  
Equivalent width of drain (well) in plane strain

\( B \)  
Equivalent half width of the plane strain cell

\( c_h \)  
Coefficient of horizontal consolidation

\( c_u \)  
Undrained shear strength

\( c_v \)  
Coefficient of vertical consolidation

\( C_c \)  
Compression index

\( C_f \)  
Ratio of field and laboratory coefficient of permeability

\( C_k \)  
Permeability change index

\( C_r \)  
Recompression index

\( C_\alpha \)  
Secondary compression index

\( d_e \)  
Equivalent diameter of band drain

\( d_s \)  
Diameter of smear zone

\( d_w \)  
Diameter of drain (well)

\( D \)  
Diameter of effective influence zone of drain

\( D_{15} \)  
Diameter of clay particles corresponding to 15% passing

\( D_{50} \)  
Diameter of clay particles corresponding to 50% passing

\( D_{85} \)  
Diameter of clay particles corresponding to 85% passing

\( e \)  
Void ratio

\( e_{cs} \)  
Void ratio on the critical state line for value of \( p' = 1 \)

\( e_0 \)  
Initial void ratio

\( E \)  
Young’s modulus

\( E_u \)  
Young’s modulus for undrained shear

\( F_t \)  
Influence factor of drain due to time

\( F_c \)  
Influence factor of drain due to drain deformation
$F_{fc}$ Influence factor of drain due to clogging
$G$ Shear modulus
$G_s$ Specific gravity
$H_d$ The longest drainage path
$H_o$ Initial thickness of compressible soil
$i$ Hydraulic gradient
$i_o$ Initial hydraulic gradient
$I_c$ Influence factor
$I_p$ Plasticity index
$k$ Permeability
$k_{ax}$ Axisymmetric permeability
$k_h$ Horizontal coefficient of permeability
$k'_h$ Horizontal coefficient of permeability in smear zone
$k_{hp}$ Equivalent horizontal coefficient of permeability in plane strain
$k'_{hp}$ Equivalent horizontal coefficient of permeability in plane strain
$k_v$ Vertical coefficient of permeability
$k_s$ Saturated permeability
$k_u$ Unsaturated permeability
$k_w$ Coefficient of permeability of drain
$l$ Length of drain
$L$ Well resistance factor
$L_L$ Liquid limit
$m_v$ Coefficient of volume change
$M$ Slope of critical state line
$n$ Spacing ratio, $R/r_w$
$N$ Volume on the normal consolidation line corresponds to $p' = 1$
$O_{95}$ Apparent opening size
$O_{15}$ Apparent opening size
$O_{85}$ Apparent opening size
$p'$ Effective mean normal pressure
\( p'c \) Isotropic preconsolidation pressure

\( P \) Normal load

\( PI \) Plasticity index

\( q \) Unit load

\( qw \) Axisymmetric vertical drain discharge capacity

\( qwa \) Axisymmetric discharge capacity

\( qwp \) Plane strain discharge capacity

\( qz \) Plane strain discharge capacity

\( Q \) Discharge capacity

\( Qw \) Discharge capacity in plane strain

\( r \) Radius

\( r_k \) Anisotropy ratio

\( r_m \) Radius of mandrel

\( rs \) Radius of smeared zone

\( rw \) Radius of vertical drain (well)

\( R \) Radius of axisymmetric unit cell

\( s \) Smear ratio, \( rs/rw \)

\( S \) Field spacing of drains

\( Se \) Effective degree of saturation

\( Sr \) Degree of saturation

\( S_{ru} \) Residual water saturation

\( t \) Time

\( Th \) Time factor for horizontal drainage

\( Thp \) Time factor for horizontal drainage in plane strain

\( Tv \) Time factor for vertical drainage

\( T50 \) Time factor for 50\% consolidation

\( T90 \) Time factor for 90\% consolidation

\( u \) Pore water pressure

\( ua \) Pore air pressure

\( u_o \) Initial pore water pressure

\( ur \) Excess pore water pressure in radial direction
$u_{sur}$  Applied surcharge pressure  

$u_{vac}$  Applied vacuum pressure in axisymmetric condition  

$u_{vacp}$  Applied vacuum pressure in plane strain condition  

$u_w$  Pore water pressure  

$\bar{U}$  Average degree of consolidation  

$U_{10}$  10 % degree of saturation  

$U_{ax}$  Degree of consolidation in axisymmetric  

$U_{pl}$  Degree of consolidation in plane strain  

$v$  Rate of flow  

$V$  Volume  

$w$  Water content  

$w_L$  Liquid limit  

$w_P$  Plastic limit  

$z$  Depth (thickness) of soil layer  

$\alpha$  Geometric parameter representing smear in plane strain  

$\beta$  Geometric parameter representing smear in plane strain  

$\beta_1$  Ratio of maximum lateral displacement to settlement  

$\beta_2$  Ratio of maximum lateral displacement to corresponding fill height  

$\chi$  Effective stress parameter  

$\varepsilon$  Strain  

$\gamma$  Unit weight of soil  

$\gamma_v$  Unit weight of water  

$\eta$  Stress ratio  

$\mu$  Smear and well resistance factor in axisymmetric  

$\mu_p$  Smear and well resistance factor in plane strain  

$\Gamma$  Volume on the critical state line corresponds to $p' = 1$; $\Gamma = e_{cs} + 1$  

$\kappa$  Slope of swelling line  

$\lambda$  Slope of consolidation line  

$\nu$  Poisson’s ratio
<table>
<thead>
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<td>Geometric parameter representing well resistance in plane strain</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Settlement</td>
</tr>
<tr>
<td>$\rho_\infty$</td>
<td>Settlement at infinity</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>Total horizontal stress</td>
</tr>
<tr>
<td>$\sigma_v$</td>
<td>Total vertical stress</td>
</tr>
<tr>
<td>$\sigma'_h$</td>
<td>Effective horizontal stress</td>
</tr>
<tr>
<td>$\sigma'_v$</td>
<td>Effective vertical stress</td>
</tr>
<tr>
<td>$\sigma'_p$</td>
<td>Preconsolidation pressure</td>
</tr>
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<td>$\sigma_r$</td>
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<td>$\sigma_z$</td>
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<tr>
<td>$\sigma_s$</td>
<td>Confining stress</td>
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1 INTRODUCTION

1.1 General

Due to the rapid increase in population and associated development activities taking place in the world during the last few decades, construction activities are now mainly concentrated on low-lying marshy areas, which comprise of highly compressible weak organic and peaty soils of varying thickness. These soft clay deposits have low bearing capacity as well as excessive settlement characteristics. Therefore, it is essential to stabilise the existing soft soils before commencing the construction activities, in order to prevent differential settlements. Also in such low-lying areas, it is necessary to raise the existing ground level to keep the surface area above the ground watertable or flood level.

Preloading is the most successful ground improvement technique that can be used in low-lying areas. It involves loading of the ground surface to induce a greater part of the ultimate settlement that the ground is expected to experience after construction. Since most compressible soils are characterized by very low permeability and considerable thickness, the time needed for the required consolidation can be long, and also the surcharge load required may be significantly high. Currently, this may not be possible with busy construction schedules. Installation of vertical drains can reduce the preloading period significantly by decreasing the drainage path length in the radial direction (Fig. 1.1), as the consolidation time is inversely proportional to the square of the length of the drainage path. Application of vacuum pressure with surcharge loading can further accelerate consolidation while reducing the required surcharge fill material
without any adverse effects on the stability of an embankment built on soft clay. The applied vacuum pressure generates negative pore water pressure, resulting in an increase in effective stress in the soil, which leads to accelerated consolidation.

Figure 1.1 Effect of vertical drain on drainage path (http://www.americandrainagesystems.com/wickdrain.htm)

Smear and well resistance are the main factors influencing vertical drain efficiency. It is almost impossible to install a vertical drain without causing some disturbance to the surrounding soil. This region of disturbance, which is called the ‘smear zone’, decreases the permeability of the soil that in turn retards the rate of consolidation process. The resistance to water flowing along the vertical drain is known as the ‘well resistance’. The deep installation of vertical drains or a limited discharge capacity of the drain will increase the well resistance. Apart from smear and well resistance, drain unsaturation is also an important factor, which affects the drain efficiency. The effect would be more noticeable in the early stages of consolidation, but diminishing as the drain material becomes increasingly saturated. In vacuum preloading, it is possible that the soil at the
drain interface may become unsaturated during the vacuum application, which also affects the drain efficiency to some extent, although the consolidation is generally accelerated by vacuum preloading.

In this study, the effectiveness of vertical drains subjected to combined surcharge and vacuum loading was evaluated by modeling its behaviour in large-scale consolidation tests. Following laboratory simulation, a new approach was adapted to model vacuum loading using the plane strain theory developed by Indraratna and Redana (2000). An attempt was made to study the unsaturation behaviour of drain soil boundary owing to vacuum loading. The subsoil properties were modeled according to the modified Cam-clay theory, and the ABAQUS finite element software was selected for numerical modeling.

1.2 Purpose and Application of Vertical Drains

The geosynthetic (prefabricated) vertical drains can accelerate the rate of primary consolidation by reducing the length of drainage path, hence, increasing the rate of pore water pressure dissipation in the radial direction. Significant secondary consolidation is not usually observed for soft clays stabilized by a sufficient number of PVD. Due to the rapid initial consolidation PVD’s will increase the stiffness and bearing capacity of soft foundation clays. Application of vacuum load can further accelerate the rate of settlement, generally compensating for the adverse effects of smear and well resistance.
Sand drains and prefabricated band shaped drains (PVD) are the most commonly used vertical drains. Although sand drains were widely used until the early 1970’s, presently, PVDs have almost totally replaced the conventional sand drains throughout the world. Sand drain, which is simply a borehole filled with sand, does not only provide good drainage, but also reinforces the soft foundation. Sand compaction piles provide significantly increased stiffness to soft compressible soils (Indraratna et al., 1997). However, sand drains are susceptible to damage due to lateral ground movement. The PVDs are usually composed of a plastic core (protected by fabric filter) with a longitudinal channel. The filter (sleeve) is made of synthetic or natural fibrous material with a high resistance to clogging. Typical types of PVD’s such as Mebra drain and Colbond drain are shown in Figure 1.2. The most common band shaped drains have dimensions of 100mm x 4mm.
The vertical drains can be installed using either a static or dynamic method. In the static procedure, the mandrel is pushed into the soil by means of a static force. In the dynamic method of installation, the mandrel is driven into the ground using a vibrating hammer or a conventional drop hammer. Static pushing is preferable for driving the mandrel into the ground, whereas the dynamic methods seem to create a higher immediate excess pore water pressure and a greater disturbance of the surrounding soil during installation. A typical installation rig is shown in Figure 1.3, where a vertical drain is driven into soft ground using a mandrel hoisted by a crane. The degree of disturbance during installation depends on several factors such as the mandrel size, mandrel shape and soil macrofabric.

![Figure 1.3 Typical installation rig](http://www.geosynthetics.colbond.com/soilimp.html)
Vertical drains are applicable for moderately to highly compressible soils, which are usually normally consolidated or lightly over consolidated, and for stabilizing a deep layer of soil having a low permeability. These soils include organic and inorganic silts and clay with low to moderate sensitivity, varved cohesive soils and decomposed peat. The vertical drains are particularly efficient where the clay layers contain many thin horizontal sand or silt lenses (microlayers). However, if these microlayers are continuous in the horizontal direction, the installation of vertical drains may not be effective, since rapid drainage of the pore water out of the soil layers may occur through the horizontal microlayers. In this case, wells for pressure relief in the sand layer might be more appropriate.

1.3 Application of Surcharge and Vacuum Preloading

When construction activities take place on low-lying marshy areas, the very soft clays will pose a stability problem or differential settlements. A common practice to overcome this problem is to support the structure on special foundations, which could accommodate differential settlement to a greater degree, or to support them on pile foundations. In the case of a deep strong bearing stratum, foundation cost may become prohibitively high and not commensurate with the cost of the super structure, for example in the case of low rise buildings subject to low to moderate loads. Also, problems arise when construction of railway and highway embankments is carried out on soft coastal clays. Alternative and economical solutions for such ground improvement include surcharge preloading that is widely employed in industry. Surcharge preloading is the application of a uniformly distributed load prior to
construction, by an appropriate fill, whose load intensity is often comparable to that of the expected structure. In order to control the development of excess pore pressures, this surcharge embankment is usually raised as a multi-stage exercise with rest provides between the stages, which is often time consuming.

When there are limitations on the space for the required embankment construction or there are no resources for suitable fill material, application of vacuum loading is another alternative technique. This practice saves time in the absence of a high surcharge embankment. The external negative load is applied to the soil surface in the form of vacuum through a sealed membrane system (Choa, 1989). The higher effective stress is achieved by rapidly decreasing the pore water pressure, while the total stress remains the same. The efficiency of vacuum loading can be further increased in conjunction with a small surcharge load. When vacuum preloading is affected via PVD, the surrounding soil tends to move radially inward, while the conventional surcharge loading causes outward lateral flow. The result of course is a reduction of the outward lateral displacements, thereby reducing risk of damage to adjacent structures, piles etc. In the case of vacuum application, it is important to ensure that the site to be treated is totally sealed and isolated from any surrounding permeable soils to avoid air leakage that adversely affects the vacuum efficiency.

Numerical modeling of vacuum preloading is still a developing research area with limited literature. In some past studies, vacuum pressure has been numerically modeled either by increasing the apparent surcharge pressure, thereby increasing the total stress
on the surface or by fixing the pore pressure boundary at the ground surface (Park et al., 1997). However, previous field data (Choa, 1989; Chu et al., 2000; Tang et al., 2000) conclude that the vacuum pressure is efficiently distributed along the depth through PVD. Modelling of vacuum preloading will be discussed in detail in Chapter 5 of this thesis.

1.4 Analysis of Soft Clay Foundations Stabilized with Vertical Drains

The behaviour of earth structures built on soft clay stabilized with vertical drains can now be predicted reasonably accurately. This is due to the significant progress that has been made in the past few years through numerical, analytical and laboratory modeling. The classical solution of radial consolidation has been well documented (Barron, 1948; Hansbo, 1981) to enable the prediction of settlement caused by vertical drains. With the rapid development of the finite element methods in Geotechnical Engineering and significant fall in price of computer hardware, a comprehensive analysis of the behaviour of the soft clay stabilized with vertical drain can now be conducted cost effectively.

Finite element technique is based on the discretization of a continuum into a number of elements, which are connected at nodal points. The deformation response of each element is defined by element shape, the displacement variation within each element and the stress-strain behaviour (constitutive model) employed to represent the behaviour of the element. Several researchers (Brenner et al., 1983; Onoue, 1988b) have developed finite difference methods for vertical drains, in which either ‘explicit’ or ‘implicit’
solutions have usually been adopted. A rigorous study between these two methods concluded that the implicit method provides a better numerical stability, although a set of simultaneous equations need to be solved at each time step (Desai et al., 1977). The main advantage of numerical analysis is that the settlement and stresses within the soil are coupled, and therefore more realistic soil behaviour can be simulated.

1.5 Objectives and the Scope of Study

The main objective of this study was to examine the effect of vacuum preloading in accelerating the consolidation of vertical drains installed in soft clay. The effect was examined by conducting several large-scale consolidometer tests in the laboratory. The effect of vacuum removal and reapplication was also examined.

Following laboratory simulation, a new approach was adopted to model vacuum loading in ABAQUS finite element code incorporating the critical state theory. The settlements observed in the laboratory tests were compared with the numerical predictions. The effect of introducing an unsaturated soil layer adjacent to the PVD was also investigated through finite element analysis. Two case studies from Thailand were analysed to verify the proposed model.

Effect of vacuum preloading on soil consolidation around vertical drain was theoretically analysed by modifying the existing theories for axisymmetric and plane strain conditions. Finally, the knowledge gained from the analysis of consolidometer cell
and real embankments was used to demonstrate how the plane strain analysis can be employed to predict the stability of embankments.

1.6 Organisation of the Thesis

In this Chapter 1 a brief introduction was presented where the concept of vacuum and surcharge preloading, the aim and scope of the present research were highlighted.

The following Chapter 2 presents a comprehensive survey of the literature associated with the present work. History of vertical drains, present theories related to vertical drains, the analyses of embankments stabilized with vertical drains and related numerical and experimental investigations are reviewed in detail.

Chapter 3 presents the development of original axisymmetric and plane strain theories to incorporate the application of vacuum pressure. The influence of various factors such as the drain spacing, well resistance and smear effect on soil consolidation around vertical drain subjected to vacuum preloading is discussed.

Chapter 4 presents the laboratory testing and discussion of test findings. The details of the large-scale radial drainage consolidometer and the instrumentation to monitor the settlement and pore water pressure are explained. The procedure of the laboratory test to study the effect of vacuum preloading and the typical settlement and pore pressure response of the soil is also discussed.
Chapter 5 presents the numerical modeling of vertical drains subjected to vacuum preloading. The advantage of 2D plane strain modeling, modeling of vacuum preloading is explained in detail. Effect of soil unsaturation due to vacuum loading, details of soil moisture characteristic curve, Mandel-Cryer effect and the effect of variable and constant permeability are also presented in this chapter.

Chapter 6 presents the application of proposed model into two embankments stabilized with vertical drains subjected to vacuum loading in Thailand. Multi drain analysis was carried out through series of models and the predicted settlements, pore pressures and lateral displacements are compared with the field measurements.

Chapter 7 discusses the use of 2-D plane strain numerical analysis of soft clay foundation under different conditions such as, with and without vertical drains, with and without vacuum preloading, different embankment geometries and different drain spacing to predict the failure height of embankment. Comparisons between various conditions are elucidated.

Chapter 8 presents the conclusions of present research and recommendations for further research followed by the Bibliography and Appendices.
2 LITERATURE REVIEW

2.1 History and Development of Vertical Drains

In the past three decades, sand compaction piles and prefabricated vertical drains have been used extensively for the purpose of soft ground improvement. Sand drains were first used in practice in and around 1920’s. The California Division of Highways has conducted the first comprehensive laboratory and field tests in 1933. In Japan, during 1940’s the behaviour of vertical sand drains was not well understood, because the bearing capacity of foundations after installation of sand drains was considered to provide sufficient reinforcement, hence, the full load was placed on the foundation too quickly resulting in frequent foundation failure (Aboshi, 1992). In 1956, the method of sand compaction piles was developed in which the vertical sand drains were compacted during installation (Aboshi, 1992). After sand was poured into the pipe (casing), it was withdrawn partway and again driven down to compact the sand column and enlarge its diameter. Although no effect of densification was expected in clayey soils, these sand columns behaved as granular piles in soft ground, thereby carrying a greater load. At the same time, they also worked as vertical drains to accelerate consolidation of clayey ground, by rapid pore pressure dissipation.

Since 1940, prefabricated band shaped drains and Kjellman cardboard wick drains have been introduced in ground improvement. Subsequently, several types of prefabricated band drains were developed such as: Geodrain (Sweden), Alidrain (England), Mebradrain (Netherlands) etc. Basically, the prefabricated band drains compose of a
plastic core with a longitudinal channel wick functioning as a drain, and a sleeve of paper or fibrous material as a filter protecting the core.

### 2.2 Installation and Monitoring of Vertical Drains

Figure 2.1 illustrates a typical scheme of vertical drains installation and monitoring instruments required to monitor the performance of the soil foundation beneath an embankment. Prior to vertical drain installation, it is necessary to conduct general site preparation. This work may include the removal of vegetation and surficial debris, establishing site grading and constructing a sand blanket. The purpose of the sand blanket is to conduct the expelled water away from the drains and to provide a sound-working mat.

![Figure 2.1 Basic instrumentation of embankment](image)

In the case of vacuum preloading (Cognon et al., 1996), the next step is to install horizontal drains in transverse and longitudinal directions. These drains are linked
through transverse connecters and are subsequently linked to the edge of the peripheral trench. The trenches are excavated around the preloaded area to a depth of 0.5m below the ground water level and filled with impervious Bentonite Polyacrylate slurry for sealing of the impermeable membrane along the perimeter. Impermeable membrane is installed on the ground surface and sealed along the peripheral trenches. The trenches are backfilled with water to improve the sealing between the membrane and Bentonite slurry. Finally the vacuum pumps are connected to the prefabricated discharge module extending from the trenches. Figure 2.2 shows a typical embankment subjected to vacuum preloading.

![Figure 2.2 Schematic diagram of embankment subjected to vacuum loading](image)

Several types of geotechnical instrumentation are required to install before and after construction, in order to monitor the performance of the embankment. Monitoring is
essential to prevent sudden failures, to record changes in the rate of settlement and to verify the design parameters. Performance evaluation is also important to improve settlement predictions and to provide sound guidelines for the future projects.

Settlement gauges (e.g. of hydraulic type) are used to measure the long-term settlements at the original ground surface and should be placed immediately after the installation of vertical drains. Settlement plates are suitable for initial reading and should be installed at the bottom of sand blanket, at intermediate depths, and at the bottom of the compressible layer to monitor its settlement. A benchmark is usually set up on a stable ground at a reasonable distance from the fill. Piezometers are used to monitor the complete pore pressure profiles and should be installed at the bottom of sand blanket, at intermediate depths and at the bottom of the compressible layer. A dummy or remote piezometer should also be installed at an adequate distance from the embankment to record the original or natural ground water level, and in general, the pore water pressure at a particular depth. Alignment stakes can be installed parallel to the embankment slope prior to the placement of the fill. The alignment stake is a simple means of measuring the lateral displacement of the foundation during construction of the embankment. In fact, they could provide an early warning of bearing capacity failure. A more sophisticated equipment to measure lateral displacement is an inclinometer, which is usually placed set around the toe of the embankment.
2.3 Drain Properties

2.3.1 Equivalent drain diameter for band shaped drain

The conventional theory of consolidation with vertical drains assumes that the vertical drains are circular in cross-section. Therefore, a band-shaped drain needs to be converted to an equivalent circular diameter, which implies that the equivalent diameter of a circular drain has the same theoretical radial drainage capacity as the band-shaped drain.

Kjellman (1948) stated, “the draining effect of a drain depends to a great extent upon the circumference of its cross-section, but very little upon its cross-sectional area”. Based on Kjellman’s consideration, Hansbo (1981) introduced the equivalent diameter for a prefabricated band-shaped drain, as given in Equation 2.1.

\[
d_w = 2 \frac{(a + b)}{\pi} \text{ or } r_w = \frac{(a + b)}{\pi}
\]  

(2.1)

Another study (Rixner et al., 1986) suggested that the more appropriate \(d_w\) is given by the less complex relationship as in Equation 2.2.

\[
d = \frac{a + b}{2}
\]  

(2.2)

where, \(a = \) the width of the PVD and \(b = \) the thickness of the PVD
Equivalent diameter:

\[ d_{w} = \frac{2(a + b)}{\pi} \]

Assumed water flownet (Pradhan et al., 1993)

Figure 2.3 Equivalent diameter of band-shaped vertical drains

Pradhan et al. (1993) suggested that the equivalent diameter of band-shaped drains should be estimated by considering the flow net around the soil cylinder of diameter \( d_{e} \) (Fig. 2.3). The mean square distance of their flow net is calculated as:

\[ \bar{s}^2 = \frac{1}{4} d_{e}^2 + \frac{l}{12} a^2 - \frac{2a}{\pi^2} d_{e} \]  \hspace{1cm} (2.3)

Then, \( d_{w} = d_{e} - 2\sqrt{\bar{s}^2} + b \)  \hspace{1cm} (2.4)

Based on finite element studies and by comparing the equivalent band shaped drains diameter represented by Equations 2.1 and 2.2, it was found that the equivalent diameter given by Equation 2.1 should be reduced by 20 percent. It was suggested by Rixner et al. (1986) that the use of Equation 2.2 is more appropriate than Equation 2.1, based on numerical modelling.
2.3.2 Filter and apparent opening size (AOS)

The drain material (sand drain) and the filter jacket of PVD have to perform two basic but contrasting requirements, which are retaining the soil particles and at the same time allowing the pore water to pass through. The general guideline of the drain permeability is given by:

\[ k_{\text{filter}} > 2 \, k_{\text{soil}} \]  \hspace{1cm} (2.5)

An effective filtration can minimise soil particles from moving through the filter. A commonly employed filtration requirement is given by:

\[ \frac{O_{95}}{D_{85}} \leq 3 \]  \hspace{1cm} (2.6)

where, AOS, \( O_{95} \) indicates the approximate largest particle that would effectively pass through the filter. Sieving is done using glass beads of successively larger diameter until 5% passes through the filter, and this size in millimeters defines the AOS, \( O_{95} \) based on ASTM D 4751 (ASTM, 1993). This apparent opening size (AOS) is usually taken to be less than 90 microns based on Equation 2.6. \( D_{85} \) indicates the diameter of clay particles corresponding to 85% passing.

The retention ability of the filter is given by:

\[ \frac{O_{50}}{D_{50}} \leq 24 \]  \hspace{1cm} (2.7)

Filter material can also become clogged if the soil particles become trapped within the filter fabric structure. Clogging is prevented by ensuring that (Christopher and Holtz, 1985):

\[ \frac{O_{95}}{D_{15}} \geq 3 \]  \hspace{1cm} (2.8)
2.3.3 Discharge capacity

The discharge capacity of the prefabricated vertical drain is required to analyse the drain (well) resistance factor. However, well resistance factor is always less significant than the drain spacing and the disturbance (smear effect). In order to measure the discharge capacity of drains, it is required to simulate as closely as possible the conditions in the field. In this case, the discharge capacity will be a function of the volume of the core or the drain channel, the lateral earth pressure acting on the drains, possible folding, bending and twisting of the drain due to large settlement, infiltration of fine soil particles through the filter, and the biological and chemical degradation. Incorporating the above factors, the actual discharge capacity, $q_w$, is then given by:

$$q_w = (F_t)(F_c)(F_{fc})q_{req}$$

(2.9)

where $F_t$, $F_c$ and $F_{fc}$ are the influence factors due to time, drain deformation and clogging, respectively. The term $q_{req}$ is the theoretical discharge capacity calculated from Barron’s theory of consolidation, which is given by:

$$q_{req} = \frac{\epsilon_f U_{10} l c_h}{4T_h}$$

(2.10)

where, $\epsilon_f$ is the final settlement of the soft soil equivalent to 25% of the length of the drain installed to the soft ground, $U_{10}$ is the 10 percent degree of consolidation, $l$ is the depth of the vertical drain, $c_h$ is horizontal coefficient of consolidation and $T_h$ is the time factor for horizontal (lateral) consolidation.
Figure 2.4 Typical values of vertical discharge capacity (after Rixner et al., 1986)

From laboratory tests, the influence factor of time, $F_t$, has been estimated to be less than 1.2, and it is usually conservatively taken to be 1.25. The reduction of the discharge capacity under the worst condition of bending, folding and twisting has been suggested to be about 48%, which gives an influence factor of deformation, $F_c$ of about 2. The filtration tests show that the trapped fine soil particles decrease the permeability of the PVD, and in turn decrease the PVD discharge capacity. This deterioration is complicated by the biological and chemical growth in the geotextile filter. From filtration tests, the value of $F_{fc}$ is suggested to vary between 2.8 and 4.2 with an average of about 3.5. After considering all the worst conditions that may occur in the field, the discharge capacity, $q_w$ of the PVD could be as high as 500-800 m$^3$/year, but reduced to 100-300 m$^3$/year where the hydraulic gradient is unity under elevated lateral pressure (Rixner et al., 1986).
The discharge capacity of various types of drains is shown in Figure 2.4, where the discharge capacity is influenced by lateral confining pressure. It is also suggested, in lieu of laboratory test data, that the discharge capacity can be conservatively assumed to be 100 m³/year. Hansbo (1981) based on laboratory test results suggested a much smaller discharge capacity of drains, as summarised in Table 2.1.

### Table 2.1 Short-term discharge capacity, in m³/year, of eight band drains measured in laboratory (Hansbo, 1981)

<table>
<thead>
<tr>
<th>Drain type</th>
<th>Lateral pressure in kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>40</td>
</tr>
<tr>
<td>Geodrain</td>
<td>26</td>
</tr>
<tr>
<td>Other drain types</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>24</td>
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<tr>
<td></td>
<td>15</td>
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<td></td>
<td>10</td>
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<td></td>
<td>21</td>
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<td>-</td>
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<tr>
<td></td>
<td>19</td>
</tr>
</tbody>
</table>

Holtz et al. (1991) reported that the discharge capacity of prefabricated vertical drains could vary from 100-800 m³/year. The discharge capacity of PVD is a function of its filter permeability, core volume or cross section area, lateral confining pressure, drain stiffness controlling its deformation characteristics, among other factors (Hansbo, 1979;
Holtz et al. 1991). For long vertical drains that are vulnerable to well resistance, Hansbo (1981) and Holtz et al. (1988, 1991) pointed out that in the field, the actual reduction of the discharge capacity can be attributed to: (a) reduced flow in drain core due to increased lateral earth pressure, (b) folding and crimping of drain due to excessive settlements, and (c) infiltration of fine silt or clay particles through the filter (siltation).

Based on a number of experimental studies, Holtz et al. (1988) concluded that as long as the initial discharge capacity of PVD exceeds 100-150 m$^3$/year, some reduction in discharge capacity due to installation should not seriously influence the consolidation rates. However, discharge capacity $q_w$ can fall below this desired minimum value due to the three reasons mentioned earlier. For certain types of PVD, affected by significant vertical compression and high lateral pressure, $q_w$ values may be reduced to 25-100 m$^3$/year (Holtz et al., 1991). Clearly, the ‘clogged’ drains are associated with $q_w$ values approaching zero.

### 2.4 Factors Influencing the Vertical Drain Efficiency

#### 2.4.1 Smear zone

The installation of vertical drains by means of a mandrel causes significant remoulding of the subsoil especially in the immediate vicinity of the mandrel. Thus, a zone of smear will be developed with reduced permeability and increased compressibility. Barron (1948) stated that if the drain wells are installed by driving cased holes and then back filling as the casing is withdrawn, the driving and pulling of the casing would distort and
remould the adjacent soil. In varved soils, the finer and more impervious layers will be dragged down and smeared over the more pervious layers, resulting in a zone of reduced permeability in the soil adjacent to the well periphery. As remoulding retards the consolidation process of the subsoil, it has to be considered in any theoretical solution. Barron (1948) suggested the concept of reduced permeability, which is equivalent to lowering the overall value of the coefficient of consolidation. Hansbo (1979) also introduced a zone of smear in the vicinity of the drain with a reduced value of permeability.

The combined effect of permeability and compressibility within the smear zone brought a different behaviour from the undisturbed soil, hence, the prediction of the behaviour of the soil stabilised with vertical drains cannot be made accurately if the effect of smear is ignored. Both Barron (1948) and Hansbo (1981) modelled the smear zone by dividing the soil cylinder dewatered by the central drain into two zones. The smear zone is the region in the immediate vicinity of the drain, which is disturbed, and the other zone is the intact or undisturbed region outside the smear zone. Onoue et al. (1991) introduced a three zone hypothesis defined by: plastic smear zone in the immediate vicinity of the drain where the soil is highly remolded during the process of installation of the drain, plastic zone where the permeability is reduced moderately, and the undisturbed zone where the soil is not at all affected by the process of drain installation. This three-zone analysis was suggested after extensive laboratory testing by Ting, et al. (1990). However, due to the complex variation of permeability in the radial direction from the
drain, the solution of the three-zone approach becomes increasingly difficult. For practical purposes, the two-zone approach is generally sufficient.

Jamiolkowski et al. (1981) proposed that the diameter of the smear zone ($d_s$) and the cross sectional area of mandrel can be related as follows:

$$ d_s = \frac{(5\text{to}6)d_m}{2} $$

(2.11)

where $d_m$ is the diameter of the circle with a area is equal to the cross sectional area of the mandrel. Based on the results of Holtz and Holm (1973) and Akagi (1979) Hansbo (1987) proposed another relationship as follows:

$$ d_s = 2d_m $$

(2.12)

Indraratna & Redana (1998) proposed that the estimated smear zone is about 3-4 times the cross-sectional area of the mandrel. The proposed relationship was verified using the specially designed large-scale consolidometer (Indraratna & Redana, 1995). The schematic section of the consolidometer and the location of the recovered specimen are shown in Figures 2.5(a) and 2.5(b). Figure 2.6 shows the variation of $k_h/k_v$ ratio along the radial distance from the central distance. According to Hansbo (1987) & Bergado et al. (1991), in the smear zone, the $k'_h/k_v'$ ratio was found to be close to unity, which is in agreement with the results of the study of Indraratna & Redana (1998). The degree of disturbance depends on several factors as described below.
Figure 2.5 a) Schematic section of the test equipment showing the central drain and associated smear and b) locations of small specimens obtained to determine the consolidation and permeability characteristics (Indraratna & Redana, 1998)

2.4.1.1 Mandrel size and shape

Generally, disturbance increases with the total mandrel cross sectional area. The mandrel size should be as close as possible to that of the drain to minimise displacement. While working on the effect of mandrel driven drains on soft clays, (Akagi, 1977, 1981),
observed that when a closed-end mandrel is driven into a saturated clay, the clay will suffer large excess pore water pressure, associated with ground heave and lateral displacement. The strength and coefficient of consolidation of the surrounding soil can then decrease considerably. However, the excess pore pressure dissipated rapidly, followed by the process of consolidation after installation of the mandrel or before the fill is placed. Bergado et al. (1991) reported from a case study of a Bangkok clay embankment stabilised with vertical drains that a faster rate of settlement took place in the area where the drains were installed using a mandrel with a smaller cross section area rather than a larger mandrel. This verifies that a smaller smear zone was developed in the former.

Figure 2.6 Ratio of \( k_h/k_v \) along the radial distance from the central drain
2.4.1.2 Soil macro fabric

For soil with pronounced macrofabric, the ratio of horizontal permeability to vertical permeability \( k_h/k_v \) can be very high, whereas the \( k_h/k_v \) ratio becomes unity within the disturbed (smear) zone. The vertical drains are particularly efficient where the clay layers contain many thin horizontal sand or silt lenses (microlayers). However, if these microlayers are continuous in the horizontal direction, the installation of vertical drains may not be effective, since rapid drainage of the pore water out of the soil layers may occur irrespective of whether the drains are installed or not.

2.4.1.3 Installation procedure

Jamiolkowski and Lancellota (1981) suggested that the smear zone is given by

\[
d_s = (5 \text{ to } 6) r_m
\]  

(2.13)

where \( r_m \) is the radius of a circle with an area equal to the mandrel’s greatest cross-sectional area, or the cross-sectional area of the anchor or tip, whichever is greater. For design purposes, it is currently assumed that within the disturbed zone, complete soil remoulding occurs.

Evaluation of the effect of installation on the degree of disturbance is a very complex matter in soil mechanics. Equation 2.13 provides only a very simple approach for accounting for disturbance. Baligh (1985) developed a strain path method to estimate the disturbance caused during the installation of piles. The state of strain during undrained axisymmetric penetration of closed end piles have three deviatoric strain components,
$E_1$, $E_2$ and $E_3$. $E_1$ is the shearing strain in a conventional triaxial test, $E_2$ is the strain from pressuremeter tests (cylindrical cavity expansion tests) and $E_3$ is the strain from simple shear tests. The second deviatoric strain invariant, the octahedral strain, $\gamma_{oc}$, is then given by:

$$\gamma_{oct} = \frac{I}{\sqrt{2}}\left[E_1^2 + E_2^2 + E_3^2\right]^{\frac{1}{2}}$$  \hspace{1cm} (2.14)

Figure 2.7 shows the theoretical distribution of octahedral shear strain ($\gamma_{oc}$) with radial distance from a circular mandrel. At the distance $d_s$ (smear zone), the estimated theoretical strain is approximately 5%, based on Equation 2.14.

Figure 2.7 Approximation of the smear zone around the mandrel.
2.4.2 Well resistance

The resistance to the water flowing in the vertical drains is known as the well resistance. The well resistance increases with the increase in the length of the drain and reduces the consolidation rate. Well resistance retards the pore pressure dissipation, hence, retards the settlement. The three main factors, which increase of well resistance, are the deterioration of the drain filter (reduction of drain cross section), passing of fine soil particles through the filter (reduction of drain cross section) and folding of the drain because of large settlement or lateral movement. However, these aspects are still difficult to quantify. Hansbo (1979, 1981) presented a closed form solution, which includes the effect of well resistance on drain performance.

2.5 Influence Zone of Drains

Vertical drains are commonly installed in square or triangular patterns (Fig. 2.8). As illustrated in Figure 2.8, the influence zone ($R$) is a controlled variable, since it is a function of the drain spacing ($S$) as given by:

\[
R = 0.546 S \text{ for drains installed in a square pattern, and} \tag{2.15}
\]

\[
R = 0.525 S \text{ for drains installed in a triangular pattern} \tag{2.16}
\]
A square pattern of drains may be easier to lay out and control during installation in the field, however, a triangular pattern is usually preferred since it provides a more uniform consolidation between drains than the square pattern.

### 2.6 Development of Vertical Drain Theory

The basic theory of radial consolidation around a vertical sand drain system is an extension of Terzaghi’s (1925) one-dimensional consolidation theory. Since it is obvious that the coefficient of consolidation in the horizontal direction is much higher than that in the vertical direction, and that the vertical drains reduce the drainage path considerably in the radial direction, the effectiveness of vertical drains in accelerating the rate of consolidation and improving the strength of soft soil is remarkably improved. The theory of vertical drain was probably first solved by Kjellman (1948). His solution
based on *equal vertical strain hypothesis*, was developed on the assumption that horizontal sections remain horizontal throughout the consolidation process. However, Barron (1948) presented the most comprehensive solution to the problem of radial consolidation by drain wells. He studied the two extreme cases of free strain and equal strain and showed that the average consolidation obtained in these cases is nearly the same. The ‘free strain hypothesis’ assumes that the load is uniform over a circular zone of influence for each vertical drain, and that the differential settlements occurring over this zone have no effect on the redistribution of stresses by arching of the fill load. The ‘equal vertical strain hypothesis’ on the other hand, assumes that arching occurs in the upper layer during the consolidation process without any differential settlement in the clay layer. The arching effect implies a more or less rigid boundary at the surface of the soil layer being consolidated with vertical drains. It means that the vertical strain is uniform in the horizontal section of the soil.

Barron (1948) also considered the influence of well resistance and smear on the consolidation process due to vertical well drains. Takagi (1957) extended Barron’s solution to incorporate a variable rate of loading, whereas Richart (1959) presented a convenient design chart for the effect of smear, where the influence of variable void ratio was also considered. Hansbo (1960) presented a solution by pointing out that the Darcy’s law might not be valid when the hydraulic gradient is in the range of magnitudes prevailing during most consolidation processes in practice. However, in this equal strain solution, the effect of smear and well resistance were not considered. Later, a simplified solution to the problem of smear and well resistance was proposed by Hansbo (1979,
1981), giving results almost identical with those given by Barron (1948) and Yoshikuni and Nakanodo (1974). Onoue (1988b) presented a rigorous solution based on the free strain hypothesis.

2.6.1 Equal vertical strain hypothesis (Barron, 1948)

The first conventional procedure for predicting radial consolidation was introduced by Barron (1948). This procedure was based on the extension of the theory of consolidation initially presented by Terzaghi (1925). Barron’s solution is based on the following assumptions: (a) all vertical loads are initially carried by excess pore water pressure, \( u \), which means that the soil is saturated, (b) the applied load is assumed to be uniformly distributed and all compressive strain within the soil occurs in the vertical direction, (c) the zone of influence of the drain is assumed to be circular and axisymmetric, (d) permeability of the drain is infinite in comparison with that of the soil, and (e) Darcy’s law is valid.

Barron (1948) developed the exact (rigorous) solution of vertical drain based on ‘free strain hypothesis’ and an approximate solution based on ‘equal strain hypothesis’. The difference in the predicted pore water pressures calculated using the free strain and equal strain assumptions are shown to be very small. Therefore, the approximate solution based on the ‘equal strain hypothesis’ gives satisfactory results compared to the rigorous free strain hypothesis.
The Figure 2.9 shows the schematic illustration of a soil cylinder with a central vertical drain, where, $r_w =$ the radius of the drain, $r_s =$ the radius of smear zone, $R =$ the radius of soil cylinder and $l =$ the length of the drain installed into the soft ground. The coefficient of permeability in the vertical and horizontal directions are $k_v$ and $k_h$, respectively and $k_h'$ is the coefficient permeability in the smear zone. The three dimensional consolidation of radial drainage (Barron, 1948) is given by:

$$\frac{\partial u}{\partial t} = c_v \left( \frac{\partial^2 u}{\partial z^2} \right) + c_h \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right)$$  \hspace{1cm} (2.17)

where $t$ is the time elapsed after the load is applied, $u$ is the excess pore water pressure at radius $r$ and at depth $z$.

For radial flow only, the above equation becomes:
\[
\frac{\partial \bar{u}}{\partial t} = c_h \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right)
\]  
(2.18)

The solution of the excess pore pressure for radial flow only, \(u_r\) of the above equation based on “equal strain” assumption is given by:

\[
u_r = \frac{4\bar{u}}{D^2 F(n)} \left[ R^2 \ln \left( \frac{r}{r_w} \right) \frac{r^2 - r_w^2}{2} \right]
\]  
(2.19)

where, \(D\) is the diameter of soil cylinder, the drain spacing factor, \(F(n)\), is given by:

\[
F(n) = \frac{n^2}{n^2 - 1} \ln(n) - \frac{3n^2 - 1}{4n^2}
\]  
(2.20)

where, \(n = \frac{R}{r_w}\) is drain spacing ratio, and the average excess pore water pressure is given by:

\[
\bar{u}(=u_{av}) = u_o \exp \left( \frac{-8T_h}{F(n)} \right)
\]  
(2.21)

The average degree of consolidation, \(\bar{U}_h\), in the soil body is given by:

\[
\bar{U}_h = 1 - \exp \left( \frac{-8T_h}{F(n)} \right)
\]  
(2.22)

where the time factor \(T_h\) is defined as:

\[
T_h = \frac{c_h t}{D^2}
\]  
(2.23)

The coefficient of radial drainage consolidation, \(c_h\), is represented by:

\[
c_h = \frac{k_h (1 + e)}{a_w Y_w}
\]  
(2.24)
where $\gamma_w$ is unit weight of water and $a_v$ is the coefficient of compressibility of the soil, $e$ is the void ratio, and $k_h$ is the horizontal permeability of the soil. The solution of Equation 2.18 taking account of smear effect is given by:

$$u_r = \bar{u}r \frac{1}{V} \left[ \ln \frac{r}{r_s} - \frac{r^2 - r_s^2}{2R^2} + \frac{k_h}{k_h'} \left( \frac{n^2 - s^2}{n^2} \right) \ln s \right]$$

(2.25)

where the smear factor $\nu$ is given by:

$$\nu = F(n, s, k_h, k_h') = \left[ \frac{n^2 - s^2}{n^2} \ln \frac{n}{s} - \frac{3}{4} + \frac{s^2}{4n^2} + \frac{k_h}{k_v} \left( \frac{n^2 - s^2}{n^2} \right) \ln s \right]$$

(2.26)

and

$$\bar{u}(= u_{av}) = u_v \exp\left( \frac{-8T_h}{\nu} \right)$$

(2.27)

In the above expression, $s$ is the extent factor of the smear zone with respect to the size of the drain and is given by: $s = r_v/r_w$.

The average degree of consolidation including smear is now given by:

$$\bar{U}_h = 1 - \exp\left( \frac{-8T_h}{\nu} \right)$$

(2.28)

Curves of average radial excess pore water pressure, $\bar{u}_r$, and average degree of consolidation, $\bar{U}_h$ (purely radial flow) versus time factor $T_h$ for various values of $n$ are shown in Figure 2.10. The average degree of consolidation, $\bar{U}_v$ due to vertical flow versus time factor $T_v$ is also indicated.
Illustrated application in practice

A road embankment is constructed on top of a 9.2m thick layer of clay, sandwiched between silty sand at the top, and dense sand at the bottom. The required degree of consolidation before the embankment construction is 90%, within 9 months. For this purpose, sand drains of 450mm diameter, need to be installed in a square arrangement. Estimate the spacing of the drain. From laboratory tests, assume that \( c_h = 0.288 \text{ m}^2/\text{month} \) and \( c_v = 0.187 \text{ m}^2/\text{month} \)

Solution

\[
T_v = \frac{c_d}{H^2}
\]

where, \( H = \) drainage path = 9.6/2 = 4.6m
Hence, \( T_r = \frac{0.187}{4.6^2} \times 9 \)

\[ = 0.08 \]

From Figure 8, \( U_v = 0.32 \)

\[ 1 - U = (1 - U_r)(1 - U_v) \] - Refer section 8

\[ 1.0 - 0.9 = (1 - U_r)(1.0 - 0.32) \]

\[ U_r = 0.85 \]

From Figure 8, assuming \( n_1 = 5 \) \( T_h = 0.25 \), and from Equation (9),

\[ D = \left( \frac{c_h t}{T_h} \right)^{1/2} \]

\[ = \left( \frac{0.288 \times 9}{0.25} \right)^{1/2} \]

\[ = 3.219 \text{m} \]

\[ n_2 = \frac{R}{r_w} = 3.219/2 \times 0.225 = 7 \]

If \( n_1 \) and \( n_2 \) are not very close, assume another value for \( n \) and repeat the calculation.

If \( n = 7 \), then \( D = 3.219 \text{m} \)

Since the vertical drains are installed in a square pattern,

From Equation 40 (Refer section 7),

Drain spacing, \( S = 3.219/1.13 = 2.85 \text{m} \)
2.6.2 Approximate equal strain solution (Hansbo, 1981)

Hansbo (1981) derived an approximate solution for vertical drain based on the ‘equal strain hypothesis’ by taking both smear and well resistance into consideration. The general concept of this solution is the same as illustrated previously in Figure 2.9. By applying Darcy’s law, the rate of flow of internal pore water in the radial direction can be estimated. The total flow of water from the slice, $dz$, to the drain, $dQ_1$, is equal to the change of flow of water from the surrounding soil, $dQ_2$, which is proportional to the change of volume of the soil mass. The average degree of consolidation, $\bar{U}$, of the soil cylinder with vertical drain is given by:

$$\bar{U}_h = 1 - \exp\left(-\frac{8T_{h'}}{\mu}\right)$$

(2.29)

$$\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k'_{h'}}\right) \ln(s) - 0.75 + \pi z (2l - z) \frac{k_h}{q_w} \left\{ 1 - \frac{k_h / k'_{h'} - 1}{(k_h / k'_{h'}) (n / s)^2} \right\}$$

(2.30)

Or, in a simplified form:

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

(2.31)

The effect of smear only is given by:

$$\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k'_{h'}}\right) \ln(s) - 0.75$$

(2.32)

The effect of well resistance only is given by:

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

(2.33)

If both smear and well resistance are ignored, this parameter becomes,

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

(2.34)
2.6.3 \( \lambda \) method (Hansbo, 1979 and 1997)

Although the classical theory of consolidation of vertical drains (Barron, 1948) and its later developments are all based on the validity of Darcy’s law, in the consolidation process, however, the permeability is subjected to gradual reduction. The pore water flow velocity, \( v \) caused by a hydraulic gradient, \( i \) might be deviated from the original Darcy’s law \( v = k i \), where under a threshold gradient \( i_o \) below which no flow takes place, the rate of flow is then given by: \( v = k (i - i_o) \). The following relations were proposed:

\[
\begin{align*}
v & = \kappa i^n \quad \text{for} \quad i \leq i_t \\
v & = k(i - i_o) \quad \text{for} \quad i \geq i_t
\end{align*}
\]

where \( i_t = \frac{i_o n}{(n - 1)} \) and \( \kappa = (n^{-1}i^{-n})k \)

In order to study the effect of the non-Darcian flow, Hansbo (1979, 1997) proposed an alternative consolidation equation, which is supported by the full-scale field test at Ska-Edeby, Sweden. The average degree of consolidation is the same as given in Equation 2.22. The time required to reach a certain average degree of consolidation including smear effect is given by:

\[
t = \frac{\alpha D^2 \left( \frac{D\gamma_w}{u_o} \right)^{n-1}}{\lambda \left( \frac{1}{1 - U_h} \right)^{n-1} - 1}
\]

(2.35)

where the coefficient of consolidation \( \lambda \) is given by \( \frac{\kappa M}{\gamma_w} \), \( M = 1/M_v \)=oedometer modulus, \( D \) is the diameter of the influence zone of the drain, \( d_s \) is the diameter of smear
zone, \( n = D/d_w \), \( d_w \) is the diameter of the drain, \( u_o \) is the initial average excess pore water pressure, \( \alpha \) is \( \frac{n^{2n} \beta^n}{4(n - 1)^{n+1}} \) and

\[
\beta = \frac{1}{3n-1} - \frac{n-1}{n(3n-1)(5n-1)} - \frac{(n-1)^2}{2n^2(5n-1)(7n-1)} + \frac{1}{2n} \left[ \left( \frac{\kappa_h}{\kappa_s} - 1 \right) \left( \frac{D}{d_s} \right)^{-(1-1/n)} \right] - \frac{\kappa_h}{\kappa_s} \left( \frac{D}{d_w} \right)^{-(1-1/n)} \]

When \( n \to 1 \) Equation 2.35 yields the same result as the average degree of consolidation given by Equation 2.22 provided that well resistance is ignored and assuming \( \lambda = c_h \) and \( \kappa_h/\kappa_s = k_h/k_s \).

### 2.6.4 Plane strain consolidation model (Indraratna & Redana, 1997)

Most finite element analyses on embankments are conducted based on the plane strain assumption. However, this kind of analysis poses a problem, because the consolidation around vertical drains is axisymmetric. Therefore, to employ a realistic 2-D finite element analysis for vertical drains, the equivalence between the plane strain and axisymmetric analysis needs to be established.

The matching of axisymmetric and plane strain conditions can be done in three ways:

1. geometric matching approach – the spacing of the drains is matched while keeping the permeability the same
2. permeability matching approach – permeability coefficient is matched while keeping the spacing of drains to be the same.

3. combination of permeability and geometric matching approach – plane strain permeability is calculated for a convenient drain spacing.

Indraratna & Redana (1997) converted the vertical drain system shown in Figure 2.9 into equivalent parallel drain well by adjusting the coefficient of permeability of the soil and by assuming the plane strain cell to have a width of 2B as shown in Figure 2.11. The half width of drains \( b_w \) and half width of smear zone \( b_s \) may be taken to be the same as their axisymmetric radii \( r_w \) and \( r_s \) respectively, which gives;

\[
\begin{align*}
  b_w &= r_w \\
  b_s &= r_s
\end{align*}
\]  

(2.36)

The equivalent drain diameter \( (d_w) \) or radius \( r_w \) for band drains can be determined by ‘perimeter equivalence’ (Hansbo, 1979) given in Equation 2.1

Rixner et al. (1986) presented the equivalent drain diameter \( d \) as the average of drain thickness and width by considering the shape of the drain and effective drainage area as given in Equation 2.2.

Indraratna & Redana (1997) represented the average degree of consolidation in plane strain condition as;

\[
\overline{U}_{hp} = 1 - \frac{\bar{u}}{u_o} = 1 - \exp \left( \frac{-8T_{hp}}{\mu_p} \right)
\]  

(2.37)
where $\bar{u}_0$ = initial pore pressure, $\bar{u}$ = pore pressure at time $t$ (average values), $T_{hp}$ = time factor in plane strain, and

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

(2.38)

Figure 2.11 Conversion of an axisymmetric unit cell into plane strain condition

where, $k_{hp}$ and $k_{hp}'$ are the undisturbed horizontal and corresponding smear zone permeabilities, respectively. The geometric parameters $\alpha$, $\beta$ and the flow term $\theta$ are given by:
\[ \alpha = 2 - \frac{2b_t}{B} \left( l - \frac{b_t}{B} + \frac{b_t^2}{3B^2} \right) \]

(2.38a) \[ \beta = \frac{l}{B^2} \left( b_t - b_w \right)^2 + \frac{b_t^2}{3B^2} \left( 3b_t^2 - b_t^4 \right) \]

(2.38b) \[ \theta = \frac{2k_w^2}{k_q z B} \left( l - \frac{b_t}{B} \right) \]

(2.38c)

where \( q_z \) = the equivalent plane strain discharge capacity.

At a given stress level and at each time step, the average degree of consolidation for both axisymmetric (\( \bar{U}_h \)) and equivalent plane strain (\( \bar{U}_{hp} \)) conditions are made equal, hence:

\[ \bar{U}_h = \bar{U}_{hp} \]

(2.39)

Combining Equations 2.37 and 2.39 with the original Hansbo (1981) theory, Equation 2.18, the time factor ratio can be given by following equation:

\[ \frac{T_{hp}}{T_h} = \frac{k_{hp} R^2}{k_h B^2} = \frac{\mu_p}{\mu} \]

(2.40)

By assuming the magnitudes of \( R \) and \( B \) to be the same, Indraratna and Redana (1997) presented the relationship between \( k_{hp} \) and \( k_{hp}' \) as follows;

\[ k_{hp} = \frac{k_h \left[ \alpha + (\beta) \frac{k_{hp}}{k_{hp}'} + (\theta)(2lz - z^2) \right]}{\ln \left( \frac{n}{s} \right) + \left( \frac{k_h}{k_h'} \right) \ln(s) - 0.75 + \pi \left( 2lz - z^2 \right) \frac{k_h}{q_w} } \]

(2.41)

If well resistance is ignored in Equation 2.41 by omitting all terms containing \( l \) and \( z \), the influence of smear effect can be represented by the ratio of the smear zone permeability to the undisturbed permeability as follows:
\[
\frac{k'_{hp}}{k_{hp}} = \frac{k_{hp}}{k_h} \left[ \beta \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right) \ln(s - 0.75) \right] - \alpha
\]  

(2.42)

If both smear and well resistance effects are ignored in Equation 2.41, then the simplified ratio of plane strain to axisymmetric permeability is readily obtained, as proposed earlier by Hird et al. (1992):

\[
\frac{k'_{hp}}{k_h} = \frac{0.67}{\ln(n) - 0.75}
\]  

(2.43)

The well resistance is derived independently and yields an equivalent plane strain discharge capacity of drains as also proposed earlier by Hird et al. (1992):

\[
q_z = \frac{2}{\pi B} q_w
\]  

(2.44)

2.7 Consolidation around Vertical Drains

2.7.1 Rate of consolidation

The main reason for using pre fabricated vertical drain is to reach the desired degree of consolidation within a specified time period. But in a vertical drain system, both radial and vertical consolidation should be considered in calculating the specified time period. Carillo (1942) gave the combined effect as:

\[
1 - U = (1 - U_r)(1 - U_v)
\]  

(2.45)

where

- U – overall degree of consolidation
- U_r – average degree of consolidation due to radial drainage
- U_v – average degree of consolidation due to vertical drainage
2.7.2 Coefficient of Consolidation with Radial Drainage

2.7.2.1 Log $U$ vs $t$ approach

Aboshi and Monden (1963) presented a curve fitting method using log $U$ and linear $t$. This method is developed by taking ‘log’ on both sides of Barron’s solution (Equation 2.22), which results in the following expression:

$$T_h = -\frac{F(n)}{8} \ln(1-U)$$  \hspace{1cm} (2.46)

Equation 2.46 represents the theoretical time factor for radial consolidation of perfect drains without considering the effect of smear. The coefficient of radial consolidation $c_h$ is determined by plotting the logarithm of the average degree of consolidation against the linear consolidation time ($\log U$ vs $T_h$), where a linear slope provides the $c_h$ value (Figure 2.12).

![Figure 2.12 Aboshi and Monden (1963) method for determining $c_h$](image-url)
In order to include the effect of smear, ‘log’ is taken on both sides of Equation 2.28, which yields a new time factor $T_h$ as given by:

$$T_h = -\frac{V}{8} \ln(I - \overline{U})$$  \hspace{1cm} (2.47)

By substituting $c_h = \frac{T_h H_d^2}{t}$ yields,

$$c_h = -\frac{D^2 \nu H_d \ln(I - \overline{U})}{8}$$  \hspace{1cm} (2.48)

By using Equation 2.28, the coefficient of radial drainage consolidation $c_h$ can be defined from the slope of log $U$-$t$ curve with settlement data. The pore water pressure data can also be plotted other than settlement, in this method.

2.7.2.2 Plotting settlement data (Asaoka, 1978)

Asaoka (1978) developed a method where a series of settlements ($\rho_1, \ldots, \rho_{t-1}, \rho_i, \rho_{i+1}$ etc.), which are observed at constant time intervals are plotted as shown in Figure 2.13. The coefficient of radial drainage consolidation in this method is derived using Barron’s (1948) solution, which is given by:

$$c_h = -\frac{D^2 \nu \ln \beta}{8 \Delta t}$$  \hspace{1cm} (2.49)

where, $\nu$ is expressed in Equation 2.26, $\beta$ is the slope of the line formed by the observed displacement data, and $\Delta t$ is the time interval between observations. To obtain the average coefficient of consolidation, $\nu$ (smear factor) is replaced by drain spacing factor, $F(n)$ as expressed in Equation 2.19 which gives:
Figure 2.13 Asaoka (1978) method to determine $c_h$

2.8 Effect of Horizontal to Vertical Permeability Ratio

Generally, the coefficient of permeability can be determined indirectly from conventional oedometer test using Terzaghi’s one dimensional consolidation theory. This indirect method of determining coefficient of permeability leads to some error due to the assumption of constant coefficient of permeability, constant coefficient of volume change and constant coefficient of consolidation ($k$, $m_v$ and $c_v$), respectively, during the consolidation process. The modified triaxial and oedometer tests equipped with the permeameter produce more reliable values of permeability. The falling head
permeability test conducted on the modified oedometer appears to be the best (simpler and faster) to determine permeabilities of natural clays.

The permeability characteristics of a number of intact clays determined using the modified apparatus explained above is reported later by Tavenas et al. (1983b). In these tests, the horizontal permeability was also determined using samples rotated horizontally (90°) and of intermediate inclination, 45°. For marine clays recovered from Champlain sea formation (Canada), the anisotropy ratio \( r_k = k_h/k_v \) estimated using the modified oedometer test was found to vary between 0.91 and 1.42 with an average of 1.1. In triaxial testing, this ratio was found to be in the range of 0.81-1.16 with an average of 1.03, which implies that anisotropy was not an influence factor in this soil.

According to the experimental results plotted in Figure 2.6 (Indraratna & Redana, 1995), the value of \( k_h' / k_v' \) in the smear zone varies between 0.9 and 1.3 with an average of 1.15. Hansbo (1987) argued that for extensive smearing, the horizontal permeability coefficient in the smear zone \( (k_h') \) should approach that of the vertical permeability coefficient \( (k_v') \), thus suggesting that the ratio \( k_h' / k_v' \) could approach 1. The experimental results shown in Figure 2.6 (Indraratna & Redana, 1995) seem to be in agreement with Hansbo (1987). For the applied consolidation pressures, it is observed that the value of \( k_h/k_v \) varies between 1.4 and 1.9 with an average of 1.63 in the undisturbed zone. Shogaki et al (1995) reported that the average values of \( k_h/k_v \) were in the range of 1.36-1.57 for undisturbed isotropic soil samples taken from Hokkaido to Chugoku region in Japan. Tavenas et al. (1983b) reported that for the soil tested in
conventional oedometer, the $k_h/k_v$ ratio was found to vary between 0.91 and 1.42 for intact natural clays, and from 1.2 to 1.3 for Matagami varved clay. Bergado et al. (1991) conducted a thorough laboratory study on the development of smear zone in soft Bangkok clay, and they reported that the horizontal permeability coefficient of undisturbed zone to the smear zone varied between 1.5 and 2 with an average of 1.75. More significantly, the ratio of $k_h^i / k_v^i$ was found to be almost unity within the smear zone.

2.9 Application of Vacuum Preloading

Kjellman (1952) was the first to propose vacuum assisted preloading to accelerate the rate of consolidation. Very few studies of vacuum assisted consolidation have continued for the next 2 decades (Holoton et al., 1965; Holtz, 1975). However, except for specialized applications like landslide stabilization, vacuum consolidation was not seriously regarded as an alternative to surcharge preloading until recently, due to (i) the low cost of placing and removing surcharge fills and (ii) the difficulties involved in applying and maintaining the vacuum pressure. However, with the increase of direct and indirect costs of placing and removing surcharge fill and the improved technology for sealing landfills with impervious membranes, vacuum assisted consolidation has now become an economical alternative to conventional surcharge preloading. Since then, field trials have been carried out in several countries (Park et al., 1997; Choa, 1989; Tang et al., 2000; Chu et al., 2000; Bergado et al., 1998), and these studies have verified the effectiveness of vacuum assisted consolidation in conjunction with vertical drains.
The field data obtained from the soil improvement project in China (Chu et al., 2000) have shown that vacuum preloading is still effective even for soils 20m below the ground surface. Similar conditions have been observed by Choa (1989) for a development project in Tianjin Harbour, China. The field data also show that in the vacuum preloaded areas, the lateral displacements over the top 10m are about 0.25 to 0.5m inward, while in the surcharge area the movement is about 0.3 to 0.5m outwards. This confirms that vacuum pressure application directly contributes to reducing lateral yield, thereby improving the soft soil stability. Another attempt was made by Leong et al. (2000) to evaluate the difference in surcharge and vacuum preloading methods quantitatively. The experimental results conclude that surcharge preloading produces greater shear strength than vacuum preloading for equivalent loading conditions. Surprisingly, the data indicate that the increase in matric suction greater than 100 kPa would actually lead to a reduction in shear strength.

2.10 Soft Clay Modelling

In order to predict the behaviour of an engineering structure, it is necessary to use an appropriate constitutive model, which represents the stress-deformation response of the material. Deformation analysis in geotechnical engineering often assumes a linear elastic material at small stresses. This assumption probably holds true for over-consolidated clay, however, most soils exhibit plastic behaviour at increased stresses.

The theories of critical state soil mechanics have been developed based on the application of the theory of plasticity. Utilising the critical state concept based on the
theory of plasticity in soil mechanics, a more sophisticated Cam-clay model has been introduced to represent the behaviour of clay (Schofield & Wroth, 1967). The Cam-clay model has received wide acceptance due to its simplicity and accuracy to model clay behaviour, especially for normally and lightly overconsolidated clay. In this model, the shear strength of the soil is related to the void ratio. To describe the state of soil during triaxial testing, the following critical state parameters are defined by:

\[ p' = \frac{\sigma'_i + 2\sigma'_3}{3} = \sigma_i + 2\sigma_3 - u \]  
(2.51)

\[ q = \sigma'_i - \sigma'_3 = \sigma_i - \sigma_3 \]  
(2.52)

where, \( \sigma'_i \) represents the effective axial stress, \( \sigma'_3 \) represents the effective confining stress and \( u \) is the pore water pressure.

In critical state theory, the virgin compression, swelling and recompression lines are assumed to be straight lines in \((\ln p' - V)\) plots with slope \(-\lambda\) and \(-\kappa\) respectively, as shown in Figure 2.14. The isotropic virgin compression line or isotropic normal consolidation line (NCL) is expressed as:

\[ V = N - \lambda \ln(p') \]  
(2.53)

where, \( N(V_\lambda) \) is the value of \( V \) when \( \ln p \leq 0 \) or \( p \leq 1 \).
The swelling or recompression line is expressed as:

\[ V = V_\kappa - \kappa \ln(p') \]  

(2.54)

The relation between \( C_c \) (compression index) and \( \lambda \) may be expressed as:

\[ \lambda = \frac{C_c}{2.307} \]  

(2.55)

The slope of the straight line in \( q - p' \) plot is called Critical State Line (CSL) as shown in Figure 2.15. The slope of the critical state line, \( M \) is expressed as:

\[ q = Mp' \]  

(2.56)

In the \( V - \ln p' \) plot, if \( \Gamma \) is used to represent the value of \( V_\lambda \) which corresponds to a \( \ln p' \leq 0 \) (i.e. \( \Gamma = e_{sc} + 1 \)), then the equation of the straight line is given by:

\[ V = \Gamma - \lambda \ln p', \text{ or } p' = \exp \frac{\Gamma - V}{\lambda} \]  

(2.57)

Hence, the critical state line must satisfy both Equations 2.51 and 2.52.
Figure 2.15 Position of the critical state line

Figure 2.16 Position of the initial void ratio on critical state line
Combining the CSL equation into the Mohr circle plot, the relationship between drained angle of friction \( \phi' \) and \( M \) may be given by:

\[
M = \frac{6 \sin \phi'}{3 - \sin \phi'}
\]  

(2.58)

The initial void ratio can be estimated at any given depth below the ground level once \( p' \), \( q \) and \( P'_c \) are known, and the \( e-ln p' \) plot is shown in Figure 2.16. The parameter \( e_{cs} \) is defined as the voids ratio on the critical state line for a value of \( p'\equiv 1 \).

The intersection between the swelling line and the insitu-stress line is assumed to be at point A and given by coordinates \( e_A \) and \( P'_A \). Point P represents the intersection between the initial void ratio, \( e \) and the effective mean normal stress \( p' \). Then, the following relation may be established:

\[
e_A = e_{cs} - \lambda \ln P'_A
\]  

(2.59)

where, \( P'_A = \frac{P'_c}{2} \) for Modified Cam-clay and \( P'_A = \frac{P'_c}{2.718} \) for Cam-clay.

Along the swelling line (\( \kappa \) line) passing through the initial stress state at point P, the following relation can be applied:

\[
e-e_A = \kappa \left( \ln p' - \ln P'_A \right)
\]  

(2.60)

Substituting \( e_A \) from Equation 2.59 gives:

\[
e_{cs} = e + \left( \lambda + \kappa \right) \ln P'_A - \kappa \ln p'
\]  

(2.61)
2.10.1 Modified Cam-clay

It was found that the original Cam-clay model was deficient in some aspects of modelling the stress-strain behaviour of soil. Two aspects of dissatisfaction were: the shape of the yield locus at increased $p'_c$ and the predicted value of $K_o$ (the coefficient of earth pressure at rest). Therefore, the modified Cam-clay was introduced to address those problems (Roscoe and Burland, 1968). The obvious difference between modified Cam-clay and Cam-clay model is the shape of the yield locus, where the yield locus of modified Cam-clay is elliptical as shown in Figure 2.17b. The flow rule for modified Cam-clay is given by:

$$\frac{\delta \nu^p}{\delta \varepsilon^p} = \frac{M^2 - \eta^2}{2\eta}$$

(2.62)

where, $\delta \nu^p$ and $\delta \varepsilon^p$ are volumetric and deviatoric plastic strain increments, respectively, and $\eta = \frac{q}{p'}$ represents the stress ratio.

![Figure 2.17 The yield locus of Cam-clay and modified Cam-clay](image.png)
The modified Cam-clay yield locus is given by:

\[ q + M^2 p' \tau^2 = M^2 p' \tau' \]  

(2.63)

The equation of the Stable State Boundary Surface (SSBS) is given by:

\[ V_{\kappa} = \Gamma + (\lambda - \kappa)(\ln(2) - \ln(1 + (\eta / M)^2)) \]  

(2.64)

2.11 Salient Aspects of Numerical Modelling

Currently, pore pressures, settlements, lateral displacements and stresses of the vertical drain installed field site can be analysed as accurately as possible using sophisticated finite element software. Commercial packages such as ABAQUS, PLAXIS and SAGE-CRISP, are capable of performing fully coupled consolidation analysis. According to past experience, finite element analysis of lateral deformation has been relatively poor in contrast to settlements (Indraratna et al., 1994). The recent finite element models applied to vertical drains are described below.

2.11.1 Drain efficiency by pore pressure dissipation (Indraratna et al., 1994)

In this study, the performance of embankment stabilised with vertical drains at Muar clay, Malaysia was analysed using the finite element code, CRISP (Britto and Gunn, 1987). The effectiveness of the prefabricated drains was evaluated according to the rate of excess-pore pressure dissipation at the soil drain interface. Both single and multi-drain (whole embankment) analyses were carried out to predict the settlement and lateral deformation beneath the embankment.
A plane strain analysis was applied to a single drain and to the whole PVD scheme. An axisymmetric horizontal permeability was used in the plane strain model. As explained in detail by Ratnayake (1991), the prediction of settlement using the single drain analysis over-predicts the measured settlement, even though the effect of smear is included. In the case of multi-drain analysis underneath the embankment, the over-prediction of settlement is more significant compared to the single drain analysis. Therefore, to enable better predictions, it was necessary to consider more accurately, the dissipation of the excess pore pressures at the drain boundaries at a given time.

Figure 2.18 Percentage of undissipated excess pore pressures at drain-soil interfaces (Indraratna et al., 1994)

In order to elaborate this technique, the average undissipated excess pore pressures could be estimated by finite element back-analysis of the settlement data at the centreline of the embankment as shown in Figure 2.18. In this figure, 100% represents zero
dissipation when the drains are fully loaded. Accordingly, at the end of the first stage of consolidation (ie., 2.5 m of fill after 105 days), the undissipated pore pressures decrease from 100% to 16%. For the second stage of loading, the corresponding magnitude decreases from 100% to 18% after a period of 284 days, during which the height of the embankment has already attained the maximum of 4.74 m. It can be deduced from Figure 2.18 that perfect drain conditions are approached only after a period of 400 days. Although the general trends between the finite element results and field data are in agreement especially during the initial stages, the marked discrepancy beyond 100 days is too large to be attributed solely to the plane strain assumption. These excess pore pressures reflect the retarded efficiency of the vertical drains (partial clogging). A better prediction was obtained for settlement, pore pressure and lateral deformation when ‘non-zero’ excess pore pressures at drain interface were input into the finite element model, simulating ‘partially clogged’ conditions. The ‘non zero’ excess pore pressures can also represent the smear effect that contributes to decreased efficiency, as discussed later.

2.11.2 Matching permeability and geometry (Hird et al., 1995)

Hird et al. (1992, 1995) presented a modelling technique of vertical drains in two-dimensional finite element analysis using CRISP (Britto and Gunn, 1987). In this analysis, the permeability and geometry matching were applied to several embankments stabilised with vertical drains in Porto Tolle (Italy), Harlow (UK) and Lok Ma Chau (Hongkong).
An acceptable prediction of settlements was obtained, although the pore water pressure dissipation was more difficult to predict. However, at Lok Ma Chau (Hongkong), the settlements were significantly over-predicted. This was because, in this case history analysis, the effect of smear was not considered although the plane strain model proposed by Hird et al (1992) allows the smear effect to be modelled.

At Porto Tolle embankment, prefabricated vertical geodrains were installed on a 3.8 m triangular grid to a depth of 21.5 m below ground level. The equivalent radius of geodrain was 31 mm, and its discharge capacity was conservatively estimated at about 140 m³/year. The embankment, which was constructed over a period of 4 months, had a height of 5.5 m, a crest width of 30 m, a length of over 300 m, and a side slope of about 1 in 3. For the purpose of finite element analysis, the clay was modelled using the modified Cam-clay model of Roscoe and Burland (1968) and their parameters are as follows: \( \lambda = 0.16 \), \( \kappa = 0.032 \), \( \Gamma = 2.58 \), \( M_{ax} = 1.16 \), \( M_{pl} = 0.92 \), \( \nu = 0.3 \), and permeability was \( k_h = 4.1 \times 10^{-9} \) m/s and \( k_v = 3.5 \times 10^{-10} \) m/s. The equivalent plane strain permeability is estimated according to Hird et al., (1992).

In this study, a single-drain analysis at embankment centreline was considered. The typical results of finite element analysis are compared in Figure 2.19a and 2.19b. Field settlement data are also plotted in Figure 2.19a and these show that both during and after construction, the magnitude of settlement was reasonably well modelled. In Figure 2.19b the observed and computed excess pore pressures mid-way between the drains are compared; only the axisymmetric computed results are shown, since in plane strain
analysis, pore pressures are not matched at corresponding points, but merely on average. During construction, the observed and computed pore pressures compared tolerably well. Afterwards, they differed greatly, although the field data may have been unreliable because of the presence of organic gas in the soil (Jamiolkowski and Lancellotta, 1984).

Figure 2.19 Result of axisymmetric and matched plane strain for Porto Tolle embankment: a) average surface settlement and b) excess pore pressure (Hird et al., 1995)
2.11.3 Modelling of discharge capacity (Chai et al. 1995)

Chai et al. (1995) extended the method proposed by Hird et al. (1992) to include the effect of well resistance and clogging. Four types of analyses were presented considering: (1) no vertical drains, (2) embankment with vertical drains and discharge capacity of the drains increasing with depth, (3) drains with constant discharge capacity and (4) the same as (3) but assuming that the drains were clogged below 9 m depth.

The discharge capacity of the drain in plane strain for matching the average degree of horizontal consolidation is given by:

\[ q_{wp} = \frac{4k_h l^2}{3B \left[ ln \left( \frac{n}{s} \right) + \frac{k_h}{k_s} ln(s) - \frac{17}{12} + \frac{2l^2 \pi k_h}{3q_{wa}} \right]} \]  

(2.65)

The model developed in this study was refined using a single drain model of 5 m long, and both elastic and elasto-plastic analyses were applied to predict its performance. Excellent agreement was obtained between axisymmetric and plane strain models especially with varied discharge capacity \( q_{wp} \) as shown in Figure 2.20, for elasto-plastic analysis. The well resistance matching also results in a more realistic excess pore water pressure variation with depth as shown in Figure 2.21, for elastic analysis. It can be seen that the varied discharge capacity yielded a more uniform and closer match between axisymmetric and plane strain methods compared to constant discharge capacity assumption.
Figure 2.20 Comparison of average degree of horizontal consolidation (Chai et al., 1995)

The model described above was calibrated verified with the performance of an embankment stabilised with vertical drains founded in Muar clay, Malaysia. This study shows that the vertical drains not only increase the settlement rate, but also reduce the lateral deformation. A more realistic excess pore pressure distribution was also obtained when the well resistance and clogging were introduced in the analysis.

Figure 2.21 Comparison of excess pore pressure variation with depth (Chai et al., 1995)
2.11.4 Deformation as a stability indicator

Indraratna et al. (1997) investigated the effect of ground improvement by preloading together with geogrid and vertical band drains, as well as sand compaction piles constructed on Muar clay, Malaysia. The settlement and lateral displacement of the soft clay foundation were analysed using the plane strain finite element formulation, and the findings were compared to the field measurements. In order to conduct a two-dimensional plane strain analysis, the vertical drain system was converted into an equivalent drainage wall as explained earlier (Fig 2.11).

The analysis employed critical state soil mechanics, and the deformations were predicted on the basis of the fully coupled (Biot) consolidation model incorporated in the finite element code CRISP (Britto and Gunn, 1987). In the analysis, the soil underneath the embankment was discretised using linear strain quadrilateral (LSQ) elements. The vertical drains were modelled as ideal and non-ideal drains, where in the former, the well resistance factor was ignored. This study shows that the accurate prediction of lateral displacement depends on the correct assessment of the value of the Cam-clay parameter $\lambda$, the shear resistance at the embankment-foundation interface and the nature of assumptions made in the modelling of drains and sand piles. The actual soil properties are influenced by the working stress range and the assumed stress path of the sub-soil at a given depth. The normally consolidated parameters associated with the Cam-clay theories over-estimate lateral displacement and settlements, if the applied stresses are smaller than the pre-consolidation pressure.
Table 2.2 Effect of ground improvement on normalised deformation factors (Indraratna et al., 1997).

<table>
<thead>
<tr>
<th>Ground improvement scheme</th>
<th>$\alpha$</th>
<th>$\beta_1$</th>
<th>$\beta_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand compaction piles for pile/soil stiffness ratio 5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(h = 9.8 m, including 1 m sand layer)</td>
<td>0.185</td>
<td>0.018</td>
<td>0.097</td>
</tr>
<tr>
<td>Geogrids + vertical band drains in square pattern at 2.0 m spacing (h=8.7 m)</td>
<td>0.141</td>
<td>0.021</td>
<td>0.149</td>
</tr>
<tr>
<td>Vertical band drains in triangular pattern at 1.3 m spacing (h = 4.75 m)</td>
<td>0.123</td>
<td>0.034</td>
<td>0.274</td>
</tr>
<tr>
<td>Embankment rapidly constructed to failure on untreated foundation (h=5.5m)</td>
<td>0.634</td>
<td>0.104</td>
<td>0.164</td>
</tr>
</tbody>
</table>

The performance of vertical band drains and sand compaction piles was compared based on normalised deformation as shown in Table 2.2. The ratio of maximum lateral displacement to fill height ($\beta_1$) and the ratio of maximum settlement to fill height ($\beta_2$) were considered as stability indicators. In comparison with an unstabilised embankment constructed to failure (Indraratna et al., 1992), the stabilised foundations are characterised by considerably smaller values for $\alpha$ and $\beta_1$, highlighting their obvious implication on stability. The normalised settlement ($\beta_2$) on its own does not seem to be a proper indicator of instability. The foundation having SCP gives the lowest values of $\beta_1$ and $\beta_2$, clearly suggesting the benefits of sand compaction piles over the band drains.
2.11.5 Application of vacuum pressure (Bergado et al., 1998)

Finite element analysis was applied by Bergado et al. (1998) to analyse the performance of embankment stabilised with vertical drains, where a combined preload and vacuum pressure were utilised at the Second Bangkok International Airport site. A simple approximate method for modelling the effect of PVD as proposed by Chai and Miura (1997) was incorporated in this study. PVD increases the mass permeability in the vertical direction. Consequently, it is possible to establish a value of permeability of the natural subsoil and the radial permeability towards the PVD. This equivalent vertical permeability \( K_{ve} \) is derived, based on equal average degree of consolidation.

The approximate average degree of vertical consolidation \( U_v \) is given by:

\[
U_v = 1 - \exp(-3.54) T_v
\]

(2.66)

where, \( T_v \) is the dimensionless time factor.

The equivalent vertical permeability, \( K_{ve} \) can be expressed as:

\[
K_{ve} = \left(1 + \frac{2.26 L^2 K_h}{F D_e^2 K_v}\right) K_v
\]

(2.67)

where:

\[
F = \ln\left(\frac{D_e}{d_w}\right) + \left(\frac{K_h}{K_s} - 1\right) \ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} + \frac{\pi 2L^2 K_h}{3q_w}
\]

(2.68)

In Eqn. 65, \( D_e \) is the equivalent diameter of a unit PVD influence zone, \( d_s \) is the equivalent diameter of the disturbed zone, \( d_w \) is the equivalent diameter of PVD, \( K_h \) and \( K_s \) are the undisturbed and disturbed horizontal permeability of the surrounding soil,
respectively, $L$ is the length for one-way drainage, and $q_w$ is discharge capacity of PVD. The effects of smear and well resistance have been incorporated in the derivation of the equivalent vertical permeability.

Table 2.3 Parameters of vertical drains

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing, $S$</td>
<td>1.0 m (triangular pattern)</td>
</tr>
<tr>
<td>Diameter of drain, $d_w$</td>
<td>50 mm</td>
</tr>
<tr>
<td>Diameter of smear zone, $d_s$</td>
<td>300 mm</td>
</tr>
<tr>
<td>Ratio of $K_h/K_s$</td>
<td>10</td>
</tr>
<tr>
<td>Drainage length, $l$</td>
<td>15 m for TV1 and 12 m for TV2</td>
</tr>
<tr>
<td>Discharge capacity, $q_w$ (per-drain)</td>
<td>50 m$^3$/year</td>
</tr>
</tbody>
</table>

Two full-scale test embankments, TV1 and TV2 each with base area of 40 x 40 m were analysed by Bergado et al.(1998). The PVDs were installed to a depth of 15 m and 12 m for embankment TV1 and TV2, respectively. The design parameters are shown in Table 2.3, and the typical cross section of embankment TV1 is shown in Figure 2.22. It can be concluded from this study that the vacuum assisted consolidation has been effectively utilised for both embankments TV1 and TV2. The performance of embankment TV2 with vacuum preloading (compared to the embankment at the same site without vacuum preloading), showed an acceleration in the rate of settlement of about 60% and a reduction in the period of preloading by about 4 months.
2.11.6 Equivalent plane strain modelling

An attempt was made by Indraratna & Redana (1997) to analyse the effect of smear zone and well resistance in a vertical drain using a 2D plane strain finite element model employing a modified Cam-clay theory. This was executed by converting the vertical drain system into equivalent parallel drain walls by adjusting the spacing of the drains and the coefficient of permeability of the soil as discussed earlier (Equations 2.36-2.44). The transformed permeability coefficient was then incorporated in the finite element code, CRISP through appropriate subroutines.

In order to verify the proposed model, a finite element analysis was executed for both axisymmetric and equivalent plane strain models. As an example, a unit drain was
considered with a drain installed to a depth of 5 m under the ground surface at 1.2 m spacing. The model parameters and the soil properties are given in Table 2.4.

Table 2.4 Model parameters and soil properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing, $S$</td>
<td>1.2m</td>
</tr>
<tr>
<td>Radius of drain, $r_w$</td>
<td>0.03m</td>
</tr>
<tr>
<td>Radius of mandrel</td>
<td>0.05m</td>
</tr>
<tr>
<td>Ratio of $K_h/K_v$ in undisturbed zone</td>
<td>2</td>
</tr>
<tr>
<td>Ratio of $K_h/K_v$ in smear zone</td>
<td>1</td>
</tr>
<tr>
<td>Plane strain permeability in undisturbed zone, $k_{hp}$</td>
<td>$2.97 \times 10^{-9}$ m/s</td>
</tr>
<tr>
<td>Plane strain permeability in smear zone, $k'_{hp}$</td>
<td>$5.02 \times 10^{-10}$ m/s</td>
</tr>
<tr>
<td>Radius of unit cell</td>
<td>0.6m</td>
</tr>
<tr>
<td>Gradient of volume vs log pressure relation for consolidation, $\lambda$</td>
<td>0.2</td>
</tr>
<tr>
<td>Gradient of volume vs log pressure relation for swelling, $\kappa$</td>
<td>0.04</td>
</tr>
<tr>
<td>Slope of critical state line, $M$</td>
<td>1.0</td>
</tr>
<tr>
<td>Void ratio at unit consolidation pressure, $e_{cs}$</td>
<td>2</td>
</tr>
<tr>
<td>Poisson’s ratio, $\nu$</td>
<td>0.25</td>
</tr>
<tr>
<td>Saturated unit weight of the soil, $\gamma_s$</td>
<td>18 kN/m$^3$</td>
</tr>
</tbody>
</table>

In the analysis with smear, the size of the smear zone was taken to be 5 times the size of the mandrel based on the experimental results. For the verification of the model a
simplified permeability variation was assumed as shown in Figure 2.23, where the coefficient of permeability within the smear zone was taken to be constant (i.e. $k'_{hp} = 5.02 \times 10^{-10} \text{ m/s}$). A higher permeability coefficient was used for the undisturbed zone ($k_{hp} = 2.97 \times 10^{-9} \text{ m/s}$). Figure 2.23 shows both the assumed axisymmetric and the converted plane strain permeability values.

![Figure 2.23 Simplified variation of permeability within and outside smear zone](image)

The results of both axisymmetric and plane strain analysis are plotted in Figure 2.24, where the average degree of radial consolidation $U_h$ (%) is plotted against the time factor $T_h$ for perfect drain conditions. As illustrated in Figure 2.24, the proposed plane strain analysis gives a good agreement with the axisymmetric analysis. The maximum difference between the two methods is less than 5%.
The well resistance can also be included using Equation 2.41. Figures 2.25 and 2.26 illustrate the settlements and excess pore pressure variations with time for single drains including smear plus well resistance. Once again, very good agreement between the axisymmetric model and the equivalent plane strain model is found. It is important to note that the inclusion of well resistance reduces the errors, notably in excess pore pressures.

Based on the above single drain analysis, Figures 2.24 – 2.26 all provide concrete evidence that the equivalent (converted) plane strain model is an excellent substitute for the axisymmetric model. In finite element modeling, the 2-D, plane strain analysis is expected to cut down the computational time considerably, in comparison with the time taken by a 3-D, axisymmetric model, especially in the case of multi drain analysis.

![Figure 2.24 Average degree of consolidation vs Time factor](image-url)

Figure 2.24 Average degree of consolidation vs Time factor
Figure 2.25 Comparison of the average surface settlement for axisymmetric and equivalent plane strain analyses with smear and well resistance (Indraratna et. al., 2000)

Figure 2.26 Comparison of the excess pore pressure for axisymmetric and equivalent plane strain analyses with smear and well resistance (Indraratna et. al., 2000)
2.12 Application of Numerical Modelling in Practice and Field Observation

Indraratna and Redana analysed (1995, 1999, 2000) the performance of several test embankments using their plane strain model. Figure 2.27 summarises the typical subsoil profile, modified Cam-clay parameters and the effective stress conditions in the site. The unit weight of the weathered crust is about 18 kN/m³ and the lowest unit weight of the soil is about 14.3 kN/m³ at a depth of 7 m.

The typical finite element meshes for embankments employing the multi-drain analysis are given in Figures 2.28 and 2.29. The foundation is discretised into linear strain quadrilateral (LSQ) elements. For the zone stabilised with Prefabricated Vertical Drains (PVD), a finer mesh was used so that each drain element includes the smear zone on either side of the PVD. The location of inclinometers and piezometers is accurately defined in the mesh, with the measurement points placed on the mesh nodes. The piezometer locations are shown in the insert of each mesh. The embankment load is applied in stages (ie. sequential construction). Figure 2.30 indicates the rate of loading and the construction history of a typical embankment.

The numerical analysis was based on the modified Cam-clay model (Roscoe and Burland, 1968) incorporated in the finite element code, CRISP92 (Britto and Gunn, 1987). The equivalent plane strain values were calculated based on Eqns. 2.42 and 2.43 (Indraratna & Redana, 1997). After a few trials to include the effect of well resistance, the minimum discharge capacity \(q_w\) was estimated to model the settlements and pore water pressure dissipation.
Figure 2.27 Sub-soil profile, Cam-clay parameters and stress condition used in numerical analysis, Second Bangkok International Airport, (after AIT, 1995)

Figure 2.28 Finite element mesh of embankment for plane strain analysis with variable drain lengths (Redana, 1999)
Figure 2.29 Finite element mesh of the embankment for plane strain analysis with constant drain length (Indraratna & Redana, 1999)

Figure 2.30 Construction loading history for embankments TS1, TS2 and TS3 at Second Bangkok International Airport (AIT, 1995)
The results of the plane strain analysis of a typical embankment together with the measured settlements are plotted in Figure 2.31. The analysis based on ‘perfect drain’ conditions (i.e. no smear, complete pore pressure dissipation) overpredicts the measured settlement, but the inclusion of smear effect improves the accuracy of the predictions. The inclusion of the effects of smear and well resistance underestimate the measured settlements. In terms of settlements, the role of well resistance could be regarded as insignificant, in comparison with the smear effect.

![Figure 2.31 Surface settlement at the centre-line for embankment TS1, Second Bangkok International Airport (Indraratna & Redana, 2000)](image)

The measured and predicted excess pore pressures along the centerline of the embankment at a depth of 2 m below the ground surface are compared in Figure 2.32. In the ‘smear only’ analysis, the pore water pressure increase is well predicted during Stage 1 and Stage 2 loading. However after Stage 3 loading, the predicted pore pressure is significantly smaller than the field measurements. As expected, the perfect drain
predictions underestimate the actual pore pressures. The inclusion of the effects of smear and well resistance gives a better prediction of the pore water pressure dissipation for all stages of loading.

The prediction of settlement along the ground surface from the centerline of a typical embankment after 400 days is shown in Figure 2.33. At the embankment centerline, the limited available data agree well with the settlement profile. Also heave could be predicted beyond the toe of the embankment, i.e. at about 42 m away from the centerline.

Measured and predicted lateral deformation for the inclinometer installed away from the centerline of the embankment is shown in Figure 2.34. The lateral displacements at 44 days and 294 days after loading are well predicted when both the effects of smear and well resistance are included. As shown in Figure 2.34b the inclusion of smear effect by itself underestimates the magnitude of lateral displacement. The ‘perfect drain’ condition yields the smallest lateral deformation. The predicted lateral deformation for ‘no drains’ is plotted for comparison. It is verified that the presence of PVD is expected to reduce the lateral movement of soft clay under embankment loading.
Figure 2.32 Variation of excess pore water pressures at 2 m depth below ground level at the centre-line for embankment TS1 (Indraratna & Redana, 2000)

Figure 2.33 Surface settlement profiles after 400 days, Muar clay, Malaysia (Indraratna & Redana, 1999)
2.13 Summary

Vertical drains have been widely used to accelerate the rate of primary consolidation. However, it is difficult to predict the settlements and pore pressures accurately due to the
complexity of estimating the correct values of soil parameters inside and outside smear zone. Therefore, it is necessary to use appropriate laboratory techniques to measure these parameters. Smear effect caused by PVD installation affects the subsequent consolidation around vertical drains.

For large construction sites, where many PVDs are installed, 2D plane strain analysis is most convenient given the computational efficiency. Recently developed axisymmetric to plane strain conversions provide good agreement with measured data and these simplified plane strain methods are now widely used in the finite element analysis successfully.

Vacuum preloading through PVD and surface membrane system effectively promotes radial consolidation while controlling lateral yield of the soil, in comparison with the conventional surcharge embankment loading, that can generate large lateral displacements in very soft clays.

However the differences between vacuum and surcharge preloading have not been investigated in depth. In the absence of comprehensive and quantitative analysis, the study of suitable methods to simulate vacuum preloading become imperative, both experimentally and numerically. The development of new models and the associated behaviour of soft clay subjected to vacuum surcharge are explained in the subsequent chapters.
3 THEORETICAL CONSIDERATIONS

3.1 General

Most finite element analyses on embankments are conducted based on the plane strain assumption, although the consolidation around vertical drains is axisymmetric. Therefore, to employ a realistic 2-D finite element analysis for vertical drains, the equivalence between plane strain and axisymmetric analysis needs to be established.

The original axisymmetric analysis for vertical drains (Barron, 1948) which has been further modified by Hansbo (1981) to include the effect of smear and well resistance was reviewed in Chapter 2. Hird et al. (1992) developed an equivalent plane strain analysis including well resistance by considering a unit cell of the vertical drain, where geometric and permeability matching techniques were adopted. In this model, several methods have been discussed with the objective of satisfying the average degree of consolidation in both plane strain and axisymmetric conditions. Firstly, the geometric matching approach was followed, where the spacing of the drains was matched while keeping the permeability the same. Secondly, the permeability matching was adopted, where the permeability coefficient was matched, while keeping the spacing of the drains the same. The third method is a combination of permeability and geometry matching, where the plane strain permeability is calculated for a convenient drain spacing. A further refinement of this method has also been reported later (Hird et al., 1995). Following the Previous developments (e.g. Cheung et al., 1991; Hird et al., 1992), Indraratna and
Redana (1997) extended the analysis based on the plane strain solution to include the effects of both smear and well resistance.

Mohamedelhassan and Shang (2002) developed an analytical model considering the vacuum pressure and surcharge based one-dimensional consolidation of clay. Following that model, an attempt was made to extend the axisymmetric theory developed by Hansbo (1981) and plane strain theory developed by Indraratna and Redana (1997) to incorporate vacuum pressure application. The effect of various factors such as the drain spacing, well resistance, smear effect on soil consolidation around vertical drains subjected to vacuum preloading are discussed.

3.2 Modelling of Axisymmetric Solution with Applied Vacuum Pressure

Figure 3.1 Schematic of soil cylinder with vertical drain (after Hansbo, 1979)
Axisymmetric analysis described by Hansbo (1981) may be extended to include the application of vacuum pressure as follows. Considering the Darcy’s law, the velocity of water flow \( v_r \) in the undisturbed zone and smear zone is given by the equations (3.1) and (3.2), respectively, as follows:

\[
\frac{\partial u}{\partial t} = \frac{k_h}{\gamma_w} \left( \frac{\partial u}{\partial r} \right) \quad (3.1)
\]

\[
\frac{\partial u'}{\partial t} = \frac{k'_h}{\gamma_w} \left( \frac{\partial u'}{\partial r} \right) \quad (3.2)
\]

where, \( k_h = \) coefficient of horizontal permeability in undisturbed zone

\( k'_h = \) coefficient of horizontal permeability in smear zone

\( \gamma_w = \) unit weight of water

\( u = \) excess pore water pressure in undisturbed zone at radius \( r \)

\( u' = \) excess pore water pressure in smear zone at radius \( r \), and

\( r = \) radial direction of flow

It is postulated that the flow of pore water through the boundary of the cylinder with radius \( r \) is equal to the change in volume of the hollow cylinder with outer radius \( R \) and inner radius \( r \), such that

\[
2\pi r v_r = \pi (R^2 - r^2) \frac{\partial \varepsilon}{\partial t} \quad (3.3)
\]

where, \( \varepsilon \) is the strain in \( z \) direction. Substituting Equation 3.1 into Equation 3.3 and subsequent rearranging gives the following equation for the pore pressure gradient in the undisturbed soil:
\[ \frac{\partial u}{\partial r} = \frac{\gamma_w}{2k_h} \left( \frac{R^2}{r} - r \right) \frac{\partial \varepsilon}{\partial t} ; \quad r_s \leq r \leq R \quad (3.4) \]

Similarly in the smeared zone, the corresponding pore pressure gradient is given by:

\[ \frac{\partial u'}{\partial r} = \frac{\gamma_w}{2k'_h} \left( \frac{R^2}{r} - r \right) \frac{\partial \varepsilon}{\partial t} ; \quad r_w \leq r \leq r_s \quad (3.5) \]

Considering the horizontal cross-sectional slice of thickness \( dz \) of a circular cylindrical drain with radius \( r_w \), the total change in flow from the entrance face to the exit face of the slice is given by Hansbo (1981) as:

\[ dQ_2 = \frac{\pi r^2 k_w}{\gamma_w} \frac{\partial^2 u}{\partial z^2} dz dt \quad \text{for} \quad r \leq r_w \quad (3.6) \]

The horizontal inflow of water into the slice from the surrounding is given by:

\[ dQ_1 = \frac{2\pi r k'_w}{\gamma_w} \frac{\partial u'}{\partial r} dz dt \quad \text{for} \quad r = r_w \quad (3.7) \]

For ensuring continuity of flow, the following equation needs to be satisfied:

\[ 2dQ_1 + dQ_2 = 0 \quad (3.8) \]

It is assumed that at the boundary of the drain \( (r=r_w) \) there is no sudden drop in pore pressure, hence \( u=u' \). Substituting Equations (3.6) and (3.7) in Equation (3.8) and subsequent rearranging with the above boundary condition yields:

\[ \left( \frac{\partial u'}{\partial r} \right)_{r=r_w} + \frac{r}{2k_c} \left( \frac{\partial^2 u'}{\partial z^2} \right)_{r=r_w} = 0 \quad (3.9) \]

After substituting Equation (3.9) into Equation (3.5) and integrating in the \( z \) direction subject to the following boundary conditions: at \( z=0 \), \( u'=u_{vac} \) (applied vacuum pressure), at \( z=2l \), \( u' = 0 \); and at \( z=l \), \( \partial u'/\partial z = 0 \), the pore pressure at \( r=r_w \), may be determined by:
\[(u')_{r=r_w} = u_{vac} + \gamma_w \frac{\partial E}{\partial t} \left( n^2 - 1 \right) \left( l_z - z^2 \right) \]  \hspace{1cm} (3.10)

Integrating Equations (3.4) and (3.5) in the \( r \) direction with the same boundary conditions as stated above, and by assuming \( u = u' \) at the interface \( r=r_w \) (see Figure 3.1) the following two expressions for \( u \) and \( u' \) can be derived:

\[ u' = u_{vac} + \gamma_w \frac{\partial E}{2k_h} \left[ R^2 \ln \frac{r}{r_w} - \frac{r^2 - r_w^2}{2} + \frac{k'_h}{k_{w}} \left( n^2 - 1 \right) \left( 2l_z - z^2 \right) \right] \text{; for } r_w \leq r \leq r_s \hspace{1cm} (3.11) \]

\[ u = u_{vac} + \gamma_w \frac{\partial E}{2k_h} \left[ R^2 \ln \frac{r}{r_s} - \frac{r^2 - r_s^2}{2} + \frac{k_h}{k_{w}} \left( R^2 \ln s - \frac{r_s^2 - r^2}{2} \right) + \frac{k_h}{k_{w}} \left( n^2 - 1 \right) \left( 2l_z - z^2 \right) \right] \text{; for } r_s \leq r \leq R \hspace{1cm} (3.12) \]

Let \( \bar{u} \) be the average excess pore water pressure between the smeared and intact zones, at depth \( z \) and for a given time, \( t \):

\[ \bar{u} \pi \left( R^2 - r^2 \right) = \int_{r_w}^{r_s} 2\pi u' rdr + \int_{r_s}^{R} 2\pi u dr \hspace{1cm} (3.13) \]

Solution of Equation (3.13) by substitution from Equations (3.11) and (3.12) and omitting the terms of minor significance gives the following expression for \( \bar{u} \) at any given depth, \( z \) and time, \( t \):

\[ \bar{u} = u_{vac} + \gamma_w R^2 \mu \frac{\partial E}{2k_h} \hspace{1cm} (3.14) \]

where, \( \mu = \ln n - 0.75 + \left( \frac{k_h}{k_{w}} - 1 \right) \ln s + \pi \left( 2l - z \right) \frac{k_h}{q_w} \)

Equation (3.14) may now be combined with the time-dependent compressibility governed by the following well-known consolidation expression:
\[
\frac{\partial \varepsilon}{\partial t} = -m_v \frac{\partial u}{\partial t} = \frac{k_{hp}}{c_{hp} \gamma_w} \frac{\partial u}{\partial t}
\]  
(3.15)

where, \( m_v \) is the coefficient of volume compressibility and \( c_{hp} \) is the horizontal coefficient of consolidation.

Substituting Equation (3.15) into Equation (3.14), and then integrating subject to the boundary condition that at \( t=0, \ u = u_{sur} \) gives the following expression:

\[
t = -\frac{R^2 \gamma_w m_v}{2k_h} \mu \ln \left( \frac{u - u_{vac}}{u_{sur} - u_{vac}} \right)
\]  
(3.16)

where, \( u_{sur} \) is the applied surcharge pressure. The equation (3.16) gives the following expression for \( \frac{u}{u_{sur}} \) as,

\[
\frac{u}{u_{sur}} = \frac{u_{vac}}{u_{sur}} + \left( 1 - \frac{u_{vac}}{u_{sur}} \right) \exp \left( -\frac{8T_h}{\mu} \right)
\]  
(3.17)

This yields the average degree of consolidation

\[
\bar{U}_h = \frac{u_i - \bar{u}}{u_f - \bar{u}} = \frac{u_{sur} - \bar{u}}{u_{sur} - u_{vac}} = 1 - \exp \left( -\frac{8T_h}{\mu} \right)
\]  
(3.18)

where, \( u_i \) = initial excess pore water pressure, \( u_f \) = final excess pore water pressure and

(a) \( \mu = \ln \left( \frac{n}{s} \right) + \left( \frac{k_h}{k_{hr}} \right) \ln(s) - 0.75 + \pi \left( 2l - z \right) \frac{k_h}{q_w} \) (both smear and well resistance)

(b) \( \mu = \ln \left( \frac{n}{s} \right) + \left( \frac{k_h}{k_{hr}} \right) \ln(s) - 0.75 \) (smear effect only)

(c) \( \mu = \ln(n) - 0.75 \) (perfect drain condition, function of drain spacing)
In the above expressions, \( n = R/r_w \) and \( s = r_s/r_w \) (see Figure 3.1), \( q_w \) = drain discharge capacity and \( l \) is the initial height of the unit cell.

Equation 3.18 is same as the Equation 2.29, which is rewritten below, developed by Hansbo (1981) without considering vacuum pressure and assuming a final excess pore pressure of zero \((u_f \rightarrow 0)\).

\[
\bar{U}_h = 1 - \frac{\bar{u}}{\bar{u}_0} = 1 - \exp\left(\frac{-8T_h}{\mu}\right) \hspace{1cm} (3.19)
\]

In the absence of surcharge pressure (i.e. vacuum pressure only, \( u_{sur}=0 \)), the average degree of consolidation can be given by,

\[
\bar{U}_h = \frac{\bar{u}_0 - \bar{u}}{\bar{u}_0 - u_f} = \frac{u_{sur} - \bar{u}}{u_{sur} - u_{vac}} = \frac{\bar{u}}{u_{vac}} \hspace{1cm} (3.20)
\]

Combining Equation (3.20) with (3.17) when \( u_{sur}=0 \), the average degree of consolidation in the absence of surcharge pressure can be given by,

\[
\bar{U}_h = 1 - \exp\left(\frac{-8T_h}{\mu}\right) \hspace{1cm} (3.21)
\]

Figure 3.2 shows the distribution of average excess pore water pressure against the time factor \( T_{hp} \) ignoring the smear and well resistance, based on Equation 3.17. Two different drain spacings (\( n = 10 \) and 20) were considered. The magnitude of both applied surcharge and vacuum pressure was considered to be 50 kPa. Figure (3.2) clearly shows that with the increase of drain spacing, the rate of pore pressure dissipation is retarded. Application of surcharge pressure alone initially generates excess pore water pressure, which is equal to the applied pressure and dissipates to zero with time, while the
application of vacuum pressure alone generates negative pore water pressure that equals the applied vacuum pressure with time. For most part of these curves, the rate of dissipation is similar. Application of both surcharge and vacuum pressure starts with excess pore water pressure equal to surcharge pressure and finally dissipates to a negative pore water pressure that is the same as the applied vacuum pressure with time.

Figure 3.2 Distribution of average excess pore water pressure

Figure 3.3 presents the variation of excess pore pressure distribution with different permeability ratios (undisturbed permeability/ smear zone permeability). The distributions for perfect drain with vacuum pressure and without vacuum pressure are also plotted in the same figure for comparison. As expected, the rate of pore pressure
Figure 3.3 Average excess pore water pressure distribution of axisymmetric unit cell with different permeability ratios (100 kPa vacuum and 25 kPa surcharge).

Figure 3.4 Average excess pore pressure distribution at 5m depth with different vacuum pressures (surcharge load=50 kPa, $k_h=1 \times 10^{-8}$ m/s)
dissipation becomes less with the higher permeability ratios. When permeability ratio is 10, the dissipation rate becomes less than the perfect drain without vacuum.

Variation of average excess pore pressure distribution at 5m depth with different applied vacuum pressures is illustrated in Figure 3.4. It clearly indicates the increased rate of dissipation with high vacuum pressures. With the presence of smear and well resistance, the rate of dissipation retards significantly (less than the perfect drain without vacuum) specially at initial time increments.

Figure 3.5 presents the distribution of excess pore pressure at 5m depth for different drain lengths and for two specific discharge capacities (50 and 500 m$^3$/year) based on Equation 3.17 (for $u_{sur}=25$ kPa and $u_{vac} = -100$kPa). Distribution of excess pore pressure for perfect drain condition with and without vacuum is also plotted. It shows that for low discharge capacity (Figure 3.5a), the rate of dissipation decreases with the increase in drain length, while this effect is not significant for a much higher discharge capacity (Figure 3.5b).

Figure 3.6 illustrates the effect of drain discharge capacity on the distribution of excess pore pressure at 5m depth for two different drain lengths (10m and 20m) based on Equation 3.17 (for $u_{sur}=25$ kPa and $u_{vac} = -100$kPa). It indicates that for both drain lengths, the effect of well resistance is negligible when the drain discharge capacity is 500 m$^3$/year or greater. Similar type of observations has been discussed by
Figure 3. 5 Average excess pore pressure distribution for different drain lengths when, (a) $q_w=50\text{m}^3/\text{year}$ (b) $q_w=500 \text{m}^3/\text{year}$
Figure 3.6 Average excess pore pressure distribution with different drain discharge capacity when (a) $l=10\text{m}$ (b) $l=20\text{m}$
Jamiolkowski et al. (1983). It was mentioned that the ratio of $q_w/k_h$ less than 500 m$^2$ might play a very important role in increasing the time required to achieve a specific degree of consolidation.

3.3 Modelling of Plane Strain Solution Incorporating Vacuum Pressure Application

3.3.1 No smear or well resistance

Considering Darcy's law, the velocity of water flow ($v_x$) in the undisturbed zone is given by the following equation:

$$v_x = \left( \frac{k_{hp}}{\gamma_w} \right) \left( \frac{\partial u}{\partial x} \right)$$  \hspace{1cm} (3. 22)

where, $k_{hp} =$ coefficient of horizontal permeability in plane strain condition;

$\gamma_w =$ unit weight of water,

$u =$ pore water pressure, and

$x =$ prescribed direction of flow.

For the plane strain model, consider a horizontal slice of thickness $dz$, of the unit cell (Figure 3.7). It is postulated that the flow in the slice at a distance $x$ from the centerline of the drain is equal to the change in volume within a block of soil of width $(B-x)$, such that:
\[ v_x = \left( \frac{\partial \varepsilon}{\partial t} \right) (B - x) \]  \hspace{1cm} (3.23)

Substituting Equation (3.23) into Equation (3.22) and rearranging gives the following equation for the pore pressure gradient in the soil domain:

\[ \left( \frac{\partial u}{\partial x} \right) = \left( \frac{\gamma_w}{k_{wp}} \right) \left( \frac{\partial \varepsilon}{\partial t} \right) (B - x), \quad B \geq x \geq b_w \]  \hspace{1cm} (3.24)

Integrating Equation (3.24) in the \( x \) direction with the boundary condition \( u = u_{\text{vacp}} \) (applied vacuum pressure in plane strain) at the interface \( x = b_w \) (see Figure 3.7) leads to the following two expression for \( u \):

\[ u = u_{\text{vacp}} + \frac{\gamma_w}{2k_{wp}} \frac{\partial \varepsilon}{\partial t} \left[ x(2B - x) - b_w (2B - b_w) \right] \]  \hspace{1cm} (3.25)

Figure 3. 7 Plane strain unit cell
Let $\bar{u}$ be the average excess pore water pressure throughout the soil cell:

$$\bar{u} = \frac{\int_{b_w}^{B} u dx}{(B - b_w)} \quad (3.26)$$

Integrating in combination with Equation (3.25) yields:

$$\bar{u} = u_{vacp} + \frac{B^2 \gamma_w}{2k_{hp}} \frac{\partial \varepsilon}{\partial t} \left[ \frac{2}{3} \left( 1 - \frac{1}{n} \right)^2 \right] \quad (3.27)$$

where, $n = B / b_w$

Equation (3.27) may now be combined with the time-dependent compressibility governed by the following well-known consolidation expression:

$$\frac{\partial \varepsilon}{\partial t} = -m_v \frac{\partial \bar{u}}{\partial t} = -c_{hp} \gamma_w \frac{\partial \bar{u}}{\partial t} \quad (3.28)$$

where, $m_v$ is the coefficient of volume compressibility and $c_{hp}$ is the horizontal coefficient of consolidation.

Substituting Equation (3.28) into Equation (3.27), and then integrating subject to the boundary condition that at $t=0$, $\bar{u} = \bar{u}_{sur}$ gives the following expression:

$$\bar{u} - u_{vacp} = \mu_p \ln \left( \frac{\bar{u} - u_{vacp}}{\bar{u}_{sur} - u_{vacp}} \right) \quad (3.29)$$

where, $\mu_p = \left[ \frac{2}{3} \left( 1 - \frac{1}{n} \right)^2 \right]$ and $\bar{u}_{sur}$ is equal to the applied surcharge pressure, Equation (3.29) gives the following expression for $\bar{u} / \bar{u}_{sur}$ as,
\[
\frac{\bar{u}}{u_{\text{sur}}} = \frac{u_{\text{vacp}}}{u_{\text{sur}}} + \left(1 - \frac{u_{\text{vacp}}}{u_{\text{sur}}} \right) \exp\left(-\frac{8T_{hp}}{\mu_p}\right)
\]  
(3.30)

Given the time factor, \(T_{hp} = \frac{c_{hp} t}{4B^2}\) and the coefficient of consolidation as \(c_{hp} = \frac{k_{hp}}{m_v \gamma_w}\), the average degree of consolidation (\(\bar{U}_{hp}\)) and the time factor (\(T_{hp}\)) for plane strain conditions, which can be represented by:

\[
\bar{U}_{hp} = \frac{\bar{u}_i - \bar{u}}{\bar{u}_f - \bar{u}_i} = \frac{\bar{u}_{\text{sur}} - \bar{u}}{u_{\text{sur}} - u_{\text{vacp}}} = 1 - \exp\left(-\frac{8T_{hp}}{\mu_p}\right)
\]  
(3.31)

3.3.2 with smear and well resistance

![Plane strain unit cell with smear zone](image)

Figure 3.8 Plane strain unit cell with smear zone
As given in Equation 3.24, pore pressure gradient in the undisturbed zone and smear zone can be given by Equations 3.32 and 3.33, respectively.

\[
\left( \frac{\partial u}{\partial x} \right) = \left( \frac{\gamma_w}{k_{hp}} \right) \left( \frac{\partial u}{\partial t} \right) (B - x); \quad B \geq x \geq b_s
\]

(3.32)

\[
\left( \frac{\partial u'}{\partial x} \right) = \left( \frac{\gamma_w}{k_{hp}'} \right) \left( \frac{\partial u'}{\partial t} \right) (B - x); \quad b_s \geq x \geq b_w
\]

(3.33)

where, \( k_{hp} \) and \( u' \) are the coefficient of permeability and pore water pressure in the smeared zone, respectively, and \( k_{hp} \) and \( u \) are the corresponding values in undisturbed zone.

For vertical flow in the z direction of the drain, the change of flow from the entrance to the exit of the slice \( dq_z \) is now given by:

\[
dq_z = \left( \frac{q_z}{\gamma_w} \right) \left( \frac{\partial^2 u}{\partial z^2} \right)_{x=b_s} \, dz \, dt
\]

(3.34)

The horizontal inflow to the drain slice from each side \( dq_x \) is determined from:

\[
dq_x = \left( \frac{k_{hp}'}{\gamma_w} \right) \left( \frac{\partial u}{\partial z} \right)_{x=b_w} \, dz \, dt
\]

(3.35)

For ensuring continuity of flow, the following flow continuity equation needs to be satisfied (see Figure 3.8):

\[
dq_z + 2dq_x = 0
\]

(3.36)
It is assumed that at the boundary of the drain \((x=b_w)\), there is no sudden drop in pore pressure, hence, \(u=u'\). Substituting Equations (3.34) and (3.35) in Equation (3.36) and subsequent rearranging with the above boundary condition yields:

\[
\left( \frac{\partial u'}{\partial x} \right)_{x=b_w} + \frac{q_z}{2k_{hp}} \left( \frac{\partial^2 u'}{\partial z^2} \right)_{x=b_w} = 0
\]  

(3.37)

By substituting Equation (3.37) into Equation (3.33) and then integrating in the \(z\) direction subject to the following boundary conditions: at \(z=0, u'=u_{vacp}\); at \(z=2l, u' = 0\); and at \(z=l, \frac{\partial u'}{\partial z} = 0\), the pore pressure in the smeared zone \((u')_{x=b_w}\) may be determined by:

\[
(u')_{x=b_w} = u_{vacp} + \frac{2(B-b_w)\gamma_w}{q_z} \frac{\partial \varepsilon}{\partial t} \left( l z - \frac{z^2}{2} \right)
\]  

(3.38)

Integrating Equations (3.32) and (3.33) in the \(x\) direction with the same boundary conditions as stated above, and by assuming \(u = u'\) at the interface \(x=b_s\) (see Figure 3.8) leads to the following two expressions for \(u\) and \(u'\):

For \(b_s \leq x \leq B\),

\[
u = u_{vacp} + \frac{\gamma_w}{2} \frac{\partial \varepsilon}{\partial t} \frac{1}{k_{hp}} \left[ x(2B-x) + \frac{2(B-b_s)k_{hp}}{q_z} \left( 2lz - z^2 \right) - 2B + b_s \right] + \frac{k_{hp}}{k_{hp}} \left( b_s - b_w \right) \left( 2B - b_s - b_w \right)
\]  

(3.39)

For \(b_w \leq x \leq b_s\),

\[
u' = u_{vacp} + \frac{\gamma_w}{2} \frac{\partial \varepsilon}{\partial t} \frac{1}{k_{hp}} \left[ x(2B-x) + \frac{2(B-b_w)k_{hp}}{q_z} \left( 2lz - z^2 \right) - b_w \left( 2B - b_w \right) \right]
\]  

(3.40)
Let \( \bar{u} \) be the average excess pore water pressure between the smeared and intact zones, at depth \( z \) and for a given time, \( t \):

\[
\bar{u} = - \frac{\int_{b_w}^{b} u \, dx + \int_{b_w}^{b} u' \, dx}{(B - b_w)}
\]  
(3.41)

Solution of Equation (3.41) by substitution from Equations (3.39) and (3.40) gives the following expression for \( \bar{u} \) at any given depth, \( z \) and time, \( t \):

\[
\bar{u} = u_{vacep} + \frac{B^2 \gamma_w}{2k_{hp}} \frac{\partial \varepsilon}{\partial t} \left[ \alpha + \frac{k_{hp}}{k_{hp}'} (\beta) + (\theta) \left( 2lz - z^2 \right) \right]
\]  
(3.42)

where,

\[
\alpha = \frac{2}{3} \left( 1 - \frac{1}{n} \right)^2 - \frac{2(s-1)}{(n-1)n} \left[ n(n-s-1) + \frac{1}{3} (s^2 + s + 1) \right]
\]  
(3.43a)

\[
\beta = \frac{2(s-1)}{(n-1)n} \left[ n(n-s-1) + \frac{1}{3} (s^2 + s + 1) \right]
\]  
(3.43b)

\[
\theta = \frac{2k_{hp}}{Bq} \left( 1 - \frac{1}{n} \right)
\]  
(3.43c)

Equation (3.42) may now be combined with the time-dependent compressibility governed by the following well-known consolidation expression:

\[
\frac{\partial \varepsilon}{\partial t} = -m_v \frac{\partial u}{\partial t} = \frac{k_{hp}}{c_{hp} \gamma_w} \frac{\partial u}{\partial t}
\]  
(3.43)

where, \( m_v \) is the coefficient of volume compressibility and \( c_{hp} \) is the horizontal coefficient of consolidation.

Substituting Equation (3.43) into Eq. (3.42), and then integrating subject to the boundary condition that at \( t=0, \bar{u} = \bar{u}_{sur} \) gives the following expression:
\[
\tau = -\frac{B^2 \gamma_w m_v}{2k_{hp}} \mu_p \ln \left( \frac{\bar{u} - u_{vap}}{\bar{u}_{sur} - u_{vap}} \right)
\]  
(3.44)

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

Given the time factor as \[T_{hp} = \frac{c_{hp} t}{4B^2}\] and the coefficient of consolidation as \[c_{hp} = \frac{k_{hp}}{m_v \gamma_w}\], the average degree of consolidation (\[\bar{U}_{hp}\]) and the time factor (\[T_{hp}\]) for plane strain conditions, can be represented by:

\[
\bar{U}_{hp} = \frac{\bar{u}_i - \bar{u}}{\bar{u}_f - \bar{u}_i} = \frac{\bar{u}_{sur} - \bar{u}}{\bar{u}_{sur} - u_{vap}} = 1 - \exp \left( -\frac{8T_{hp}}{\mu_p} \right)
\]  
(3.45)

\[\mu_p\] can be rewritten as:

\[\mu_p = [F_1 + F_2 + F_3]\]

where \[F_1\], \[F_2\] and \[F_3\] represent the effect of drain spacing, smear zone and well resistance, respectively, and can be written as:

\[F_1 = \frac{2}{3} \left( 1 - \frac{1}{n} \right)^2\]

\[F_2 = \frac{2(s-1)}{(n-1)n^2} \left[ n(n-s-1) + \frac{1}{3} \left( s^2 + s + 1 \right) \left( \frac{k_{hp}}{k_{hp}' - 1} \right) \right]\]

\[F_3 = \frac{2k_{hp}}{Bq_z} \left( 1 - \frac{1}{n} \right)\]
3.4 Comparison of Axisymmetric vs Plane Strain Conditions

For a perfect drain (no smear or well resistance), the average excess pore pressure variation in axisymmetric (Equation 3.17) and plane strain (Equation 3.30) unit cell can be rewritten as follows;

For axisymmetric,

\[
\bar{u} = u_{vac} + \left( u_{sur} - u_{vac} \right) \exp \left( - \frac{8T_h}{\mu} \right)
\]  (3.46)

For plane strain,

\[
\bar{u} = u_{vacp} + \left( u_{sur} - u_{vac} \right) \exp \left( - \frac{8T_{hp}}{\mu_p} \right)
\]  (3.47)

where, \( \mu = \ln (n) - 0.75 \) and \( \mu_p = \frac{2}{3} \left( 1 - \frac{1}{n} \right)^2 \)

Assuming geometry and permeability of axisymmetric and plane strain cell are the same, the average excess pore pressure distribution is plotted in Figure 3.9. It can be seen that the rate of pore pressure dissipation is higher in the plane strain cell as discussed by several researchers in the past (for the case of surcharge load only). This is attributed to different boundary conditions in the plane strain and axisymmetric cell. Therefore, the use of a proper matching procedure is important when a plane strain analysis is conducted, instead of a true axisymmetric situation.
3.5 Matching Principle and Theoretical Considerations

Analytical modelling of radial flow to a central drain involves a cylinder of soil around a single vertical drain with simplified boundary conditions (ie. axisymmetric). In a two-dimensional finite element analysis, in which the plane strain model is utilised, it is pertinent to convert the system of vertical drains into an equivalent drain wall following the approach discussed by Hird et al. (1992). In this section, the model developed by Indraratna and Redana (1997) is modified to include the application of vacuum pressure.
Figure 3.10a shows a unit cell with an external radius, $R$ and an initial length, $l$. The radius of the vertical drain and the smear zone are $r_w$ and $r_s$, respectively. In comparison with model developed by Hird et al. (1992), the Figure 3.10 depicts a smear zone (hatched), which is modelled explicitly.

![Diagram of unit cell with external radius $R$ and initial length $l$. The radius of the vertical drain is $r_w$ and the smear zone is $r_s$.](image)

(a) Axisymmetric Radial Flow  
(b) Plane Strain

Figure 3.10 Conversion of an axisymmetric unit cell into plane strain

In the method proposed here, the vertical drain system is converted into equivalent parallel drain walls by adjusting both the spacing of the drain wall and the coefficient of permeability of the soil. In Figure 3.10, the appropriate conversion was conducted by assuming the plane strain unit cell to have a width of $2B$. The width of the drain is determined by considering the total capacity of the drain in both systems to be the same.
For example, in a system of vertical drains arranged at a spacing of $S$ in a square pattern, the width of the drain and the smear zone can be expressed by:

$$b_w = \frac{\pi r_w^2}{2S} \text{ and } b_s = \frac{\pi r_s^2}{2S}, \text{ respectively.} \quad (3.48a)$$

For drains arranged in a triangular pattern, the equivalent widths are given by:

$$b_w = \frac{1.143 \pi r_w^2}{2S} \text{ and } b_s = \frac{1.143 \pi r_s^2}{2S} \quad (3.48b)$$

where, $S$ is the field spacing (center to center) between any two adjacent drains.

Another alternative is to keep the geometry (e.g. width of drain and smear zone) the same in both axisymmetric and plane strain conditions, which gives:

$$b_w = r_w \text{ and } b_s = r_s \quad (3.49)$$

The above drain and smear zone dimensions are defined in Figure 3.10.

At each time step and at a given stress level, the average degree of consolidation for both axisymmetric ($\overline{U}_h$) and equivalent plane strain ($\overline{U}_{hp}$) conditions are made equal, hence:

$$\overline{U}_h = \overline{U}_{hp} \quad (3.50)$$

Combination of Equations (3.45) and (3.50) with the original Hansbo (1981) theory (Equation 2.29) defines the time factor ratio by the following equation:

$$\frac{T_{hp}}{T_h} = \frac{k_{hp}}{k_h} \cdot \frac{R^2}{B^2} = \frac{\mu_p}{\mu} \quad (3.51)$$

For square and triangular patterns of vertical drains, the influence diameter for each drain ($D=2R$) is given by: $D = 1.13 \, S$, and $D = 1.05 \, S$, respectively (Barron, 1948). In the
equivalent plane strain model, the spacing $S$ is equal to the total width of the unit cell, $2B$. Therefore, for simplicity, the writer has assumed the magnitudes of $R$ and $B$ to be the same, which results in the following expression for the equivalent plane strain permeability.

$$
k_{hp} = k_h \left[ \alpha + \left( \beta \frac{k_{hp}}{k_{hp}'} \right) + \theta \left( 2lz - z^2 \right) \right]
$$  \hspace{1cm} (3.52)

Ignoring the well resistance in Equation (3.52), where all terms containing $l$ and $z$ are omitted, the influence of the smear effect can be isolated and represented by the ratio of the smear zone permeability to the undisturbed permeability as follows:

$$
\frac{k_{hp}'}{k_{hp}} = \frac{\beta}{k_h \left[ ln(n) + \left( \frac{k_{hp}}{k_{hp}'} \right) ln(s) - 0.75 + \pi \left( 2lz - z^2 \right) \frac{k_h}{q_w} \right] - \alpha}
$$  \hspace{1cm} (3.53)

The influence of well resistance is not pronounced when significant flows ($q_w$ and $q_z$) take place within the drains, whereby the parameter $\theta$ becomes small in comparison with the smear effect terms. Ignoring well-resistance and smear in Equation (3.52), where all terms containing $s$, $l$ and $z$ are omitted, the simplified ratio of plane strain to axisymmetric permeability is readily obtained as:

$$
\frac{k_{hp}}{k_h} = \frac{0.67(1 - \frac{1}{n})^2}{\ln(n) - 0.75}
$$  \hspace{1cm} (3.54)
The well resistance is derived independently and yields an equivalent plane strain discharge capacity of drains as also proposed earlier by Hird et al. (1992):

\[ q_z = \frac{2}{\pi B} q_w \]  

(3.55)

Vacuum pressure is now matched separately as,

\[ u_{vacp} = u_{vac} \]  

(3.56)

In this study, Eqs. (3.53), (3.54) and (3.56) are incorporated in the numerical analysis (employing ABAQUS) to compare the laboratory data and to study selected case histories, as discussed in the following Chapters.

### 3.6 Summary

In this Chapter, axisymmetric theory for vertical drain developed by Hansbo (1981) and plane strain theory developed by Indraratna and Redana (1997) were modified to include the vacuum pressure application. It was assumed that the applied vacuum pressure is distributed uniformly along the drain depth and along the top surface. Analytical results showed that the combined vacuum and surcharge preloading is a superposition of the application of vacuum and surcharge preloading considered separately. The expression for average excess pore pressure dissipation concludes that the form of the equation is the same whether preloading is applied as a vacuum pressure or as a surcharge pressure.

The effect of different parameters on pore pressure dissipation around a single drain was also studied. It was found that the well-resistance of a drain with a relatively high
discharge capacity is not significant. The equivalent plane strain model is useful, especially where the axisymmetric field situation (radial drainage around many vertical drains) becomes too complex for numerical analysis. This conversion is necessary in order to reduce the large computational time generally required for a 3-D finite element analysis with many drains, each having their own axisymmetric zone. Since Hansbo’s theory of vertical drains has gained wide acceptance among engineers, it was appropriate that a plane strain model could be derived following the modification of his axisymmetric theory. Subsequent analysis by Hird et al. (1992) and Indraratna and Redana (1997) also introduced different models of equivalent plane strain for vertical drain extending the original work of Hansbo (1981), incorporating smear and well resistance. In this chapter, these models were further modified to include the effect of vacuum pressure application.
4 LABORATORY TESTING OF PVD INSTALLED CLAY

4.1 General

Although the final settlements associated with vacuum drainage and conventional surcharge preloading are the same, the loading mechanisms of the two techniques are distinctly different. In surcharge preloading, increase in total stress also leads to a higher effective stress, whereas in vacuum based consolidation, an increased effective stress is achieved due to reduced pore water pressure, while the total stress remains constant. The efficiency of vacuum preloading can be further increased by applying it in conjunction with a small surcharge load.

In the writer's point of view, large-scale laboratory consolidation testing is the best way of evaluating the effectiveness of prefabricated vertical drains (PVD), directly or indirectly incorporating all the soil and drain parameters. In this research, a large-scale consolidometer (Indraratna and Redana, 1995) was utilized to examine the effect of vacuum preloading in conjunction with surcharge loading. Several tests were carried out to study the effect of vacuum loading as well as the effect of vacuum application, removal and reapplication. The results show that a significant increase in settlement rate occurs with the application of vacuum pressure. Also, the settlement associated with combined vacuum and surcharge load indicates the various influences of vacuum removal and re-application during the loading history.
4.2 Large-scale Tests on Vertical drains

In the past, Bergado et al. (1991) conducted laboratory tests to study the behaviour of vertical drain installed soft clay using a specially designed large-scale consolidation test apparatus with a centrally located prefabricated drain. The cell (455 mm inner diameter, 920 mm high, 10 mm thick) was a transparent PVC cylinder with a steel base plate. However, in this setup, there were no measuring facilities for excess pore water pressures. Remoulded soft Bangkok clay was placed inside the cylinder in layers. ‘Ali drain’ (type of PVD) with reduced dimensions of 4 x 60 mm was installed using the 6 x 60 mm mandrel. The settlement behaviour was monitored under a surcharge pressure of 47.8 kPa. Permeability coefficients were calculated from conventional oedometer tests carried out for horizontal and vertical specimens taken at several locations.

A much better development was described by Indraratna and Redana (1995,1998) where they investigated the effect of smear due to installation sand compaction piles (SCP) and prefabricated vertical drains (PVD) using a large-scale consolidometer with the facility for measuring excess pore water pressure at various locations in the cell. It was found that even though a larger width of the drain may cause a greater smear zone, for PVD’s, the measurements and predictions indicate slightly increasing settlements due to the increased surface area, facilitating efficient pore water pressure dissipation. In contrast, for SCP, increasing the pile diameter does not necessarily improve pore pressure dissipation. In fact, a greater pile diameter increases the overall stiffness, thereby decreasing the surface settlement.
Xiao (2000) conducted a series of large-scale tests to study the behaviour around vertical drains installed in soft clay using remoulded kaolin clay. The consolidometer cell made of stainless steel had an internal diameter of 1 m and a wall thickness of 20 mm. The cell could be split horizontally into upper and lower cylinders that were 0.6 m and 0.4 m high, respectively. The cell was fully instrumented with several pore pressure transducers and soil pressure cells and a linear variable differential transformer (LVDT) to measure displacements. Conventional oedometer tests were carried out for horizontally and vertically selected samples from the cell. Settlement and pore pressures were also monitored.

4.3 Tests on Vacuum Preloading

Mohamedelhassan and Shang (2002) designed, manufactured and assembled a small-scale test apparatus, which had the capacity of applying both vacuum and surcharge pressures to a soil specimen without a drain. The cell could accommodate a soil sample of 70 mm diameter and 25 mm in height. It allows for the measurement of the excess pore water pressure, settlement and volume change. The results show that the vacuum pressure generates nearly identical effects compared to a surcharge pressure of the same magnitude under one-dimensional conditions.
4.4 Apparatus and Test Procedure Employed by the Writer

4.4.1 Apparatus

A digital image of the large-scale radial drainage consolidometer is shown in Figure 4.1, which consists of two half sections made of stainless steel. Each half section has a flange running the length of the cylinder allowing the two halves to be bolted together. The internal diameter and the overall height of the assembled cell are 450 mm and 1000mm, respectively and the cell is erected on a steel base. The schematic illustration of the cell is shown in Figure 4.2.

Figure 4.1 Large-scale radial drainage consolidometer
In order to reduce the friction effect along the cell boundary, a 1.5 mm thick Teflon sheet was inserted around the internal cell boundary and at the bottom of the cell. The friction of the Teflon sheet was very small, less than 2 degrees. The surcharge loading system with a maximum capacity of 1200 kN was applied by an air jack compressor system via a rigid piston of 50mm thick, while a vacuum loading system with a maximum capacity of 100 kPa was applied through the central hole of the rigid piston. The settlement was measured by a LVDT placed at the top of this piston (Figure 4.1). An array of strain gauge type pore pressure transducers complete with wiring to supply recommended 10 V DC supply, was installed to measure the excess pore water pressures at various points. Design of these transducers incorporated a ceramic pressure element in a stainless steel enclosure and bleed value to eliminate air traps (Figure 4.3). The range of transducers used was up to 500kPa maximum pore water pressure. The transducers were calibrated using a Budenburg dead weight testing machine. The piezometer tips were saturated and kept in position using thin stainless steel tubes. The LVDT and pore pressure transducers were connected to a PC based data logger.

A simple computer programme was written (Appendix A) using the calibration data to convert the transducer output in voltage to an appropriate pore pressure and settlement units (kPa and mm). Data logger can be set to necessary time periods to take readings. The cell is also equipped with a specially designed steel hoist from which a synthetic vertical drain can be inserted along the central axis of the cell.
Figure 4.2  Schematic of large-scale consolidation apparatus

Figure 4.3 Pore pressure transducer used in consolidometer testing
4.4.2 Test procedure

4.4.2.1 Test sample

The size of the each soil sample required for the large-scale consolidometer is approximately 0.15m$^3$. As it is not feasible to obtain undisturbed sample of this size, reconstituted alluvial clay from Moruya, NSW was used to make large samples. Atterberg limit and specific gravity tests were conducted to determine the properties of the clay and selected geotechnical properties of the sample as shown in Table 4.1. The clay size particles (<2μm) and particles smaller than silt size (<6μm) accounted for about 40%-50% and 90%, respectively. The clay with geotechnical properties given in Table 4.1 could be categorized as CH (high plasticity clay), on the basis of the Casagrande Plasticity Chart.

Table 4.1 Soil properties of the reconstituted Moruya clay sample

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay Content (% )</td>
<td>40-50</td>
</tr>
<tr>
<td>Silt Content (% )</td>
<td>45-60</td>
</tr>
<tr>
<td>Water content, w (%)</td>
<td>40</td>
</tr>
<tr>
<td>Liquid Limit, w_L (%)</td>
<td>70</td>
</tr>
<tr>
<td>Plastic Limit, w_P (%)</td>
<td>30</td>
</tr>
<tr>
<td>Plasticity Index, PI (%)</td>
<td>40</td>
</tr>
<tr>
<td>Unit Weight, t/m$^3$</td>
<td>1.81</td>
</tr>
<tr>
<td>Specific Gravity, G_s</td>
<td>2.63</td>
</tr>
</tbody>
</table>
4.4.2.2 Test procedure

Prior to placing the clay in the large cylinder, it was thoroughly mixed with water and kept for several days to ensure saturation. Then the saturated clay was placed and compacted in layers. In order to reduce friction between the side wall of the cylinder and the soil, a teflon sheet was laid around the inner periphery of the cell and the bottom of the cell. After specimen preparation, filter material was put on the surface of the compacted clay so that the water could be drained out easily.

After placing the soft saturated clay (unit weight 18.1 kN/m$^3$) in the chamber and then the piston, an initial consolidation pressure of 20 kPa was applied. At the end of the pre-consolidation phase, a vertical band drain of size 100mm x 3mm was installed using a specially designed rectangular mandrel and hoist. The PVD was inserted through the mandrel slot, which was only slightly larger than its rectangular cross-section. The end of the drain was attached to a shoe (anchor) to ensure that the drain remained in the proper position when the mandrel was withdrawn. Schematic illustration of the vertical drain and associated smear zone in the large-scale consolidometer are illustrated in Figure 4.4. Basically, three types of tests were conducted namely:

*Test 1*: The subsequent surcharge load was applied in two stages, 50kPa with duration of 17 days and 100kPa with a duration of 14 days. No vacuum pressure was applied. The loading increments represent the earth fill (unit weight of 18.1kN/m$^3$) heights of about 2.5m and 6m, respectively.

*Test 2*: The vacuum pressure was set to 100 kPa and it was applied to the PVD and the soil surface via the central hole of the rigid piston. Surcharge pressure was applied in
two stages 50 kPa and 100 kPa as in Test 1, but with a duration of 14 days between each stage.

Test 3: Conditions are similar to Test 2. During a total duration of 28 days, the vacuum pressure was released in two stages for short periods to investigate the effects of vacuum unloading and reloading.

To determine the permeability coefficients of smear zone and undisturbed zone, the procedure discussed by Indraratna and Redana (1998) was followed, as discussed earlier in Section 2.4.1

4.5 Results and Discussion

4.5.1 With and without vacuum pressure

Figure 4.5 shows the comparison of surface settlement measured by LVDT in tests 1 and 2, with and without the application of vacuum pressure. In the first stage of loading, sample with 50 kPa surcharge load combined with 100 kPa vacuum pressure shows a settlement of 28mm in 14 days, whereas the sample with only 50 kPa surcharge load produces 18mm settlement within the same period of time. Similar type of settlement distribution is shown by the second stage of loading. The comparison clearly indicates that the accelerated consolidation due to combine surcharge and vacuum preloading is more effective in comparison with the conventional multi-stage surcharge loading at the same time.
The excess pore water pressures measured at transducer T3 located 480 mm from bottom, and transducer T6 that is located 240 mm from the bottom are shown in Figure 4.6. The exact locations of the 2 transducers were shown earlier in Figure 4.4. Data clearly show that the application of vacuum pressure accelerates the pore pressure dissipation, which is in agreement with the settlement measurements. Transducer T3 readings show that the pore pressure is slightly greater than the expected value of 30 kPa (load increment) during the initial time period, whereas transducer T6 readings do not show any unexpected trend. This is because transducer T3 shows some Mandrel-Cryer effect due to its position away from the drainage face, while the Transducer T6 was closer to the PVD. As none of those transducers were located close to the drain, the vacuum pressure distribution along the drain depth could not be investigated in greater detail.
Figure 4.4 Schematic section of the large-scale, radial drainage consolidometer showing the central drain, associated smear zone, and typical locations of pore pressure transducers (after Indraratna and Redana, 1998)

Figure 4.5 Surface settlement associated with surcharge load and combined surcharge and vacuum loading
Figure 4.6 Measured excess pore water pressure at (a) transducer T3 and (b) transducer T6
4.5.2 Effect of removal and reapplication of vacuum pressure

The suction measured at the location of transducer T6 (Figure 4.7) and the surface settlement measured by LVDT in Test 3 are shown in Figure 4.8. Transducer readings indicate that the maximum measured vacuum pressure is approximately 80 kPa at this depth. The subsequent release of vacuum pressure readily decreases the suction, and the reapplication of vacuum pressure increases the suction rapidly towards 80 kPa again. This also indicates that the suction head decreases with the drain depth, as the maximum suction of 100 kPa could not be maintained along the entire drain length. The settlement associated with combined vacuum and surcharge load is shown in Figure 4.8b, and it clearly reflects the effect of vacuum removal and re-application by the corresponding change of gradient of the settlement plot.

Figure 4.9 presents the measured excess pore water pressure at transducer T2 (240mm away from the bottom) and transducer T5 located 230 mm from top soil surface. The transducer T2 shows accelerated dissipation of excess pore water pressure compared to the transducer T5, because the location of transducer T2 is much closer to the top draining surface. Due to the fact that both these transducers T2 and T5 are located away from the surfaces at which the vacuum pressure is applied, the effect of vacuum removal and reapplication is not as clearly observed here, as in the case of Transducer T6.
Figure 4.7 Locations of transducers for Test 3

Figure 4.8 a) Suction in the drain (240mm from bottom) and b) surface settlement
surface settlement associated with simulated vacuum loading and removal

(a)

(b)
Figure 4.9 Pore pressure measured at transducers (a) T2 and (b) T5
Chapter 4 described the effectiveness of combined vacuum and surcharge preloading. Tests were carried out using the large-scale consolidometer for remoulded alluvial clay. Results clearly show that the rates of settlement and pore pressure dissipation increase when preloading is applied as a combination of vacuum and surcharge pressure. Removal and reapplication of vacuum pressure is often indicated by the settlement curve with a change in slope. The pore pressure indicates the change of vacuum pressure when the transducer is very close to the drain (e.g. transducer T6). When the transducers are away from the drain, changes could not be clearly seen from pore pressure data. This observation will be further discussed in Chapter 5. Although the drain length used in the laboratory test is relatively short, the reduction in vacuum pressure is still observed at the bottom of the drain.
5 NUMERICAL MODELLING OF VERTICAL DRAINS

5.1 Introduction

Although the final settlements associated with vacuum drainage and conventional surcharge preloading are the same, the loading mechanisms of the two techniques are distinctly different. In surcharge preloading, increase in total stress also leads to a higher effective stress, whereas in vacuum based consolidation, an increased effective stress is achieved due to reduced pore water pressure, while the total stress remains constant. In some past studies, vacuum pressure application has been commonly modeled either by introducing an equivalent (increased) surcharge pressure on the ground surface, or by ‘fixing’ a negative pore pressure distribution at the ground surface (Park et al., 1997). These methods have not been able to predict the observed pore pressures accurately, even though acceptable matching of settlements has been possible. Unsaturation of soil adjacent to the drained boundaries can occur due to the application of vacuum pressure through prefabricated vertical band drains (PVD) and soil surface.

Following initial laboratory simulation in a large-scale radial drainage consolidometer (Chapter 4), an attempt was made to explain the observed retardation of pore pressure dissipation around a single prefabricated vertical drain subjected to vacuum preloading, through a series of 2D plane strain finite element models incorporating the modified Cam-Clay theory. The piezometer results confirm that the suction head propagates to the bottom of the drain, but less than the applied maximum at the top. In view of these observations, a different approach was followed to analyse the vacuum assisted
consolidation around vertical drains. The writer has made an attempt to assess the different methods used in modeling vacuum application, evaluating the unsaturation effect due to vacuum application and factors affecting the soil unsaturation, as well as investigating some interesting problems such as the Mandel-Cryer effect.

5.2 Conversion of Three-Dimensional Drain Pattern to an Equivalent Two-Dimensional System

For construction sites with a large number of PVDs, two-dimensional (2D) plane strain conversion is the most convenient with regards to computational efficiency. It is far less time consuming than a three dimensional (3D) multi-drain analysis with each drain having its own axisymmetric zone, which substantially affects the mesh complexity and the corresponding convergence. In the analysis described in this chapter, axisymmetric to equivalent plane strain conversion was executed using the method initially proposed by Indraratna and Redana (1997) and modified to include vacuum pressure application as in Section 3.5. The modified relationship between the axisymmetric and plane strain permeability coefficients neglecting well resistance is given by:

\[
\frac{k'_{hp}}{k_{hp}} = \frac{0.67(1 - \frac{1}{n^2}) - \alpha}{\left\{ \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right)\ln(s) - 0.75 \right\} - \alpha}
\]

(5.1)

where, \( n = \frac{R}{r_w} \), \( s = \frac{r_e}{r_w} \) and \( \alpha = \frac{2(n-s)^3}{3(n-1)n^2} \)
If both the smear and well resistance are ignored, then the simplified ratio of plane strain to axisymmetric horizontal permeability is represented by:

\[
\frac{k_{hp}}{k_h} = \frac{0.67}{\ln(n) - 0.75}
\]  

(5.2)

In the above equations, \(k_{hp}\) and \(k_{hp}'\) are the plane strain permeability coefficients of the undisturbed and smear zone, respectively; \(k_h\) and \(k_h'\) are the corresponding permeability values in axisymmetric condition; \(r_w\) and \(R\) are the axisymmetric radii of the drain and its influence zone, respectively, and \(r_s\) is the radius of the smear zone. The value of \(\alpha\) represents the geometric transformation when converted from axisymmetric to the plane strain model.

### 5.3 Types of Finite Elements

The finite element software code CRISP based on the modified Cam-clay model (Britto and Gunn, 1987) has been used in the past by many researchers for soft clay embankment modeling. For the numerical analysis in this thesis, the writer has employed HKS/ABAQUS standard version 6.3, since ABAQUS has been designed as a flexible tool for multi-purpose finite element analysis including the unsaturation effect. It provides both linear and complex nonlinear response options involving contact, plasticity and large deformations. ABAQUS element library has 2D, 3D, axisymmetric and infinite elements to model consolidation behaviour. Infinite elements can be used to model the infinite domain, but gives a linear response only.
The basic element type in ABAQUS, which can be used for consolidation analysis, is the 4-node bilinear displacement and pore pressure element (CPE4P) consisting of 4 displacement and pore pressure nodes at the corners. The higher order of this element is the 20-node triquadratic displacement, trilinear pore pressure, reduced integration element (C3D20RP), which contains 20 displacement nodes and 8 pore pressure nodes. Reduced integration elements use a lower order integration to form the element stiffness. ABAQUS recommends the use of reduced integration elements when second order
elements are used, because, it usually gives more accurate results and is less expensive than full integration. The common element type used in the analysis presented here is the CPE8RP element, which contains 8 displacement nodes and 4 pore pressure nodes as shown in Figure 5.1 together with some other elements.

### 5.4 Modelling of Vacuum and Surcharge Preloading

In some past studies (eg. Park et al., 1997), vacuum application has been simulated by (1) simply fixing the negative pore pressure along the top boundary only or (2) increasing the surcharge pressure by a value equal to the vacuum pressure. However, the laboratory results indicated that the applied vacuum pressure could be effectively propagated towards the bottom of the drain, even though gradual loss of suction is inevitable with depth. This is also in agreement with some field data presented by Choa (1989). The efficiency of distribution of vacuum pressure along the drain length depends on the sealing (i.e. no air leaks) and the type of soil around the drain. If vacuum application is modelled as an increased (additional) surcharge pressure, then the maximum pore pressure (initial) will be equal to the total surcharge (i.e. conventional gravity surcharge + applied vacuum pressure), which decreases with time to zero.

Various analyses were conducted under plane strain conditions for a perfect unit drain to study the differences between each modelling technique. The discretised finite element mesh used in a typical analysis is shown in Figure 5.2, considering the drain length as 0.95m and the drain spacing as 0.45m. The mesh consists of 8-noded CPE8RP elements, which have 8 displacement nodes and 4 pore pressure nodes. Because of symmetry, it
was sufficient to consider one half of the unit cell. Three models were tested as described below.

Model 1 – Conventional, fully saturated soil with constant vacuum pressure of 50kPa with 50kPa surcharge pressure along the top soil surface. Vacuum pressure is modelled by fixing the negative pore pressure boundary at the top surface. At the drain boundary pore pressure is fixed to zero. The soft clay behaviour is fully governed by the modified Cam–clay properties (Table 5.1). The calculation of Cam-clay parameters based on oedometer test is given in Appendix B.

Model 2 – In addition to Model 1 conditions, vacuum pressure of 50kPa is applied along the drain by fixing the negative pore pressure along the drain boundary.

Model 3 – Vacuum pressure is modelled as an additional surcharge pressure. So 100kPa surcharge pressure is applied along the top soil surface. Drainage is allowed at the top surface by fixing the zero pore pressure at the boundary as well as along the drain depth.

Russel (1992) showed that even the lowest likely discharge capacity has a negligible effect on the rate of consolidation by doing two types of analysis. In the first approach, the drain was modelled using drainage elements with a discharge capacity of 140m³/year corresponding to the minimum likely value (Holtz et al, 1991). In the second, the drain was modelled as infinitely permeable by setting the excess pore water pressure to zero at the drain boundary. The average surface settlement for each analysis concludes the effect of discharge capacity is not significant on the rate of consolidation. Plane strain permeability coefficient for all 3 models was assumed to be 3.93 x 10⁻¹¹ m/s, as the basis of laboratory results discussed in Chapter 4.
Figure 5.2 Discritised finite element mesh

Table 5.1. Modified Cam-clay properties used in analysis

<table>
<thead>
<tr>
<th>Soil properties</th>
<th>Magnitudes</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\kappa$</td>
<td>0.05</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>0.15</td>
</tr>
<tr>
<td>Critical state line slope, $M$</td>
<td>1.1</td>
</tr>
<tr>
<td>Critical state void ratio, $e_{cs}$</td>
<td>1.55</td>
</tr>
<tr>
<td>Poisson’s ratio, $\nu$</td>
<td>0.25</td>
</tr>
</tbody>
</table>

$k, \lambda$: slope of specific volume vs. log pressure for swelling and consolidation, respectively.
The predicted settlements at the centre of the soil surface from 3 models are shown in Figure 5.3. The results show that irrespective of whether the vacuum pressure is modelled as a surcharge pressure or negative pore pressure, the predicted settlements are the same for both cases. Settlement rate is very slow if the vacuum pressure is applied on the top surface only, as expected. Models 2 and 3 predictions conclude that the nature of the consolidation pressure, either surcharge or vacuum has no effect on soil consolidation settlement.

![Figure 5.3 Predicted surface settlement](image-url)
Predicted pore pressure distribution for 4 different nodes in the mesh (Figure 5.2) is presented in Figure 5.4. It shows that although the consolidation settlement is the same for Model 2 and Model 3, the pore pressure distribution is different in the 3 models. Model 3 (simulated surcharge load only) generates a maximum positive excess pore water pressure equivalent to the total surcharge load of 100 kPa that dissipates to zero with time. In Model 2, the consolidation starts with excess pore water pressure
equivalent to the conventional surcharge pressure (50 kPa) and attains a negative pore pressure approaching the applied vacuum pressure (-50 kPa) with time. Consolidation of Model 1 starts with positive excess pore water pressure as in Models 2 and 3 and dissipates slowly with time. Depending on the time considered, the excess pore pressure becomes negative.

Figure 5.5 illustrates how the excess pore water pressure distributes throughout the soil cell after 50 days of consolidation based on the three models. It clearly shows that although the values are different, dissipation pattern is almost same for both Models 2 and 3. Model 1 shows the completely different pattern of pore pressure distribution due to the difference in load application.
Figure 5. 5 Contour plot of excess pore water pressure after 50 days
5.5 Soil MoistureCharacteristic Curve

Although often difficult to measure accurately, soil suction is one of the most important parameters governing the response of unsaturated soil, as positive pore water pressure governs the behaviour of saturated soils. The total soil suction can be divided into two components, namely matric and osmotic suction. Osmotic suction arises from the salt content in the pore fluid while the matric suction is usually related to capillary action. Matric suction strongly depends on the geometrical factors such as pore size and shape. Among various methods for measuring soil suction such as pressure plate apparatus, suction plate methods, pressure membrane, psychrometers, resistance methods and filter paper technique, the filter paper method is still the most widely used technique to measure suction (Houston et al., 1994) due to its simplicity and wide range of measurements (near zero to -100MPa). Soil suction in fine-grained soils is commonly higher than the suction in coarse or medium grained soils. Soil moisture characteristic curve, also called the water retention curve represents the variation of suction within the pores of a soil with the water content of the soil. This curve is generally plotted as the gravimetric water content, or volumetric water content or degree of saturation. Figure 5.6 shows the soil moisture characteristic curves developed by Houston et al. (1994) for silt, clay and sand.

5.6 Unsaturation of Soil due to Vacuum Preloading

In vacuum preloading it is possible that the soil becomes unsaturated owing to the application of the negative pore water pressure. The efficiency of vacuum preloading
can be retarded due to this unsaturation effect. In surcharge preloading, the change in shear strength can be described using the Mohr-Coulomb failure criterion for a saturated soil.

Figure 5.6 Soil moisture characteristic curves for three different soils using the filter paper method (Houston et al, 1994)
However, Leong et al. (2000) suggest that it is more appropriate to use Equation 5.3, which is known as extended Mohr-Coulomb failure criterion (Fredlund and Rahardjo, 1993) to describe the change in shear strength in vacuum preloading due to this unsaturation effect.

\[ \tau_{yy} = c' + \left( \sigma_f - u_a \right)_f \tan \phi' + \left( u_a - u_w \right)_f \tan \phi^b \]  \hspace{1cm} (5.3)

where \( c' \) is the effective cohesion intercept, \( \phi' \) is the effective angle of shearing resistance, \( \phi^b \) is the angle indicating the rate of change in shear strength relative to the change in matric suction \((u_a-u_w)_f\), \( u_a \) is the pore air pressure and \( u_w \) is the pore water pressure.

The analysis was conducted under plane strain condition for a ‘perfect’ unit drain to study the effect of soil unsaturation upon application of vacuum pressure. ABAQUS can be employed for solving problems related to partially saturated flow, and in this study, this capability has been extended to study the unsaturation behaviour owing to vacuum application. For this purpose, the soil moisture characteristic curve was introduced for CPE8RP elements governed by elastic properties. A summary of the suction-saturation relationship used in the analysis is given in Table 5.2 as Type A. The values are obtained from Figure 5.6 (curve for clay) in which soil moisture characteristic curves were developed using filter paper method by Houston et al. (1994). The ratio of apparent permeability in the unsaturated zone \( (k_u) \) to the fully saturated permeability \( (k_s) \) is evaluated as a cubic function of the effective degree of saturation \( (S_e) \), in the ABAQUS subroutines, thus,
\[
\frac{k_u}{k_s} = (S_e)^3
\]  

(5.4)

where, \(S_e = \frac{S_r - S_{ru}}{1 - S_{ru}}\), \(S_r\) = the degree of saturation and \(S_{ru}\) = the residual water saturation.

Table 5.2. Assumed suction-saturation relationship at PVD-soil boundary

<table>
<thead>
<tr>
<th>Type</th>
<th>Degree of Saturation ((S_r))</th>
<th>Suction (kPa)</th>
<th>Degree of Saturation ((S_r))</th>
<th>Suction (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.25</td>
<td>-100 000</td>
<td>0.1</td>
<td>-160</td>
</tr>
<tr>
<td></td>
<td>0.35</td>
<td>-10 000</td>
<td>0.15</td>
<td>-150</td>
</tr>
<tr>
<td></td>
<td>0.55</td>
<td>-1 000</td>
<td>0.3</td>
<td>-125</td>
</tr>
<tr>
<td></td>
<td>0.86</td>
<td>-100</td>
<td>0.55</td>
<td>-100</td>
</tr>
<tr>
<td></td>
<td>0.9</td>
<td>-80</td>
<td>0.8</td>
<td>-75</td>
</tr>
<tr>
<td></td>
<td>0.98</td>
<td>-20</td>
<td>0.95</td>
<td>-50</td>
</tr>
</tbody>
</table>

* \(k_u/k_s = (S_e)^3\) where, \(k_s\) = saturated permeability

In ABAQUS, effective stress in the unsaturated soil is defined based on the effective stress concept for unsaturated soil developed by Bishop (1959), which can be written as,

\[
\sigma' = \sigma - u_a + \chi (u_a - u_w)
\]

(5.5)

where, \(\sigma\) and \(\sigma'\) represent the total and effective stresses, respectively, \(u_a\) is the pore-air pressure, \(u_w\) is the pore water pressure as described earlier and \(\chi\) is the effective stress parameter depending on the degree of saturation and soil type. The value of \(\chi\) is equal to 1, when the medium is fully saturated, while it is between 0 and 1 when the medium is unsaturated. Many criticisms have been made in the use of Bishop’s equation because of the uncertainty of the value of \(\chi\), which depends on a number of factors such as the
degree of saturation, soil type and exhibits hysteresis effects due to wetting, drying and stress change (Khabbaz, 1997). Because of this uncertainty, ABAQUS simply assumes that $\chi$ is equal to the saturation of the medium.

The discritised finite element mesh used in the analysis is shown in Figure 5.2. The soil behaviour is governed by the elastic properties ($E=1000$ kPa, $\nu=0.25$) and plane strain permeability of $3.93 \times 10^{-11}$ m/s. Four models were numerically analysed.

**Model 1:** The soil is assumed to be fully saturated. Surcharge load (50kPa and 100kPa) is applied in two stages of which duration for each load increment is 20 days. Vacuum pressure of 50 kPa is made to act along the drain as well as along the top surface throughout the whole period of 40 days.

**Model 2:** Same as Model 1, but the soil moisture characteristic curve is included so that with the application of vacuum pressure, the unsaturated elements can be activated.

**Model 3:** Same as Model 1. Applied vacuum pressure is changed to 100 kPa.

**Model 4:** In addition to the Model 3 conditions, soil elements are made to become unsaturated upon application of vacuum pressure according to the soil moisture characteristic curve. Model 4 differs from Model 2, because 100 kPa vacuum is applied in Model 4 instead of 50 kPa vacuum in latter.
The change of saturation with time at elements 5, 15 and 20 (Figure 5.7) along the drain for Models 2 and 4 is shown in Figure 5.8. Time-saturation curves for Models 1 and 3 are not plotted, because, the degree of saturation remains unity throughout the consolidation process in both cases. With the application of vacuum, saturation decreases rapidly and after some time the rate of desaturation is retarded. Figure 5.8 also shows the effect of the drainage boundary on soil unsaturation. Element 20, as expected, remains more unsaturated than the other elements as a result of its proximity to the surface, which is also subjected to vacuum pressure apart from the drain boundary. Unlike element 5, which does not feel the effect of surface vacuum, Element 20 is
affected by the application of vacuum pressure on two surfaces. Figure 5.9 presents the change of saturation with time at elements 15,55,95,135 (Figure 5.7) along the radius of the cell for Models 2 and 4. It indicates that the change of saturation is high closer to the drain boundary where the effect of vacuum is high. When it comes to the undrained boundary saturation still remains at unity.

Figure 5.10 illustrates how the saturation changes throughout the soil cell with time for Models 2 and 4 after 40 days of consolidation. It clearly indicates that the soil closer to the boundaries where the vacuum pressure is applied becomes increasingly unsaturated, while the soil away from these boundaries still remains fully saturated.

Figure 5.11 shows the predicted surface settlement from the 4 models. The results show that the assumption of complete saturation with time (Models 1 and 3) predicts higher settlement than the predictions from Models 2 and 4, in which the soil becomes unsaturated with time. As expected, the settlement becomes larger with the increased vacuum pressure.

Figure 5.12 shows the predicted excess pore water pressure at 2 different nodes (1015 and 1031 in Figure 5.2), in the soil cell based on the 4 Models. As expected, Models 1 and 3 (fully saturated soil) indicate the lowest pore pressures, confirming that the unsaturated soil contributes to retarded pore pressure dissipation. The excess pore pressures plotted in Figure 5.12 are in compliance with the corresponding settlements shown earlier in Figure 5.11.
Figure 5.8 Change of saturation at the center of several elements along the drain boundary for (a) Model 2 and (b) Model 4
Figure 5.9 Change of saturation at the center of several elements along the radius of the cell for (a) Model 2 and (b) Model 4
Figure 5. 10 Contour plot of saturation after 40 days

Figure 5. 11 variation of settlement at the top of the soil cell
Figure 5. Excess pore pressure distribution at different nodes in the soil cell.
5.6.1 Effect of surcharge loading

To study the behaviour of soil unsaturation under various surcharge loadings, 3 numerical models were executed using the same discretised finite element mesh as shown in Figure 5.2. The 3 models are explained below.

*Model A* - vacuum pressure of 100 kPa is applied along the drain and as well as along the top surface for a duration of 40 days without any surcharge load. Soil moisture characteristic curve is included so that with the application of vacuum pressure, unsaturated elements can be activated.

*Model B* – with the vacuum pressure of 100 kPa as in Model A, surcharge load is applied in two stages (25kPa and 50kPa). The duration of each loading stage is 20 days.

*Model C* - similar to Model B except for the magnitude of surcharge load. In this case, it is 50kPa and 100kPa applied in two stages.

The soil behaviour for all three models was governed by the elastic properties ($E= 1000$ kPa, $\nu =0.25$) and a plane strain permeability coefficient of $3.93 \times 10^{-11}$m/s.

Figure 5.13 illustrates the predicted change of saturation with time at elements 15, 20, 55 and 95 (Figure 5.7) of the cell for Models A, B and C. It indicates that the incremental loading has some effect on soil unsaturation. However, the effect of magnitude of incremental loading is not significant. Upon absence of incremental loading, the three predicted curves would be parallel and the difference between the curves increases when the element is further away from the drainage boundaries.
Figure 5. 13 Change of saturation under different surcharge loading at several elements

5.6.2 Effect of soil type

As discussed in Section 5.5, soil moisture characteristic curve may vary from soil to soil. To study the effect of soil moisture characteristic curve, two different curves were tested on the same finite element mesh as shown in Figure 5.2. The two suction saturation relationships included in the analyses were given earlier in Table 5.2. For both cases,
vacuum pressure of 100kPa is applied along the drain boundary and along the top surface without any surcharge load.

Figure 5.14 Change of saturation with different moisture characteristic curves at several elements

Figure 5.14 illustrates how the degree of saturation changes with time with the 2 different soil types. As expected, elements close to the drain for type (eg. Elements 15 and 20) show increasingly reduced degree of saturation. For elements such as 55 and 95 which are further away from the drain boundary, much larger suction pressures are required to make the soil elements unsaturated (i.e. Type A) in comparison with the
Type B elements subjected to smaller suction pressures (also see Table 5.2). However, the results conclude the real necessity of accurately measuring the soil moisture characteristic curve in predicting accurate results.

5.6.3 Effect of initial saturated permeability

To study the behaviour of soil unsaturation attributed to the vacuum surcharge, three models with different initial permeability coefficients were tested (Table 5.3) using the same finite element mesh given in Figure 5.2. The soil moisture characteristic curve was included in all the three models so that upon the application of vacuum pressure, unsaturated elements could be activated. The degree of saturation within the soil mass is a function of the distance away from the drain boundary as well as depth.

Table 5.3 Description of Models

<table>
<thead>
<tr>
<th></th>
<th>Model 1</th>
<th>Model 2</th>
<th>Model 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surcharge load</td>
<td>50kPa and 100kPa in two stages; each loading stage lasts for 20 days</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vacuum pressure</td>
<td></td>
<td>50kPa throughout the 40 days</td>
<td></td>
</tr>
<tr>
<td>Saturated (initial) permeability (m/s)</td>
<td>3.93 x 10^{-11}</td>
<td>7.86 x 10^{-11}</td>
<td>1.97 x 10^{-10}</td>
</tr>
<tr>
<td></td>
<td>2 (3.93 x 10^{-11})</td>
<td>2 (3.93 x 10^{-11})</td>
<td>5(3.93 x 10^{-11})</td>
</tr>
<tr>
<td>Elastic modulus (E)</td>
<td></td>
<td>1000 kPa</td>
<td></td>
</tr>
<tr>
<td>Poissons ratio</td>
<td></td>
<td></td>
<td>0.25</td>
</tr>
</tbody>
</table>
Figure 5.15 shows the predicted changes of soil saturation at the centroid of four different elements (Elements 15, 20, 55 and 95 in Figure 5.7) in the cell, based on the three models. With the higher permeability, the drainage becomes quicker and the elements away from the drainage boundary will experience the suction generated by vacuum pressure more rapidly. Therefore, such elements become increasingly unsaturated with time as shown in Figure 5.15. For example in Model 3 (highest permeability), the degree of saturation is the least at any given time. Even if the saturated permeability is changed slightly, the subsequent effect is significant in terms of the change in soil saturation.

Figure 5. 15 Change of saturation with initial saturated permeability at several elements
5.7 Numerical Modelling of Large Scale Consolidometer Cell

Following the analyses described in Sections 5.5 and 5.6, the consolidation behaviour of soft clay in the large-scale consolidometer under combined vacuum and surcharge preloading was analysed using the finite element code ABAQUS, incorporating the modified Cam-clay theory (Rosco and Burland, 1968). The discretized plane strain finite element mesh is shown in Figure 5.16, where only one half of the cell is considered by exploiting symmetry. The mesh consists of eight noded linear strain quadrilateral elements (CPE8RP) with 8 displacement nodes and four pore pressure nodes positioned at the corners of each element. These elements can be implemented as elastic interface elements or elasto-plastic elements following the modified Cam-Clay theory for studying the behaviour of soft clays stabilised with vertical drains. The surcharge load is simulated by applying an incremental vertical load to the upper boundary.

To study the effect of unsaturation, the soil moisture characteristic curve was introduced for a thin layer of CPE8RP elements governed by elastic properties. In fact, the moisture characteristic curve developed for Moruya alluvial clay (tested by the writer) was similar in plasticity to remoulded silty clay with some fine sand discussed by Houston et al. (1994). Table 5.4 presents the suction-saturation relationship (Figure 5.6) used in the writer’s analysis with the calculated unsaturated permeability ($k_u$) according to the Equation 5.4, which is given below the table, for a degree of saturation exceeding 88%.

For the numerical analysis, the following 3 models were examined:
Figure 5. 16 a) Simulation of vacuum pressure along drain boundary b) Finite element discretization for plane strain analysis of the soil in large-scale consolidometer

Table 5.4. Assumed suction-saturation relationship at PVD-soil boundary

<table>
<thead>
<tr>
<th>Degree of Saturation ($S_r$)</th>
<th>Suction (kPa)</th>
<th>$k_u \times 10^{-11}$ (unsat. permeability, m/s)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.88</td>
<td>-100</td>
<td>2.05</td>
</tr>
<tr>
<td>0.90</td>
<td>-80</td>
<td>2.27</td>
</tr>
<tr>
<td>0.92</td>
<td>-50</td>
<td>2.50</td>
</tr>
<tr>
<td>0.94</td>
<td>-40</td>
<td>2.75</td>
</tr>
<tr>
<td>0.96</td>
<td>-30</td>
<td>3.02</td>
</tr>
<tr>
<td>0.98</td>
<td>-20</td>
<td>3.3</td>
</tr>
</tbody>
</table>

* $k_u / k_s = (S_e)^3$ where, $k_s$ = saturated permeability
Model 1 – Conventional, fully saturated soil with constant vacuum pressure along the top soil surface and linearly varying vacuum pressure along the drain depth is considered. The soft clay behaviour is fully governed by the modified Cam–clay properties (Table 5.1).

Model 2 – Initially the soil is assumed to be fully saturated. With the application of vacuum pressure (same as Model 1), a thin layer of unsaturated elements of predetermined thickness that does not vary with time is activated at the PVD boundary. For simplicity, the unsaturated layer of elements is modeled by elastic properties ($E=1000$ kPa, $\nu=0.25$), while the remaining outer soil elements are made to obey the modified Cam–clay theory.

Model 3 – Conditions are similar to Model 2 except, for the variation of vacuum pressure with time (including vacuum removal and reloading) as shown in Figure 5.17.

The surcharge load (two stages) for all 3 models was 50 kPa and 100 kPa, where each load was applied for duration of 14 days.

For all 3 models, equivalent plane strain permeability coefficients were calculated using Equation 5.1, where for the given dimensions of PVD (100mm x 3mm) and the diameter of the cell (450mm), the value of $\alpha$ was 0.13. Since the smear effect is more significant than the well resistance for relatively shorter drains, it has been assumed on the basis of previous data (Indraratna and Redana, 1998), that the smear zone is approximately four times the equivalent drain diameter. The converted permeability coefficients in the smear zone and undisturbed zone are $3.6 \times 10^{-11}$ m/s and $9.1 \times 10^{-11}$ m/s, respectively.
The change of degree of saturation with time at unsaturated elements 5, 10, 15 and 20 (Figure 5.16) along the drain for Model 3 is shown in Figure 5.18. With the removal of vacuum, this thin soil layer becomes saturated, but it becomes increasingly unsaturated again upon reapplication of vacuum.

The settlement of the soil surface predicted from the 3 models, together with the measured laboratory data is plotted in Figure 5.19. The results show that the assumption of complete saturation (Model 1) overpredicts the settlement most, while the analysis with an unsaturated soil layer at the drain boundary (Model 2) improves the accuracy of predictions. The consideration of vacuum pressure variation (Model 3) makes the predictions even closer to the laboratory data. The difference between Model 1 and Model 2 predictions is mainly attributed to the effect of thin unsaturated soil layer at the drain boundary, given the same modified Cam-clay properties used in both models for the remaining soil.
Figure 5.20 shows the location of selected pore pressure transducers in the laboratory consolidation cell. Figure 5.21 shows the predicted and measured values of excess pore water pressure at the transducer T1, which is located 230 mm below the soil surface and transducer T5, which is located 240 mm above the base of the cell. The observed pore pressures show acceptable agreement with the numerical models 2 and 3, where the drain boundary is assumed to be unsaturated. As expected, Model 1 (saturated drain boundary) indicates the lowest pore pressures, confirming that the unsaturated drain boundary contributes to the retarded pore pressure dissipation.

Figure 5.22 illustrates the excess pore pressure variation with time for elements located at varying radial distances from the drain boundary at two different levels coinciding with the depth of transducers T1 and T5. As expected, the elements closest to the drain boundary will feel the effect of vacuum loading, removal and reapplication more than the soil elements further away from the drain boundary. In Figure 5.22, this is reflected by more pronounced ‘peaks’ for r/R =0.09-0.18, in comparison with outer elements. For r/R >0.70, the vacuum pressure changes (loading and unloading) are not recorded to any considerable extent. The effect of varying the initial saturated permeability for r/R=0.45 at 2 different depths (transducers T1 and T5 levels) is shown in Figure 5.23. At increased values of permeability, the effect of vacuum loading, unloading and reloading is more marked, due to the more rapid drainage response.
Figure 5.18 Predicted variation of degree of saturation at the center of various elements along the drain boundary for Model 3

Figure 5.19 Predicted and measured settlement at the top of consolidometer cell
Figure 5.20 Schematic section of consolidometer cell showing the locations of pore pressure transducers

5.8 Mandel-Cryer Effect

Mandel-Cryer effect that is discussed by Mandel (1950,1953) and Cryer (1963) is characterized by an increase in the excess pore water pressure during early times of consolidation above the expected initial excess pore water pressure. While field observations are sparse, there is evidence that the effect has been observed in the development of excess pore water pressures in thick deposits of normally consolidated clay (Suklje, 1964). The lack of field evidence should be mainly due to the fact that in the field, outer boundary of the unit cell is not confined and the loading is not instantaneously applied (Xiao, 2000).

When one considers the drainage around a single vertical drain, in the zone of soil closest to the drainage boundary, the dissipation of excess pore water pressure takes place rapidly during the initial time period.
Figure 5. Predicted and measured excess pore water pressure at transducers T1 and T5
Figure 5. 22 Excess pore pressure distribution across the radius of the cell at the level of transducer (a) T1 and (b) T2
Figure 5. Effect of permeability on distribution of vacuum pressure at the level of transducer (a) T1 and (b) T5
This will cause the development of strains and the changes in effective stress in the
drained elements. At this time the soil elements near the outer boundary (larger drainage
path) has not yet begun to drain. To maintain the strain compatibility, part of the total
stress increments developed in the elements close to the drain will then be transferred to
the elements further away from the drain. Therefore during early stages of consolidation,
the excess pore pressure will show an increase over their initial value. The magnitude of
increase depends on the particular drainage conditions and soil properties.

For Model 3 (Section 5.4), Figure 5.24 shows the variation of normalized excess pore
water pressure (pore pressure at any time, Δu/initial pore pressure Δu₀) distribution
along the width of cell at two different sections of the unit cell. The figure clearly shows
the rapid dissipation of excess pore water pressure close to the drain boundary especially
at initial time increments. This may generate the Mandel- Cryer effect at the outer
elements away from the drain boundary, as clearly seen in both Figures when r/R ratio
exceeds around 0.4. It is also noted that the Madel- Cryer effect is less pronounced for
deeper soil elements (eg. Figure 5.24)

Looking carefully again at Figure 5.21, one can observe the Mandel- Cryer effect in the
response of pore pressure transducers T1 and T5. After the first stage loading, the
maximum pore pressure after about 2 days is slightly greater than the expected 30 kPa
(surcharge load), which can be attributed to the Mandel- Cryer effect as discussed
earlier.
Figure 5. Excess pore pressure distribution across the radius of the cell at the level of node (a) 615 and (b) 631.
5.9 Summary

In this chapter several numerical models were tested to investigate the proper modeling technique for vacuum preloading and to study the behaviour of soil unsaturation due to vacuum preloading.

Different techniques, which have been used for modeling vacuum preloading, conclude that to predict both settlement and pore pressure accurately, it is necessary to include the proper distribution of vacuum pressure along the drain length. Modelling of vacuum pressure as an additional surcharge load may give the same settlement, but the predicted pore pressures are different to the model that includes vacuum pressure as a negative pore pressure along drain boundary. Based on the analysis conducted here as well as the past field data and laboratory data presented in Chapter 3, it is more realistic to introduce a negative pore pressure boundary along the drain length as well as along the top surface.

The numerical models demonstrate the possibility of soil unsaturation due to vacuum pressure and the effectiveness of vacuum preloading is a function of soil permeability. The change in soil saturation depends on the saturated (initial) permeability, soil moisture characteristic curve (soil type) and the magnitude of the applied vacuum pressure. For obtaining realistic predictions, the use of an appropriate soil moisture characteristic curve, correct saturated permeability coefficient and the actual distribution of vacuum pressure (with time and depth) is important.
Finite element models were executed to predict the performance of a single vertical drain tested in large-scale consolidometer under vacuum pressure. The results clearly indicate that the inclusion of unsaturation effect and the actual distribution of vacuum pressure improves the accuracy of prediction. Mandel-Cryer effect was also noticed in these models, but its validation in a laboratory environment requires further testing.
6 APPLICATION OF NUMERICAL MODEL IN PRACTICE

6.1 General

The proposed site of the Second Bangkok International Airport (SBIA) is located in the Samutprakan Province, about 30 km east of the capital city Bangkok. In the past, the site was occupied by ponds for fish farming and agricultural usages. The area is often flooded during the wet season and the soil generally retains a very high moisture content. The site plan for the two test embankments TV1, TV2 is shown in Figure 6.1, which also shows the location of boreholes and field vane shear test prior to the construction of the embankments and the pumps.

Figure 6.1 Site plan for the test embankments at Second Bangkok International Airport (after Sangmala, 1997)
Figure 6.2 General soil profile and properties at Second Bankkok International Airport (after Sangmala, 1997)

The subsoil profile at the site can be divided into several sub layers as shown in Figure 6.2. It consists of a top weathered crust about 2m thick overlying a layer of soft clay approximately 10m thick. A medium clay layer underlies the soft clay and extends to a depth of 15.5m. The light brown stiff clay layer is encountered below 15.5m. Figure 6.2 gives the summary of average water content, Atterberg limits and the unit weight with depth. These parameters were obtained from 4 boreholes BH1, BH2, BH3 and BH4 carried out at the location of the test embankment TV1 and TV2, respectively, which were subsequently stabilised with vacuum assisted prefabricated vertical drains. The unit weight of the weathered clay is about 18 kN/m$^3$, and the lowest unit weight of the soil is about 14.5 kN/m$^3$ at 8.5 m depth.
The average compressibility parameters of the sub-soil are plotted in Figure 6.3. The compression index ratio ranges from 0.18 to 0.53. All other properties necessary for the analysis including modified Cam-clay parameters, vertical and horizontal coefficient of permeability will be discussed in the next section.

Figure 6.3 Average strength and compressibility parameters at Second Bangkok International Airport (After Sangmala, 1997)

Two test embankments TV1 and TV2 were constructed and stabilised with Prefabricated Vertical Drains (PVD) installed in a triangular pattern with 1m spacing to a depth of 15m and 12m, respectively. Each embankment with a base area of 40m x 40m was constructed with different drainage systems. In embankment TV1, hyper net drainage system was used, while perforated and corrugated pipes combined with nonwoven heat
bonded geotextiles were used in embankment TV2. The array of instrumentation of the embankments includes piezometers, surface settlement plates, multipoint extensometers, inclinometers, observation wells and benchmarks. The cross-section of one embankment including the vertical drain pattern at this location is shown in Figure 6.4. The type of drain installed in both embankments is Mebra (MD-7007) drains (100 mm x 3 mm) having grooved polypropylene channels wrapped in a nonwoven polypropylene filter. In these embankments, the drains were installed using a mandrel, which was continuously pushed into the soil using a static weight (in lieu of vibration), in order to reduce the soil disturbance (smear) as much as possible.

Figure 6.4 Cross section at embankment with sub-soil profile, Second Bangkok International Airport, Thailand
The geotextile utilized in the embankment TV1 consisted of 136g/m² nonwoven spunbonded polypropylene with high modulus. The hypernet used in the embankment TV2 consists of a grid of HDPE threads, which are melted together at their intersections. The perforated pipe is a five roll of mebra tube, which is 80mm diameter. In both embankments, on top of the drainage layer water and airtight LLDPE geomembrane liner was placed. The borders of the liner are completely sealed of from the atmosphere by placing the liner borders at the bottom of the trench. On the bottom of the trench, a 30cm thick layer of sand-bentonite was placed. The water collection system in each embankment was connected to a vacuum pump having a capability of supplying continuous vacuum pressure.

Figure 6.5 Construction loading history for embankments TV1 and TV2 at Second Bangkok International Airport

The embankment load in conjunction with the vacuum pressure was applied in four stages. For embankment TV2, Stage 1 loading is equivalent to 14.4kPa, which included the working platform also serving as a drainage blanket, while it was equivalent to
5.4kPa for embankment TV1. Both embankments were constructed in stages up to a maximum height of 2.5m. The loading history for both embankments is shown in Figure 6.5. Conventional load combined with vacuum pressure was applied throughout the construction period of about 5 months. During the application of vacuum pressure, it was felt that the suction head transmitted to the soil could not be maintained at the same level throughout the vacuum pressure application period as shown in Figure 6.6 (Sangmala, 1997). This fluctuation has not been uncommon in various soft clays, and has often been associated with air leaks through the surface membrane or the loss of suction head beneath the certain depth for long PVD. Intersection of natural macro-pores with drains at various depths can also lead to suction head drops. The pore pressure variation along the depth (Figure 6.6) was measured by piezometers installed at 3m spacings along the depth.

6.2 Numerical Analysis

The consolidation behaviour of soft clay beneath the embankment centreline under combined vacuum and surcharge preloading was analysed using the finite element code ABAQUS, incorporating the modified Cam-clay theory (Rosco and Burland, 1968). The Cam-clay parameters for each soil layer and the in-situ stress state under the embankment are given in Tables 6.1 and 6.2, respectively. The modified Cam-clay parameters include the gradients of specific volume against log pressure relations for consolidation and swelling ($\lambda$ and $\kappa$, respectively); slope of the critical state line based on effective stress ($M$); void ratio on critical state line (CSL) at unit normal effective pressure ($e_{cs}$); Poisson’s ratio ($\nu$); and saturated unit weight of soil ($\gamma_s$).
Figure 6.6 Observed variation of pore pressure with time and depth in the field for two embankments TV1 and TV2

Table 6.1 Modified Cam-clay parameters used in FE analysis for Second Bangkok International Airport (Asian Institute of Technology, 1995)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$e_o$</th>
<th>$\lambda$</th>
<th>$\kappa$</th>
<th>$\nu$</th>
<th>$M$</th>
<th>$\gamma$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2.0</td>
<td>1.8</td>
<td>0.3</td>
<td>0.03</td>
<td>0.3</td>
<td>1.2</td>
<td>16.0</td>
</tr>
<tr>
<td>2.0-8.5</td>
<td>2.8</td>
<td>0.73</td>
<td>0.08</td>
<td>0.3</td>
<td>1.0</td>
<td>14.5</td>
</tr>
<tr>
<td>8.5-10.5</td>
<td>2.4</td>
<td>0.5</td>
<td>0.05</td>
<td>0.25</td>
<td>1.2</td>
<td>15.0</td>
</tr>
<tr>
<td>10.5-13.0</td>
<td>1.8</td>
<td>0.3</td>
<td>0.03</td>
<td>0.25</td>
<td>1.4</td>
<td>16.0</td>
</tr>
<tr>
<td>13.0-18.0</td>
<td>1.2</td>
<td>0.1</td>
<td>0.01</td>
<td>0.25</td>
<td>1.4</td>
<td>18.0</td>
</tr>
</tbody>
</table>
Table 6.2 In-situ stress condition used in FE analysis for Second Bangkok International Airport

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$\sigma_{ho}'$</th>
<th>$\sigma_{vo}'$</th>
<th>u</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>5</td>
<td>5</td>
<td>-5</td>
</tr>
<tr>
<td>0.5</td>
<td>8</td>
<td>8</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>11</td>
<td>15</td>
</tr>
<tr>
<td>8.5</td>
<td>28</td>
<td>39.75</td>
<td>80</td>
</tr>
<tr>
<td>10.5</td>
<td>35</td>
<td>49.75</td>
<td>100</td>
</tr>
<tr>
<td>13.0</td>
<td>49</td>
<td>64.75</td>
<td>125</td>
</tr>
<tr>
<td>15.0</td>
<td>57</td>
<td>80.75</td>
<td>145</td>
</tr>
</tbody>
</table>

Table 6.3 Undisturbed and smear zone permeabilities for embankments TV1 & TV2

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$k_h$ (m/s)</th>
<th>$k_s$ (m/s)</th>
<th>$k_{hp}$ (m/s)</th>
<th>$k_{hp}'$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-1.0</td>
<td>3.01E-08</td>
<td>3.01E-09</td>
<td>8.98E-09</td>
<td>5.86E-10</td>
</tr>
<tr>
<td>1.0-8.5</td>
<td>1.27E-08</td>
<td>6.37E-10</td>
<td>3.80E-09</td>
<td>2.48E-10</td>
</tr>
<tr>
<td>8.5-10.5</td>
<td>6.02E-09</td>
<td>3.01E-10</td>
<td>1.80E-09</td>
<td>1.17E-10</td>
</tr>
<tr>
<td>10.5-13.0</td>
<td>2.55E-09</td>
<td>1.27E-10</td>
<td>7.60E-10</td>
<td>4.96E-11</td>
</tr>
<tr>
<td>13.0-18.0</td>
<td>6.02E-10</td>
<td>3.01E-11</td>
<td>1.80E-10</td>
<td>2.71E-12</td>
</tr>
</tbody>
</table>

The measured horizontal and vertical permeability coefficients of the undisturbed soil ($k_h$ and $k_v$), and their equivalent plane strain values based on Equation 5.1 as discussed in Chapter 5 are given in Table 6.3. The equivalent vertical band drain radius $r_w$ was determined using the method proposed by Rixner et al. (1998), which gives $r_w = 0.03$ m.
Based on large-scale consolidometer results as explained by Indraratna and Redana (1998), the radius of the smear zone was taken to be 0.15m, which is five times the radius of the mandrel. The equivalent plane strain width of drain and smear zone were taken to be the same as axisymmetric radii (Indraratna and Redana, 2000). For simplicity, the equivalent diameter of the soil cylinder was based on a drain spacing of $D_e = S$ (in theory, $D_e = 1.05S$ for a triangular grid), where $S$ is the field spacing. The error due to this simplification is small with respect to the predicted settlements, lateral displacements and pore pressures.

Single drain analysis was carried out to model the soil behaviour along the embankment centerline, while multi-drain analyses were carried out given the need for incorporating the effect of changing gravity along the embankment width as well the effect of drain spacing, to accurately predict settlement and lateral displacements underneath the embankment. While a single-drain analysis is often sufficient to model the soil behaviour at the embankment centerline, only a multi-drain analysis can provide the settlements, pore pressures and lateral displacements in the remaining regions. However, it is important to realise that the soft soil under wide embankments does not always experience a maximum settlement at the centerline, sometimes due to geometric and construction rate variations, deviating the centerline from the assumed line of symmetry (Du and Zhang, 2001). A true practice of soil consolidation under wide embankments can only be obtained by a multi-drain analysis.
For the single drain analysis, the discretized finite element mesh near the drain boundary is shown in Figure 6.7. Because of symmetry, it was sufficient to consider one half of the unit cell. Figure 6.8 presents a typical discretised finite element mesh used for multi-drain analysis for embankments TV1 and TV2, where one half of the embankment is sufficient to be considered due to symmetry.

The different drain lengths for the two embankments were modelled by fixing the pore pressure boundary along the appropriate depths. The meshes consist of linear strain quadrilateral elements (CPE8RP) with four pore pressure nodes at the corners. A finer mesh was employed for the zone of PVDs in multi-drain analysis, so that each unit cell represents a single drain and smear zone, unsaturated drain-soil boundary and undisturbed zone on either side of the drain. The instruments used to monitor the performance of the embankment (inclinometers and piezometers) were placed in the mesh in such a manner that the measuring points coincided with the mesh nodes. As shown in Figure 6.7, the electrical piezometer at 3m depth was placed in the mesh along the centreline of the clay body because it is located at 0.5 m away from the embankment centreline. The clay foundation was characterised by drained conditions at the upper boundary only, because of the presence of a stiff clay layer below 15m depth. The embankment loading was simulated by applying incremental vertical loads to the upper boundary, where the sequential construction history of the 2 embankments (Figure 6.5) was taken into account. At the exact locations of PVD, a pseudo-\( \lambda \) value of 0.1 was employed with a very high permeability in multi-drain analysis, while open consolidation boundary is used in single drain analysis.
Figure 6.7 Finite element mesh for plane strain single drain analysis
Figure 6.8 Finite element mesh of embankment for plane strain multi-drain analysis
The following 5 models were numerically examined under both plane strain single drain and multi-drain analysis:

*Model 1* – Conventional, fully saturated soil with constant vacuum pressure (-60kPa) along the top soil surface and also assumed constant along the drain length. The soil behaviour is fully governed by modified Cam–clay properties (Table 6.1).

*Model 2* – Initially the soil was assumed to be fully saturated. With the application of vacuum pressure (same as Model 1) a thin layer of unsaturated elements of predetermined thickness (30mm) that does not vary with time was activated at the drain boundary. For this purpose, the soil moisture characteristic curve was introduced for the thin layer of elements governed by elastic properties. A summary of the suction-saturation relationship used in the analysis is given in Table 6.4. The values were obtained from clay curve in Figure 5.6. For simplicity, the unsaturated layer of elements was modeled by elastic properties ($E = 1000$ kPa, $\nu = 0.25$). The remaining outer soil elements obeyed the modified Cam–clay theory.

*Model 3* – Similar to Model 2 with constant vacuum pressure (60 kPa) along the top soil surface, but a linearly varying vacuum pressure (60 kPa at top and zero at bottom) applied along the drain depth.

*Model 4* – Similar to Model 2, but the vacuum pressure was changed with depth and time as in the field (Figure 6.6). It is of interest to note that the most significant loss of vacuum pressure (suction) occurs within the very soft clay layer, ironically the soil layer that should benefit most by the vacuum application.

*Model 5* – For comparison purposes this most basic model employs surcharge load only (no vacuum) and does not contain any unsaturated elements along the PVD. The entire
soil behaviour is governed by the modified Cam–clay theory. This simple model is helpful to demonstrate the influence of vacuum pressure on settlement and pore pressure dissipation.

Table 6.4 Suction-saturation relationship

<table>
<thead>
<tr>
<th>Degree of Saturation ($S_r$)</th>
<th>Suction (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>-100 000</td>
</tr>
<tr>
<td>0.35</td>
<td>-10 000</td>
</tr>
<tr>
<td>0.55</td>
<td>-1 000</td>
</tr>
<tr>
<td>0.86</td>
<td>-100</td>
</tr>
<tr>
<td>0.9</td>
<td>-80</td>
</tr>
<tr>
<td>0.98</td>
<td>-20</td>
</tr>
</tbody>
</table>

All the models included the smear effect but neglected well resistance, as previous studies (Indraratna & Redana, 2000) indicated that well resistance was not significant for drain lengths shorter than about 20m. For embankment TV2, where the drains were installed to a depth of 12m, pore pressure boundary was fixed up to 12m depth only.

The results of the plane strain analysis based on 5 models together with the measured settlements of the unit cell are plotted in Figure 6.9. As expected, the settlement at the surface is well predicted by Model 4, which is included the actual distribution of vacuum pressure with respect to time and depth. The difference between Models 1 and 2 shows the effect of unsaturation at the drain-soil boundary owing to vacuum pressure. The difference is greater in embankment TV2, where the acting vacuum pressure is high.
compared to Embankment TV1. Nevertheless, even with reduced vacuum pressure, the benefits of vacuum assisted consolidation are clear.

The predicted surface settlements from multi-drain analysis are plotted in Figure 6.10, together with the field measurements for the 2 embankments. Careful observation of field settlement points suggests the possible loss of vacuum pressure (air leakage) with time. Although there was some discrepancy between the measured vacuum pressure in the sand mat and at the ground surface (Sangmala, 1997), the assumed time-dependent variation of vacuum pressure based on surface measurements (i.e. author’s Model 4) improves the accuracy of settlement predictions. The difference between Models 1 and 2 shows the effect of unsaturation at the drain-soil boundary owing to the vacuum pressure. Figure 6.11 shows the predicted and measured settlement at 6m depth for two embankments. Once again, settlements are generally well predicted by Model 4, which includes the field variation of vacuum pressure. Even with the reduced vacuum pressure, the benefits of vacuum assisted consolidation are obvious.

Figures 6.12 and 6.13 present the predicted and measured pore water pressures at 3m and 6m depths for the embankments TV1 and TV2. The ‘sharp’ peaks of the numerical models accurately represent the sudden rise of pore pressure associated with various loading stages. However, the piezometer data do not show these sharp increases, as the dates of field measurements do not coincide with the exact times at which the maximum peaks are predicted. The field measurements at both depths indicate the increase in pore
Figure 6.9 Settlement at ground surface for embankment (a) TV1 and (b) TV2
water pressure after about 110 days. This verifies that the constant suction could not be sustained during the entire period of construction, and Model 4 provides a reasonable match to this behaviour by including the time-dependent vacuum pressure variation shown earlier in Figure 6.6. All other numerical models that do not include the time-dependent vacuum pressure variation cannot predict the field behaviour to an acceptable accuracy.

Lateral deformations measured by the inclinometer installed at the toe of the embankments together with the predicted results are shown in Figure 6.14. Not surprisingly, all 4 models incorporating vacuum pressure cause ‘inward’ lateral movements. Even though the field measurements of TV1 and TV2 do not represent this ‘inward’ trend (Figure 6.14), the role of vacuum pressure in reducing the otherwise inevitable lateral movement is clearly demonstrated by comparing with Model 5 (no vacuum). The effect of the compacted crust is not clearly reflected by the field data, which suggests that the depth of the crust is no more than 1m in the field, whereas the numerical analysis assumed the depth of the crust to be 2m. Given the difficulties of modelling lateral displacements in plane strain as discussed clearly by Poulos (1972) and Indraratna et al. (1997), Model 4 still provides an acceptable comparison. It is noted that the stiff over-consolidated crust near the surface will reduce the sensitivity of lateral movement reduction upon the application of vacuum pressure. The effect of loss of vacuum head increases the lateral movements more towards Model 5 as evident from Figure 6.14.
Figure 6.10 Surface settlement for embankments (a) TV1 and (b) TV2 of SBIA
Figure 6.11 Surface settlement at 6m depth for embankments (a) TV1 and (b) TV2
Figure 6.12 Variation of excess pore water pressure at 3m depth below ground level, 0.5m away from the centerline for embankments (a) TV1 and (b) TV2
Figure 6.13 Variation of excess pore water pressure at 6m depth below ground level, 0.5m away from the centerline for embankments (a) TV1 and (b) TV2
Figure 6.14 Lateral displacement profiles through the toe of the embankments (a) TV1 and (b) TV2 after 150 days.
Figure 6.15 Surface settlement profile predicted for embankments (a) TV1 and (b) TV2 after 150 days.
Figure 6.15 shows the predicted surface settlement for embankments TV1 and TV2 based on all the 5 models after 150 days. However, only one-field measurement was available to compare with the numerical analysis. Predictions verify that there is no heave taking place in the presence of vacuum pressure. As expected, Model 5 (no vacuum) shows significant heave closer to the toe of the embankment, while the other models do not. Model 1 results are omitted for the purpose of clarity in Figure 6.15.

6.3 Summary

The performance of two test embankments stabilised with vertical drains subjected to vacuum loading was investigated using a plane strain finite element analysis. It has been demonstrated that the inclusion of time and depth dependent vacuum pressure distribution improves the settlement prediction significantly, but the pore pressures are harder to match. Apart from the settlement, the pore pressure and lateral displacements were also analysed. The best predictions of settlement and pore pressures were obtained when the numerical analysis (Model 4) included the time-dependent change in vacuum pressure in addition to having an unsaturated layer of elements along the PVD. Further numerical refinements may be achieved by improved procedures for accurate modelling of the surficial over-consolidated crust that does not strictly obey the modified cam-clay theory.

Three of the numerical models consider the effect of a thin layer of unsaturated soil at the drain boundary. The application of vacuum pressure increases the rate of pore
pressure dissipation due to the increased pore pressure gradient towards the drain. It has been demonstrated that the vacuum preloading was beneficial, but the occurrence of soil unsaturation at the PVD boundary could retard the pore pressure dissipation, as clearly reflected by the corresponding difference in settlement curves. The presence of even a thin unsaturated soil layer at the PVD boundary could retard the efficiency of vacuum application. The use of appropriate suction-permeability relationships is important in obtaining realistic predictions, if the soil adjacent to the vacuum affected PVD becomes unsaturated. Therefore, the adoption of correct soil moisture characteristic curves in conjunction with numerical modelling is highly desirable.

As also confirmed by field studies (Choa, 1989), vacuum effect may decrease substantially with depth due to various practical limitations, improper sealing, and the nature of soil conditions (e.g. presence of fissures and macro-pores). Therefore, the assumption of diminishing suction values along the drain depth is justified in the finite element modelling. Further research with ‘instrumented PVD’ in the field may provide further insight to the vacuum pressure distribution with depth in the stabilisation of soft clays. Although the vacuum assisted soft clay improvement is more costly than the conventional surcharge loading, the use of sufficient vacuum pressure with proper sealed surface membranes will significantly accelerate pre-construction settlement, thereby compensating for the initial capital costs by enhanced speed of construction and minimising post-construction settlement.
7 A FEM PERSPECTIVE FOR GENERAL DESIGN

7.1 General

The rapid development strategies and the associated urbanization in certain parts of the world have compelled engineers to construct earth structures such as embankments and major highways over soft clay deposits having low bearing capacities coupled with excessive settlement characteristics. Especially in Southeast Asia, soft clays are fairly widespread, and some of these deposits exist extensively in the vicinity of capital cities. As these soft soils are weak, unreinforced embankments are usually built to a relatively small height of approximately 3m to 4m. However, higher embankments are often needed and rapid construction of these embankments is also important. To achieve these goals, special construction measures such as the use of light weight embankment fill, the provision of reinforcement at the bottom of the embankment, use of suitable ground improvement technique and staged construction of embankment must be considered.

In staged construction, after each stage of loading, a rest period is allowed for dissipation of excess pore water pressures in the soft clay. Use of ground improvement techniques such as installation of prefabricated vertical drains or application of vacuum pressure with staged construction improves the rate of pore pressure dissipation. Such dissipation is accompanied by consolidation and a gain in the soil strength. This increased soil strength enables the embankment to be raised to a greater height in the next stage of construction. At the design stage, it is necessary to decide the factors such as number and timing of the construction stages and the number and locations of the
drains that must be provided in the foundation, magnitude of the vacuum pressure and slope of the embankment. With the availability of sophisticated finite element software, finite element analysis becomes an important part of the current design processes (Potts and Zdravkovic, 2000). Potts and Zdravkovic (2000) have demonstrated the effect of surface crust, reinforcement and anisotropic soil behaviour on the failure height of embankment.

In this chapter, selected numerical studies have been carried out to study the effect of embankment slope, construction rate of embankment, drain spacing and application of vacuum pressure on the failure of soft clay foundation. The effect of surface crust is also examined. The main objective is to demonstrate how the research know-how described in previous Chapters 5 and 6, can be employed to predict the stability of embankments in a systematic manner, under various conditions. The 'tell-tale' sign of failure are simulated and discussed, purely through a finite element methodology.

7.2 Embankment Constructed on Soft Clay without any Improvement

7.2.1 Soil conditions

The subsoil profile considered in the analysis is assumed to consist of 5 sub layers, namely weathered clay, very soft clay, soft clay, medium clay and stiff to hard clay. The Cam-clay parameters, initial stress conditions and plane strain permeability coefficients
for smear zone \((k'_{hp})\) and undisturbed zone \((k_{hp})\) with different undisturbed permeability \((k_h)\) to smear zone permeability \((k_s)\) ratios are given in Tables 7.1, 7.2 and 7.3.

Table 7.1 Modified Cam-clay parameters used in FE analysis

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>(e_o)</th>
<th>(\lambda)</th>
<th>(\kappa)</th>
<th>(\nu)</th>
<th>(M)</th>
<th>(\gamma) (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2.0</td>
<td>1.8</td>
<td>0.3</td>
<td>0.03</td>
<td>0.3</td>
<td>1.2</td>
<td>16.0</td>
</tr>
<tr>
<td>2.0-8.5</td>
<td>2.8</td>
<td>0.73</td>
<td>0.08</td>
<td>0.3</td>
<td>1.0</td>
<td>14.5</td>
</tr>
<tr>
<td>8.5-10.5</td>
<td>2.4</td>
<td>0.5</td>
<td>0.05</td>
<td>0.25</td>
<td>1.2</td>
<td>15.0</td>
</tr>
<tr>
<td>10.5-13.0</td>
<td>1.8</td>
<td>0.3</td>
<td>0.03</td>
<td>0.25</td>
<td>1.4</td>
<td>16.0</td>
</tr>
<tr>
<td>13.0-18.0</td>
<td>1.2</td>
<td>0.1</td>
<td>0.01</td>
<td>0.25</td>
<td>1.4</td>
<td>18.0</td>
</tr>
</tbody>
</table>

Table 7.2 In-situ stress condition used in FE analysis

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>(\sigma'_{ho}) (kPa)</th>
<th>(\sigma'_{vo}) (kPa)</th>
<th>(u) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>5</td>
<td>5</td>
<td>-5</td>
</tr>
<tr>
<td>0.5</td>
<td>8</td>
<td>8</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>11</td>
<td>15</td>
</tr>
<tr>
<td>8.5</td>
<td>28</td>
<td>39.75</td>
<td>80</td>
</tr>
<tr>
<td>10.5</td>
<td>35</td>
<td>49.75</td>
<td>100</td>
</tr>
<tr>
<td>13.0</td>
<td>49</td>
<td>64.75</td>
<td>125</td>
</tr>
<tr>
<td>15.0</td>
<td>57</td>
<td>80.75</td>
<td>145</td>
</tr>
</tbody>
</table>
7.2.2 Finite element mesh

Figure 7.1 presents a typical discretised finite element mesh, which is used for analysis in this chapter, where one half of the embankment is sufficient to be considered due to symmetry. The basic input file used to generate the models in this Chapter is given in Appendix C. A foundation depth of 15m was considered adequate for the purpose of analysis because of the existence of stiff clay layer beneath this depth. The lateral boundary of the finite element is defined 60m away from the embankment centreline. The mesh consists of linear strain quadrilateral elements (CPE8RP) with four pore pressure nodes at the corners. A finer mesh was employed for the zone under the embankment, which is considered, has a width of 20m. In the case of vertical drain installed foundation, vertical drains are simulated by a line of zero pore pressure nodes at the exact locations of PVD. The clay foundation was characterised by drained conditions at the upper boundary only, assuming the presence of a stiff clay layer below 15m depth. The embankment loading was simulated by applying incremental vertical loads.

Table 7.3 Axisymmetric and plane strain permeabilities used in the analysis

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$k_h$ (m/s)</th>
<th>$k_{hp}$ (m/s)</th>
<th>$k'_{hp}$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$k_h/k_s=2$</td>
<td>$k_h/k_s=5$</td>
<td>$k_h/k_s=10$</td>
</tr>
<tr>
<td>0-1.0</td>
<td>3.01 x 10^{-8}</td>
<td>8.98 x 10^{-9}</td>
<td>2.42 x 10^{-9}</td>
</tr>
<tr>
<td>1.0-8.5</td>
<td>1.27 x 10^{-8}</td>
<td>3.80 x 10^{-9}</td>
<td>1.02 x 10^{-9}</td>
</tr>
<tr>
<td>8.5-10.5</td>
<td>6.02 x 10^{-9}</td>
<td>1.80 x 10^{-9}</td>
<td>4.83 x 10^{-10}</td>
</tr>
<tr>
<td>10.5-13.0</td>
<td>2.55 x 10^{-9}</td>
<td>7.60 x 10^{-10}</td>
<td>2.05 x 10^{-10}</td>
</tr>
<tr>
<td>13.0-18.0</td>
<td>6.02 x 10^{-10}</td>
<td>1.8 x 10^{-10}</td>
<td>4.83 x 10^{-11}</td>
</tr>
</tbody>
</table>
Figure 7.1 Finite element mesh
7.2.3 The effect of the slope of the embankment

To illustrate the effect of the embankment slope on foundation failure, two plane strain finite element analyses have been performed using the finite element mesh shown in Figure 7.1. Two slopes are considered in the analysis as 2:1 and 3:1. The influence of embankment loading is simulated by a constant rate of 0.1m/week using the AMPLITUDE option, which represents the time-dependent variation of loading in ABAQUS. Failure is identified when the solution fails to converge and displacement continues to increase without further addition of load.

Hunter and Fell (2003) have analysed the deformation behaviour and excess pore pressure dissipation of 13 well-monitored embankments on soft clay constructed to failure. They observed that by monitoring lateral displacement at the toe of the embankment and vertical displacement at the toe and about 5m beyond the toe, failure of embankments on soft clay could be detected. It was noticed that the excess pore pressure response at the centreline of the embankment would not provide an indication of failure, because these points were generally not located within the localised failure zone.

Predicted centreline surface settlements from two models are shown in Figure 7.2. As expected, settlements increase at the same rate with the increase of embankment height. The settlement curves do not provide a clear indication of the failure point. Figure 7.3 shows the predicted settlement (in this case it is heave) at the toe of the embankment based on the two models. A measurable change in settlement rate is observed close to failure, and finally, the settlement increases without having to increase the embankment
Figure 7.2 Settlement at the center of the embankment foundation

Figure 7.3 Settlement at the toe of the embankment
height. The decrease in embankment slope has the effect of increasing the embankment height at failure from 1.8m to 2.1m. Figure 7.4 presents the excess pore pressure distribution at 2m depth below ground level at two locations, one is 0.5 m away from the embankment centreline and the other is at the embankment toe. As expected, according to the load application, excess pore pressure increases uniformly at the point close to the centreline (Figure 7.4a). When this point is close to the embankment toe, increment is not gradual and finally there is a sudden increase, because the point considered is located within the expected failure zone.

Predicted surface settlement profile at failure based on two models is presented in Figure 7.5. In contrast to the expected variation, maximum settlement is predicted at a point approximately 10 m away from the centreline. Du and Zhang (2001) discussed about the four possible settlement patterns according to the characteristics of the shear stress distributions, namely one dimensional, sagged, transitional and typical (Figure 7.6). If an embankment is extremely wide compared to its height, the stress condition in the central area of the foundation is approximately one-dimensional, and those in the edge areas are two-dimensional. In many cases, the width of an embankment is not so large as to result in a shear-free central area, but also not small enough to form a typical settlement basin (i.e. maximum at centreline). The settlement pattern with the embankment is a transit from the 'typical' type to the 'sagged' type and is characterised by a wide flat bottom as seen in Figure 7.6.
Figure 7.4 Excess pore pressure distribution at 2 m depth (a) 0.5 m away from the centerline (b) at the embankment toe
Fast field studies reveal this type of settlement patterns. In the Muar test embankment in Malaysia (Indraratna et al., 1993), the maximum settlements were observed beneath the edge of the embankment at 2m and 3m heights. This location gradually shifted toward the center as the construction of the embankment proceeded. In the parking terminal of the of the Shenzhen airport in South China (Wang and Xiong, 1992), the treated area is 400 m x 400 m wide and the monitored foundation settlement were large close to the edges but smaller in the central area.

### 7.2.4 Effect of loading rate of the embankment

To study the effect of construction rate of the embankment on failure height, plane strain finite element analyses have been conducted for two different construction rates using the finite element mesh (Figure 7.1) discussed in Section 7.2.2. Soil conditions are the
same as discussed in Section 7.2.1. Embankment slope is 3:1 and two different construction rates considered in the analysis are 0.1 m /week and 0.35 m /week.

Figure 7.6 Patterns of settlement under embankment (Du and Zhang, 2001)

Figure 7.7 illustrates the predicted centreline surface settlement based on a plane strain finite element analysis carried out using the two different construction rates as stated earlier. Higher construction rate gives a higher rate of settlement, which is uniform with time. However, it does not give any indication of failure.
Figure 7.7 Predicted centerline surface settlement for different construction rates

Figure 7.8 Settlement (heave) at the toe of the embankment
The predicted heave at the toe of the embankment based on the two construction rates is shown in Figure 7.8. The positive values of settlement indicate that there is heave at the toe for both construction rates. As discussed earlier, the rate of settlement changes considerably without any significant increase in the embankment height. The slow rate of construction permits a greater embankment height at failure, because, this gradual rate allows the soft clay to gain shear strength upon pore pressure dissipation. Figure 7.9 shows the contour plots of resultant displacement vectors. It can be clearly identified that the failure surface of the soft clay foundation is directly 'pictured' by these displacement vectors. Maximum resultant displacement vectors occurred around the toe of the embankment, where the lateral displacements are much greater than their vertical counterparts.

![Figure 7.9 Contours of resultant displacement for construction rate of (a) 0.1m / week (b) 0.35m / week](image)
7.3 Staged Construction and Effect of Surface Crust

In the situations discussed above, the embankment is likely to fail at smaller heights, because the strength of the foundation soil is not sufficient to withstand higher loads. In this kind of situation, the potential failure can be avoided by halting construction and then allowing a sufficiently long period of time to dissipate pore pressure. This additional shear strength (upon draining) supports the increased height of embankment during the next loading stage.

To evaluate how the staged construction affects the failure height of embankment, a plane strain analysis was conducted using the same finite element mesh shown in Figure 7.1 and for the same subsoil properties. Effect of surface crust is also studied. With the presence of surface crust, \( \lambda = 0.1 \) (less than normally consolidated clay) is used for top layers of over consolidated clay up to a depth of 2m. The slope of the embankment is assumed to be 3:1. The embankment is assumed to be raised in four stages up to 4m, and the assumed construction loading history is shown in Figure 7.10.

Figure 7.11 illustrates the predicted centerline surface settlement for the two cases, with and without the surface crust. It verifies that the presence of a compacted surface crust decreases the surface settlement, and increases the failure height of embankment. This is the same effect as having a geogrid underneath the embankment. Predicted heave at the toe of the embankment based on two cases is shown in Figure 7.12. Both embankments are raised to failure before reaching the final stage of loading, which concludes that the allowed rest periods are not enough for gaining sufficient strength. Lo and Stermac
(1965) and Stermac et al. (1967) reported that in some cases, there may be little or no measurable strength gain within a reasonable time period and the conventional stage construction may not be always useful. However, the presence of the surface crust has the effect of increasing the embankment height at failure from 2.4m to about 3.0m. Figure 7.13 shows the predicted excess pore pressure variation at 2m depth below the embankment toe. It shows that when the embankment reaches its failure height, there is a sudden increase in pore pressure. Moreover, towards the failure the difference between 'crust' and 'no crust' situations is significant with respect to predicted pore pressure.

Figure 7.10 Construction loading history
Figure 7.11 Centerline surface settlement

Figure 7.12 Predicted heave at the toe of the embankment
As observed in the previous section, in thick deposits of soft clay, the time required for
dissipating excess pore pressure and gaining additional strength under a applied load
may unacceptably long and instability may be a serious concern. In such cases, it is
often advantageous economically to install vertical drains in the soft clay foundation to
decrease the length of drainage path, and thereby speed up consolidation. It will reduce
the rest periods between each stage, thereby accelerating the construction of
embankment. If there are any berms, size of the berm can be reduced. To investigate the
effect of vertical drains on embankment stability, a plane strain analysis was conducted

Figure 7.13 Predicted excess pore pressure distribution at 2m depth below the embankment toe

### 7.4 Embankments Constructed on Soft Clay Stabilized with Vertical Drain

As observed in the previous section, in thick deposits of soft clay, the time required for
dissipating excess pore pressure and gaining additional strength under a applied load
may unacceptably long and instability may be a serious concern. In such cases, it is
often advantageous economically to install vertical drains in the soft clay foundation to
decrease the length of drainage path, and thereby speed up consolidation. It will reduce
the rest periods between each stage, thereby accelerating the construction of
embankment. If there are any berms, size of the berm can be reduced. To investigate the
effect of vertical drains on embankment stability, a plane strain analysis was conducted
using the discretised finite element mesh shown in Figure 7.1. Four different drain spacings were considered in the analysis; 1 m, 1.5 m, 2 m and 3 m. The Embankment is assumed to be raised to a maximum height of 4 m, with 2 different construction rates, 0.1 m / week and 0.35 m / week. Total time period considered in the analysis is 1 year. Slope of the embankment is assumed to be 2:1 in this case.

Figure 7.14 shows the predicted centerline surface settlement in the presence of vertical drains. Higher drain spacing increases the drainage path and leads to a slower consolidation rate. Three curves except for 3 m drain spacing reach the ultimate settlement after 300 days (Figure 7.14(a)). For the construction rate of 0.1 m / week, impending failure is not noticed for smaller drain spacing up to 2 m, which suggests that the higher dissipation of pore pressure and slow construction rate allow the soft clay foundation to gain the required strength to support the 4 m height embankment. If construction rate is increased to 0.35 m / week, the foundation stabilized with PVD of 1 m spacing reaches its ultimate settlement within a shorter period (100 days) compared to the construction rate of 0.1 m / week (300 days), as expected. However, it is unable to reach the final embankment height if the drain spacing is 1.5 m, 2 m or 3 m (Figure 7.14b).
Figure 7.14 Predicted centerline surface settlement for different drain spacing at a construction rate of (a) 0.1m / week (b) 0.35m / week
Figure 7.15 illustrates the settlement at the toe of the embankment for different values of drain spacing. No heave is predicted at the toe if the drain spacing is 1m or 1.5m and if the construction rate is 0.1m / week. However, heave is predicted with the increased embankment height for an increased drain spacing (2 m and 3 m). For higher construction rate as shown in Figure 7.15b, more heave is predicted (except for 1m drain spacing) for smaller height of embankment, and finally indicating the possible failure before reaching the maximum height. These results conclude that smaller drain spacing can increase the embankment height at failure by reducing the length of drainage path, and thereby accelerating the pore pressure dissipation. This expectation is further supported by the excess pore pressure distribution, which is shown in Figure 7.16. Foundation with 1.5 drain spacing shows the higher dissipation rate at the point of interest because the last drain is only a 0.5m away from the toe of the embankment. For the other three cases last drain is 1m and 2m away from the toe. However, the pore pressure distribution at a point 0.5m away from the embankment and at 2m depth (Figure 7.17) shows the expected variation, i.e., higher the dissipation for reduced drain spacing. As discussed by Hunter and Fell (2003), indication of failure is not yet noticed because the point considered is not within the failure zone (Figure 7.17b). Figure 7.18 presents the contour plots of resultant displacement vectors for different drain spacing at a construction rate of 0.35m / week. It clearly indicates the development of a potential failure surface with the increase of drain spacing from 1 m to 3 m.
Figure 7.15 Predicted surface settlement at the toe of the embankment for different drain spacing at a construction rate of (a) 0.1m / week (b) 0.35m / week
Figure 7.16 Predicted excess pore pressure distribution at 2m depth below the toe of the embankment for different drain spacing at a construction rate of (a) 0.1m / week (b) 0.35m / week
Figure 7.17 Excess pore pressure distribution at 0.5m away from the centerline and 2m depth for different drain spacing at a construction rate of (a) 0.1m / week (b) 0.35m / week
Figure 7.18 Contour plots of resultant displacement for drain spacing (a) 1 m (b) 1.5 m (c) 2 m (d) 3 m (construction rate = 0.35 m / week)
7.4.1 Effect of smear

Finite element analysis has been conducted to investigate the effect of smear on the stability of vertical drain installed in soft clay using the finite element mesh shown in Figure 7.1. For the purpose of using the same mesh, one column of elements is employed with reduced permeability. In the field, radius of the smear zone is normally around 100-150 mm. Three different ratios of undisturbed permeability to disturbed permeability are considered in the analysis; 2, 5 and 10. The converted permeability coefficients are given in Table 7.3. Slope of the embankment, drain spacing and construction rate considered in the analysis are 2:1, 1 m and 0.1 m/week, respectively.

Figure 7.19 presents the predicted centerline surface settlement for the 3 different permeability ratios. As expected, higher settlement is shown by the lower permeability ratio. Any special observations have been not noticed at the failure point, which means that the point of interest is not located within the failure zone. As seen in Figure 7.20, more heave is predicted at the toe of the embankment, when the ratio of undisturbed permeability to smear zone permeability is relatively high (5 or 10), which decreases the pore pressure dissipation. If the ratio is equal to 2, the height of embankment can be increased to its final height without endangering its stability. Similar observations can be found from pore pressure plots as shown in Figures 7.21. When the embankment reaches its failure height, the sudden increment in pore pressure is noticed at a point below the embankment toe (Figure 7.21b).
Figure 7.19 Predicted centerline settlement for different permeability ratio

Figure 7.20 Predicted heave/settlement at embankment toe
Figure 7.21 Predicted excess pore water pressure at 2 m depth (a) 0.5 m away from the center line (b) at toe
Figure 7.22 presents the contour plots of displacement vectors. It clearly indicates how the contours of maximum displacement change to create a shear failure surface with the increase of permeability ratio.

Figure 7.22 Contour plots of resultant displacement vectors for (a) \( k_h/k_s = 2 \) (b) \( k_h/k_s = 5 \) and (c) \( k_h/k_s = 5 \)
7.5 Embankments Constructed on Soft Clay Subjected to Vacuum Loading

As seen in earlier sections, it is difficult to construct fill embankment several meters high rapidly on soft clay. Several solutions are currently been used in practice to increase the height of embankment to a final desired height. Construction of berms requires space and increased earthwork costs; stage construction loading is time consuming and lightweight fill is generally costly. However, the application of vacuum pressure combined with conventional surcharge preloading above PVD, will strengthen the soft soil by accelerated consolidation and will allow the embankment to be constructed to its final height at a rapid rate of construction.

7.5.1 Effect of drain spacing upon vacuum application

To numerically examine the effect of vacuum application on embankment stability, plane strain analysis was conducted using the discretised finite element mesh (Figure 7.1). Three different drain spacings were considered in the analysis, namely, 1m, 1.5m and 2m. Embankment is assumed to be constructed to a maximum height of 4m with a construction rate of 0.35m / week, and a slope of 2:1. Constant vacuum of 50kPa is applied along the top surface beneath the embankment and along the drain length by fixing the pore pressure boundary as discussed in Chapter 5. Total time duration considered in the analysis is one year.
Figure 7.23 presents the predicted surface settlement at the centerline of the embankment. As expected maximum settlement is predicted for the smaller drain spacing (1m), where the vacuum pressure distributed throughout the soil depth due to close drain spacing. With the application of vacuum, the time taken to reach the ultimate settlement is less compared to the foundation stabilized with vertical drain only without vacuum pressure (compare Figure 7.23 with 7.14b).

Predicted surface settlement at the toe of the embankment is shown in Figure 7.24, where three different drain spacings are considered at a construction rate of 0.35m / week. Negative settlement predictions indicate that there is no heave at the point of interest. Gradual increase in settlement with the embankment height up to 4m indicates that there is no indication of failure within the period considered in this analysis.

Figure 7.25 illustrates the pore pressure distribution at 2m depth below the toe of the embankment. Surprisingly, a higher pore pressure dissipation rate is shown by 1.5 m drain spacing instead of 1 m drain spacing, because, in the former, the last drain is only 0.5 m away from the point of analysis, while for the latter it is about 1m. Excess pore pressure decreases gradually, and finally it will reach the 50 kPa limit, which is equal to the applied vacuum pressure.
Figure 7.23 Predicted centerline surface measurement for different drain spacing at a construction rate of 0.35m / week

Figure 7.24 Surface settlement at the toe of the embankment for different drain spacing at a construction rate of 0.35m / week
7.5.2 Effect of construction rate upon vacuum application

Two plane strain analyses have been performed to study the effect of construction rate on the stability of embankment subjected to vacuum loading using the same finite element mesh shown earlier in Figure 7.1. Again, the Embankment is assumed to be constructed to a maximum height of 4 m with the same construction rates of 0.35 m/week and 0.5 m/week. The slope of the embankment assumed in the analysis is 1:1 and the drains are installed at 2 m spacing. A constant vacuum of 50 kPa is applied along the top surface beneath the embankment, and along the drain length as discussed in Section 7.5.1. Total time period considered in the analysis is 1 year (365 days).
Figure 7.26 compares the predicted centerline surface settlement for the two different construction rates. As expected, higher construction rates increase the rate of settlement, but the ultimate settlement is the same. Variation of settlement at the embankment toe with the embankment height is presented in Figure 7.27 for the two different construction rates. No heave is predicted, and the embankment can be built to a 4 m height without any indication of failure, although the drain spacing is large and the construction rate is high. This clearly confirms the role of vacuum application that enables increased rate of construction.

Figure 7.28 presents the contours of resultant displacement vectors. Settlement is almost uniform below the embankment and is negligible away from the embankment toe. The pattern of displacement contours suggests that there is no possibility of shear failure within this time period. Figure 7.29 illustrates the pore pressure distribution at two locations across the foundation at 2 m depth. The pore pressure at a point close to the centerline (Figure 7.29a) attains -50 kPa (applied vacuum pressure) after 150 days, while it is dissipated to only about -25 kPa at the same time for a point located below the toe of the embankment.
Figure 7.26 Predicted centerline surface settlement for different construction rates

Figure 7.27 Settlement at the toe of the embankment for different construction rates
7.5.3 Effect of smear upon vacuum application

Plane strain analysis has been performed to investigate the effect of smear upon vacuum application on the stability of soft clay. The finite element mesh, 3 permeability ratios, radius of the smear zone and the permeability coefficients considered in the analysis are the same as used in Section 7.4.1. The Slope of the embankment is 2:1 and the construction rate is 0.35 m/week.
Figure 7.29 Excess pore pressure distribution at 2m depth (a) 0.5m away from the centerline (b) below the embankment toe for different construction rates
The predicted centerline surface settlement based on the 3 cases analysed is illustrated in Figure 7.30. The low permeability ratio shows the highest settlement rate and finally all the three curves end up with their ultimate settlement. Predicted settlement at the toe of the embankment, which is shown in Figure 7.31 indicates that no heave is predicted for the three cases analysed. Embankment can still be constructed to its maximum height with a rapid rate of construction (0.35 m/week), even though the expected soil disturbance is high. Figure 7.32 presents the predicted excess pore pressure distribution at two locations beneath the embankment. As seen by the predicted settlement curves, the excess pore pressure at a point closer to the embankment centerline with a low permeability ratio reaches its ultimate possible value (-50 kPa), which is the applied vacuum pressure) within a shorter period of time. Figure 7.33 displays the contour plots of displacement vectors for the three different cases. Possible shear failure surface is not observed although the pore pressure dissipation is significantly reduced by the presence of smear.
Figure 7.30 Predicted centerline settlement for different permeability ratios upon vacuum application

Figure 7.31 Predicted settlement at embankment toe
Figure 7.32 Excess pore pressure distribution at 2m depth  (a) 0.5m away from the centerline (b) below the embankment toe for different permeability ratios
Figure 7.33 Contour plots of resultant displacement vectors for (a) $k_h/k_s = 2$
(b) $k_h/k_s = 5$ and (c) $k_h/k_s = 5$ after 365 days
7.6 Summary

This chapter described the use of finite element approach in a design perspective, where the role of soil properties, embankment geometry, use of vertical drains, staged construction and application of vacuum pressure have been assessed.

The stability of a typical embankment constructed on soft clay is analysed using a plane strain finite element model. Embankments on soft clays can usually be built to a small height about 2 m without any special measures. However, for higher embankments, special construction techniques are required such as providing reinforcement, staged construction, use of ground improvement technique and providing berms. Staged construction with intermediate rest periods allows embankments to be constructed to considerable heights. Nevertheless, it is a time consuming process, because in some cases, there may be little or no measurable strength gain within a reasonable period of time. The surface crust can also have a significant influence on the height to which an embankment can be constructed, and its presence is analogous to having a geogrid at the base of the embankment.

Introduction of vertical drains (PVD) to a soft clay foundation will enable the increase of the failure height of embankment due to accelerated consolidation of the soil. Also in such cases, the subsequent construction rate can be increased, if the drain spacing is carefully selected. Application of vacuum pressure to PVD installed soft clay can further accelerate the rate of embankment construction without affecting its stability. Soil disturbance due to drain installation can have a significant influence on the height to
which the embankment can be constructed. Unsaturation effect due to vacuum pressure is not considered in the analysis, because it is not significant, when the vacuum pressure is relatively low (60 kPa) as seen in Chapter 6. The possible unsaturation of the soil-drain interface should be considered if the vacuum pressure is high (e.g. close to 100 kPa).
8 CONCLUSIONS AND RECOMMENDATIONS

8.1 General Summary

The review of literature shows that there has always been a discrepancy between the observed and predicted behaviour of embankments stabilised with vertical drains, although significant progress has been made in the recent years through analytical and numerical modeling (Zeng et al., 1987; Hird et al., 1992; Bergado and Long, 1994; Chai et al., 1995; Indraratna and Redana, 1997 among others). This marked discrepancy is attributed to numerous factors such as: uncertainty of soil properties, the effect of smear, and improper conversion of axisymmetric condition to plane strain (2-D) analysis of vertical drains. For construction sites, where many PVDs are installed, 2D plane strain analysis becomes more popular given the computational efficiency.

Consolidation of compressible soils by vacuum pressure is not a new idea, but only until recently it has been successfully applied in a predictable manner to extremely soft soils and in large-scale projects, while controlling the lateral yield of the soft soil. Maintaining a permanent non-submerged medium between the sealing membrane and the soil mass to be consolidated through a specific pumping system and vertical drains, together with the need for properly isolating the boundaries of the soil mass, have been the key factors to the predictable performance of vacuum application. However, the differences between vacuum and surcharge preloading have not been investigated in depth. In the absence of comprehensive and quantitative analysis, the study of suitable
methods to simulate vacuum preloading becomes imperative, both experimentally and numerically.

On the basis of the above, this study was directed to clarify the efficiency of vacuum pressure combined with surcharge preloading, by conducting appropriate laboratory tests using a large-scale consolidometer. The writer discussed a different approach to model vacuum pressure based on the test findings. Hansbo’s (1981) theory for axisymmetric condition and Indraratna and Redana’s (1997) theory for plane strain condition were modified to incorporate vacuum pressure application (Chapter 3). Finally, an attempt was made to investigate the use of the finite element approach in a design perspective, considering various factors (Chapter 7).

In clay soils, a good estimate of the preconsolidation pressure will provide a better estimate of settlement as described in detail by Casagrande (1936). Very often, the values of preconsolidation pressure show considerable scatter when plotted with depth. In stage by stage construction of embankments, it is important to note that the first stage of loading might not exceed the natural preconsolidation pressure of the foundation soil, hence proper values of compression index (\( C_r \) and \( C_c \) or \( \kappa \) and \( \lambda \)) should be selected, which may represent a state of over-consolidation.

Accurate prediction of lateral displacements depends on the correct value of \( \lambda \), the nature of assumptions made in the numerical modeling of drains, and the ways of simulating the vacuum pressure. Modelling of vacuum pressure as an additional
surcharge pressure will predict definite ‘outward’ lateral movement. But in reality, under vacuum pressure, the movement could be ‘inward’. The inclusion of the correct distribution of vacuum pressure along the drain depth, is proven to predict the lateral displacement correctly, as demonstrated in this study.

The surface crust has higher undrained strength than would be associated with lightly overconsolidated soil. Moreover, the stiffness of the crust can be greater than that of the soil below it. This strength and stiffness and their distribution with depth can have a significant effect on the stability and deformation of soft the soil foundation in question. Further accuracy of numerical predictions may be achieved by using improved procedures for accurate modeling of the surficial over-consolidated crust that does not strictly obey the modified Cam-clay theory.

The use of an equivalent plane strain model for multi drain analysis cannot be considered superior to a more sophisticated three-dimensional analysis of flow towards the vertical drains. However, the plane strain predictions are still acceptable in terms of both the accuracy and the computational efficiency. Multi-drain analysis is important especially for wider embankments compared to its height, because, the settlement pattern could deviate from a conventional symmetrical embankment (maximum settlement at the center) perfectly following plane strain.
8.2 Specific Observations

In this study, an equivalent 2-D plane strain analysis of embankments stabilised with vertical drains subjected to vacuum pressure application has been carried out, incorporating the effects of smear. Well resistance has been ignored in most cases because, the previous studies have shown that the effect of well resistance is negligible, if the drain depth is less than say 20m. The possible unsaturation at the drain-soil boundary due to rapid vacuum application is also considered. The sudden withdrawal of mandrel can also induce unsaturation at the soil-drain interface. The efficiency of the combined surcharge and vacuum preloading was studied in the laboratory using a large-scale consolidometer apparatus. The behaviour of 2 embankments stabilised with vertical drains and subjected to vacuum preloading was analysed in detail. The predictions based on the proposed 2-D plane strain model incorporating smear and unsaturation of drain-soil boundary were compared to the laboratory observations and field measurements. The proposed 2-D plane strain model was then employed to predict the stability of embankments built on soft clay under different conditions. Specific conclusions, which can be drawn on the basis of this study, are presented below.

8.2.1 Modification to the existing theories to incorporate vacuum application

1. The numerical results verified that the combined vacuum and surcharge preloading is in effect, follows the principle of a superposition.

2. Whether preloading is applied as a vacuum pressure or surcharge pressure, the form of the governing equation for analyzing the average excess pore pressure
dissipation is the same. However, the combination of vacuum and surcharge pressure further accelerates the rate of pore pressure dissipation due to the increased hydraulic gradient towards the PVD.

3. The analytical and numerical results further conclude that the well resistance of a drain with a relatively high discharge capacity is not significant.

### 8.2.2 Laboratory Program

1. The laboratory tests utilizing the large-scale consolidometer revealed that the accelerated consolidation due to vacuum preloading is more effective in comparison with the conventional multi-stage loading. The application of vacuum pressure increases the rate of pore pressure dissipation due to the increased pore pressure gradient towards the drain. Using a traditional earthfill surcharge load combined with vacuum pressure will shorten the duration of preloading, especially in soft clays with low shear strength. However, the cost implications of vacuum preloading in large projects, in relation to the conventional gravity preloading, should be taken into account. With the use of sufficient vacuum pressure with properly sealed surface membranes will significantly accelerate pre-construction settlement, thereby compensating for the initial capital costs by enhancing speed of construction and minimising post-construction settlement.

2. The efficiency of distribution of vacuum pressure along the drain length depends on the sealing (i.e. no air leaks) and the type of soil around the drain. The readings of the transducer located close to the drain bottom indicated that the
suction head decreases with the drain depth, as the maximum suction of 100 kPa could not be maintained along the entire drain length. Maximum measured vacuum pressure was approximately 80 kPa at this depth, indicating about 20% head loss over a length of about 1 m. The release of vacuum pressure readily decreases the suction, and the reapplication of vacuum pressure increases the suction rapidly again. The effect of fluctuation of vacuum pressure (e.g. removal and re-application) is clearly reflected by the corresponding change of gradient of the settlement plot.

8.2.3 Numerical modeling of vertical drain subjected to vacuum pressure

1. In terms of predicting settlement, modeling of vacuum pressure as an additional surcharge pressure or distributing negative pore pressures along the drain boundary are both acceptable. However, for correct pore pressure predictions, modeling of vacuum pressure by correctly distributing the negative pore pressure along the drain boundary as well as at the top soil surface is essential.

2. The effectiveness of vacuum preloading is a function of the soil permeability. The efficiency of vacuum preloading is reduced by soil unsaturation at the soil-PVD interface.

3. There may be the possibility of soil unsaturation also due to rapidly applied vacuum pressure. The change in saturation varies upon the initial saturated permeability, soil moisture characteristic curve (soil type) and the magnitude of applied vacuum pressure. For realistic predictions, the use of appropriate soil moisture characteristic curve, correctly evaluated saturated permeability
coefficient and the estimation of actual distribution of vacuum pressure (both laterally and with depth) are highly desirable.

4. In large-scale laboratory testing, the application of 100 kPa vacuum pressure over the soil surface and through a relatively short PVD demonstrated that vacuum preloading was definitely beneficial. Nevertheless, the occurrence of soil unsaturation at the PVD boundary would retard the pore pressure dissipation, as clearly reflected by the corresponding difference in settlement curves. The presence of even a thin unsaturated soil layer at the PVD boundary could retard the efficiency of vacuum application. Conventional saturated soil models overestimate settlement, while the inclusion of a thin layer of unsaturated soil elements at the PVD boundary improves the accuracy of numerical predictions.

8.2.4 Case history analysis

1. The application of the proposed equivalent plane strain model incorporating the smear effect and soil unsaturation provides a convenient alternative to a complex 3-D analysis involving a large number of axisymmetric drains. The capability of the equivalent model was verified by acceptable agreement between the analysis and the field data, with respect to two embankments reported in this study. The 2-D plane strain multi-drain analysis proved that the inclusion of a time and depth dependent vacuum pressure distribution improves the settlement and lateral displacement predictions significantly. The accurate prediction of pore
water pressure is still difficult, even though some improvement from pressure analyses could be seen.

2. Unlike in the case of conventional surcharge loading, application of vacuum pressure generates ‘inward’ lateral displacement rather than ‘outward’. Due to this inward movement, tensile cracks may develop adjacent to the stabilised area. This aspect was not modeled within the scope of this study.

3. The exact value of compression index ($C_c$) or gradient $\lambda$ in Cam-clay model is sensitive to the loading stages (fill heights). The values of $\lambda$ should be associated with the applied working stress range in relation to the effective preconsolidation pressure of the soil at a particular depth. The realistic value of $\lambda$ should be based on appropriate stress paths as pointed out by Indraratna et al. (1997) (Figure 8.1), depending on whether the applied load is much less or considerably greater than the pre-consolidation pressure of the clay.

### 8.2.5 Application of Finite element modeling for general design

1. The height of embankment, which can be raised on soft clay without any stability problems, depends on the embankment geometry, subsoil properties and the method of construction.

2. The presence of surface crust (overconsolidated) beneath the embankment can resist lateral displacement, thereby significantly increasing the height to which the embankment can be constructed. The presence of a surface crust has a similar influence as placing geogrid at the base of embankment.
3. Installation of PVD in soft clay beneath the embankment can increase the failure height of embankment and rate of construction. Introduction of PVD for subsurface drainage can provide increased stiffness of the soft clay and curtail the lateral displacement substantially, thereby minimizing the risk of shear failure. However, the drain spacing and the propagation of smear effect can have significant influence on the final height of embankment. Indraratna et al. (2002) demonstrated that the stiffness of vertical drains is of secondary importance in comparison with the need for appropriate spacing for controlling...
lateral deformation. Even though the ultimate settlement of the soil is not changed by the pattern of PVD, the spacing and length of PVD can still affect the rate of embankment construction and its final height. The drain installation method should be carefully implemented to reduce the smear effect as much as possible.

4. Application of vacuum pressure to PVD installed soft clay (prior to construction) can further accelerate the rate of construction without affecting stability. Inward movement of soil due to suction will also reduce the risk of bearing capacity failure. Therefore, PVD used in conjunction with vacuum preloading is more beneficial than conventional surcharge loading.

8.3 Recommendations for Future Work

Further analytical, numerical and experimental studies associated with embankments stabilised with vertical drains subjected to vacuum preloading are recommended. Future work should focus more on the following aspects:

1. Detailed stress path testing of typical NSW soft foundation soils, in order to obtain the most accurate soil properties, and to quantify in a rational manner the role of applied stresses in relation to the pre-consolidation pressure of the foundation soils.

2. It is important to evaluate the coefficient of saturated permeability of the soil as accurately as possible, based on large soil samples. Very often, the coefficient of permeability obtained from conventional oedometer tests was not accurate, in
contrast to the field permeability data (when available). It is recommended that a more realistic permeability coefficient is obtained through back analysis of tests conducted in the large scale consolidometer using 300 mm diameter samples, even though some practical difficulties may arise due to the need for handling large specimens.

3. The numerical model should also incorporate the role of both axial and lateral stiffness of vertical drains where warranted, in order to obtain better predictions of settlements and lateral displacements in the soft clay foundation. Especially, in plane strain models where a continuous vertical drain ‘wall’ is assumed, the increased stiffness of this drain ‘wall’ may become higher than the soft clay (of same wall thickness), hence the need for modeling the correct stiffness should also be considered.

4. More laboratory tests should be carried out to study the behaviour of soft clay under different combinations of vacuum and surcharge pressures and using PVD of different properties and soils of varying compressibility and undrained strength. For each soil type, moisture characteristic curve should be developed and use in the proposed finite element model, at the soil-drain interface.

5. Further research with selected ‘instrumented PVD’ in the field is desirable as it may provide further insight to the vacuum pressure distribution development with depth.

6. Anisotropic soil properties have a significant effect on Embankment behaviour (Potts and Zdravković, 2000). To predict the embankment failure more accurately, use of complex constitutive models such as MIT-E3 (Whittle, 1991) incorporating anisotropy observed in laboratory and field is recommended.
7. Soil constitutive models (e.g., Cam-clay) in ABAQUS should be further extended to be more realistic under conditions of soil unsaturation. For this purpose, a separate subroutine incorporating the moisture characteristic curves may be developed to directly link the laboratory data and then used at the drain-soil interface.

8. In order to detect the failure of embankment constructed on soft clay, it is recommended to carry out a parametric study on the assumed plane strain model. Various parameters such as the construction rate, embankment slope, and drain spacing etc. should be taken into account, and finally, a numerical scheme may be developed to estimate the maximum fill height that will be of paramount importance to the designer.
REFERENCES


Indraratna, B., Bamunawita, C., Redana, I W., McIntosh, G., (2002). keynote paper: Modelling of prefabricated vertical drains in soft clay and evaluation of their
effectiveness in practice, Proc. 4th Int. Conf. on Ground Improvement Techniques. Malaysia, pp 47-62


APPENDIX A

COMPUTER PROGRAMME TO CONVERT VOLTAGE VALUES TO APPROPRIATE UNITS

*******************************************************************
RESET
CLEAR
D=\D
T=\T
S1=100,400,3.95,14.88
S2=100,400,4.35,15.35
S3=100,400,4.48,15.46
S4=100,400,3.45,14.3
S5=100,400,4.13,15.03
S6=100,400,3.43,14.34
S7=20,94,850,3260
RA1S1V(S1,"T1kPa=)2V(S2,"T2kPa=)3V(S3,"T3kPa=)4V(S4,"T4kPa=)
5V(S5,"T5kPa=)6V(S6,"T6kPa=)7V(ST,"T7mm=)
***********************************************************************

Description of some terms in the programme
CLEAR- clear all previous data
D=\D set the date
T=\T set the time

S1=100,400,3.95,14.88
3.95 is voltage output, when the load is 100 kPa
14.88 is voltage output, when the load is 400 kPa

S7=20,94,850,3260
20 is the load in mm
94 is the voltage output in mV
850 is the load in mm
3260 is the voltage output in mV

RA1S1V
1S represents the time interval at which the readings should be taken. In this case it is only a one second.
APPENDIX B
MODIFIED CAM-CLAY PARAMETERS

B.1 Gradients $\lambda$ and $\kappa$

In order to obtain the modified Cam-clay parameters, the results from the standard
oedometer test are plotted in Figure. B.1. The Schmertmann (1955) graphical procedure
is applied to correct the laboratory virgin compression curve, which is extend to 0.42 $e_o$.
The compression index, $C_c$ and recompression index $C_r$ are obtained using the following
expressions:

$$C_c = \frac{e_1 - e_2}{\log \frac{\sigma_i}{\sigma_i'}} = 0.34$$  \hspace{1cm} \text{(B-1)}

where $\sigma_i'$ and $\sigma'_2$ are the effective stresses on the consolidation line

$$C_r = \frac{e_1 - e_2}{\log \frac{\sigma'_i}{\sigma'_i}} = 0.14$$  \hspace{1cm} \text{(B-2)}

where $\sigma'_i$ and $\sigma'_2$ are the effective stresses on the swelling line

The Cam-clay parameters are estimated using Equation (2.55) as follows:

$$\lambda = \frac{C_c}{2.307} = 0.147$$ and

$$\kappa = \frac{C_r}{2.307} = 0.05$$
Figure B.1. Determination of compression properties of soil using standard consolidometer test.

B.2 Slope of Critical State Line (CSL), $M$

The slope of the critical state line, $M$ in $V-p'$ plot is obtained using Equation (2.58) as follows: 

$$M = \frac{6 \sin \phi'}{3 - \sin \phi'} = 1.02 \text{ for } \phi = 26^\circ.$$ 

B.3 Initial Void Ratio, $e_{cs}$

The initial void ratio, $e_{cs}$ can be estimated using Equations 2.59 and 2.60 (Chapter 2). The soil was initially loaded of about 20 kPa prior to consolidation stages as explained in Chapter 4. At depth 0.45, the total vertical stress, pore water pressure and effective stress are estimated to be: $\sigma_v = 28 kPa$, $u = 4.5 kPa$, and $\sigma'_v = 23.5 kPa$, respectively. The coefficient of earth pressure at rest, $K_{nc}$ which is required to estimate effective vertical stress is determined using the following expression:
\( K_{nc} = 1 - \sin \phi' = 0.531 \), where \( \phi' \) is 26°.

The effective vertical stress is estimated as follows:
\[
\sigma_h' = K_{nc} \sigma_v' = 12.5 \text{kPa}
\]

The critical state coordinate are determined as follows:
\[
q = \sigma_v' - \sigma_h' = 11 \text{kPa}
\]
\[
p' = \left( \sigma_v' + 2\sigma_h' \right)/3 = 16.2 \text{kPa}
\]

The modified Cam-clay yield locus is given by:
\[
P'_c = q^2/M^2 + p' = 22.4 \text{kPa}
\]

The intersection between CSL and swelling line is estimated at:
\[
P_A' = \frac{P_c'}{2} = 11.2 \text{kPa}
\]

Following Equation (2.61), the \( e_{cs} \) is estimated to be:
\[
e_{cs} = e + (\lambda + \kappa) \ln P_A' - \kappa \ln p' = 1.54
\]
APPENDIX C

PARAMETRIC INPUT FILE

*HEADING
PARAMETRIC GEOMETRY INPUT WITH SMEAR
**-----UNITS: LENGTH=m,FORCE=N,TIME=s,MASS=kg-------
**********************************************************************
parameter description: (can be changed under parameter)
rs = radius of smeared zone
R  = radius of influence
H  = drain height
S  = drain spacing
W  = half of the embankment width
L  = total foundation width considered
H1, H2, H3, H4, H5 = height to each clay layer from bottom of
the drain
A,B,C,D,E,F,G,A1,A2,A3,A4 = points as below
a_bels = number of elements between A and B
b_cels = number of elements between B and C
c_dels = number of elements between C and D
d_eels = number of elements between D and E
e_fels = number of elements between E and F
f_gels = number of elements between F and G
a_a1els = number of elements between A and A1
a1_a2els = number of elements between A1 and A2
a3_a4els = number of elements between A3 and A4
a_bbias= bias value between a and b when using NFILL
b_cbias= bias value between b and c when using NFILL
c_dbias= bias value between c and d when using NFILL
d_ebias= bias value between d and e when using NFILL
e_fbias= bias value between e and f when using NFILL
f_gbias= bias value between f and g when using NFILL
a_a1bias= bias value between a and a1 when using NFILL
a1_a2bias= bias value between a1 and a2 when using NFILL
a3_a4bias= bias value between a3 and a4 when using NFILL

G--G1----G2..............................................G3---------------------G4
F--F1----F2..............................................F3---------------------F4
E--E1----E2..............................................E3---------------------E4
D--D1----D2..............................................D3---------------------D4
C--C1----C2..............................................C3---------------------C4
B--B1----B2..............................................B3---------------------B4
A--A1----A2..............................................A3---------------------A4
********************************************************************
**depending on the drain spacing some changes may be necessary in
NCOPY and EL COPY options
********************************************************************
**ASSIGN VALUES TO PARAMETERS**

*PREPRINT, MODEL=YES

*PARAMETER

S=1.0
RS=0.25
R=0.5
W=20
L=60
H1=2.0
H2=4.5
H3=6.5
H4=13.0
H5=14.5
H=15.
a_bels=1
b_cels=2
c_dels=2
d_eels=10
e_fels=3
f_gels=1
a_alels=1
a1_a2els=1
a3_a4els=10
a_bbias=1.0
b_cbias=1.0
c_dbias=1.0
d_ebias=1.0
e_fbias=1.0
f_gbias=1.0
a1_abbias=1.0
a1_a2bias=1.0
a3_a4bias=0.7

**the following parameters are dependent on those above**

S2=2*S
S3=3*S
S4=4*S
S5=5*S
S6=6*S
S7=7*S
S8=8*S
S9=9*S
S10=10*S
S11=11*S
S12=12*S
S13=13*S
S14=14*S
S15=15*S
S16=16*S
S17=17*S
S18=18*S
S19=19*S
S20=20*S
R1=600*a3_a4els
R2=R-RS
R3=R+(R-RS)

**ELEMENT NUMBERS**

**INCREASE BY 2 UP AND 600 TO THE RIGHT**

aer=1
bed=aer+2*(a_bels-1)
bet=bed+2
ced=bed+2*(b_cels-1)
cet=ced+2
ded=cet+2*(c_dels-1)
det=ded+2
eed=det+2*(d_eels-1)
eet=eed+2
fed=eet+2*(e_fels-1)
ket=fed+2
gel=fet+2*(f_gels-1)
aer=gel+600*(a_alels)
b1er=gel+600*(a_alels)
c1er=gel+600*(a_alels)
d1er=gel+600*(a_alels)
e1er=gel+600*(a_alels)
f1er=gel+600*(a_alels)
g1er=gel+600*(a_alels)

R5=600*(2*a_alels+2*a1_a2els)
R6=2*R5
R7=3*R5
R8=4*R5
R9=5*R5
R10=6*R5
R11=7*R5
R12=8*R5
R13=9*R5
R14=10*R5
R15=11*R5
R16=12*R5
R17=13*R5
R18=14*R5
R19=15*R5
R20=16*R5
R21=17*R5
R22=18*R5
R23=19*R5
R24=20*R5
R25=R23
R26=600*a1_a2els
R4=R23+(a_alels+2*a1_a2els)*600
R30=600*(a_alels+2*a1_a2els)

**INCREASE BY 2 UP AND 600 TO THE RIGHT**

aer=1
bed=aer+2*(a_bels-1)
bet=bed+2
ced=bed+2*(b_cels-1)
cet=ced+2
ded=cet+2*(c_dels-1)
det=ded+2
eed=det+2*(d_eels-1)
eet=eed+2
fed=eet+2*(e_fels-1)
ket=fed+2
gel=fet+2*(f_gels-1)
a1er=gel+600*(a_alels)
b1er=gel+600*(a_alels)
c1er=gel+600*(a_alels)
d1er=gel+600*(a_alels)
e1er=gel+600*(a_alels)
f1er=gel+600*(a_alels)
g1er=gel+600*(a_alels)
g3el5=g3el4-600

... (repeated 25 times)...

g3el46=g3el45-600

... (repeated 25 times)...

g4el=g3el4+600*(a3_a4eels-1)

*******************************************************************************

**NODE NUMBERS

**INCREASE BY 2 UP AND 600 TO THE RIGHT

*******************************************************************************

an=1
ant=an+1
an2=an+2
an3=an+4
an4=an+6
an5=an+8
an6=an+10
an7=an+12
an8=an+14
an9=an+16
an10=an+18
an11=an+20
a2n = a1n + 600 * a1_a2els
a2nt = a2n + 1
a2n2 = a2n + 2
b2n = a2n + 2 * (a_bels)
b2nd = b2n - 1
b2nt = b2n + 1
b2n2 = b2n - 2
c2n = b2n + 2 * (b_cels)
c2nd = c2n - 1
c2nt = c2n + 1
c2n2 = c2n - 2
d2n = c2n + 2 * (c_dels)
d2nd = d2n - 1
d2nt = d2n + 1
d2n2 = d2n - 2
e2n = d2n + 2 * (d_eels)
e2nt = e2n + 1
e2n2 = e2n - 1
f2n = e2n + 2 * (e_fels)
f2nd = f2n - 2
f2nt = f2n + 1
g2n = f2n + 2 * (f_gels)
g2nd = g2n - 2
a3n = an + R24
a3nt = a3n + 1
a3n2 = a3n + 2
b3n = a3n + 2 * (a_bels)
b3nd = b3n - 1
b3nt = b3n + 1
b3n2 = b3n - 2
c3n = b3n + 2 * (b_cels)
c3nd = c3n - 1
c3nt = c3n + 1
c3n2 = c3n - 2
d3n = c3n + 2 * (c_dels)
d3nd = d3n - 1
d3nt = d3n + 1
d3n2 = d3n - 2
e3n = d3n + 2 * (d_eels)
e3nt = e3n + 1
e3n2 = e3n - 2
f3n = e3n + 2 * (e_fels)
f3nd = f3n - 2
f3nt = f3n + 1
f3n2 = f3n - 1
f3nt = f3n + 1
g3n = f3n + 2 * (f_gels)
g3n2 = g3n - 2
g3nd = g3n - 1
g3nt = g3n + 600

**-------------------------------
a4n = a3n + 600 * a3_a4els
a4nt = a4n + 1
a4n2 = a4n + 2
b4n = a4n + 2 * (a_bels)
b4nd = b4n - 1
b4nt = b4n + 1

b4n2 = b4n - 2
c4n = b4n + 2 * (b_cels)
c4nt = c4n + 1
c4n2 = c4n - 2
d4n = c4n + 2 * (c_dels)
d4nt = d4n + 1
d4n2 = d4n - 2
e4n = d4n + 2 * (d_eels)
e4nt = e4n + 1
e4n2 = e4n - 2
e4nd = e4n - 1
f4n = e4n + 2 * (e_fels)
f4nt = f4n + 1
f4n2 = f4n - 2
f4nd = f4n - 1
g4n = f4n + 2 * (f_gels)
g4nt = g4n + 1
g4n2 = g4n - 2
g4nd = g4n - 1

******************************************************************************
**NODES ALONG THE LAST DRAIN BOUNDARY
**R25 WILL VARY WITH DRAIN SPACING
******************************************************************************
a6n = an + R25
a6nt = a6n + 1
a6n2 = a6n + 2
a6n3 = a6n + 3
b6n = a6n + 3 * (a_bels)
b6nd = b6n - 1
b6nt = b6n + 1
b6n2 = b6n - 2
c6n = b6n + 3 * (b_cels)
c6nd = c6n - 1
c6nt = c6n + 1
c6n2 = c6n - 2
d6n = c6n + 3 * (c_dels)
d6nd = d6n - 1
d6nt = d6n + 1
d6n2 = d6n - 2
e6n = d6n + 3 * (d_eels)
e6nt = e6n + 1
e6n2 = e6n - 1
f6n = e6n + 3 * (e_fels)
f6nd = f6n - 2
f6nt = f6n + 1
f6n2 = f6n - 1
f6nt = f6n + 1
g6n = f6n + 3 * (f_gels)
g6nt = g6n + 1
g6n2 = g6n - 2

******************************************************************************
**NODES OF EACH MASTER ELEMENT
******************************************************************************
aer1 = an
aer2 = an + 600
aer3 = aer2 + 2
aer4 = aer1 + 2
aer5 = an + 300
aer6 = aer2 + 1
aer7 = aer5 + 2
aer8 = aer1 + 1
bet1=bn
cet1=cn
det1=dn
eet1=en
fet1=fn
aer1=an
b1er1=bn
a1er1=an
cler1=cn
cler2=cn+600
cler3=cler2+2
cler4=cler2+2
cler5=cler2+1
cler6=cler2+2
cler7=cler2+2
cler8=cler2+1
dler1=dn+300
dler2=dn+600
dler3=dlr2+2
dler4=dlr1+2
dler5=dlr2+1
dler6=dlr2+1
dler7=dlr5+2
dler8=dlr1+1
e1er1=eln
e1er2=elr+600
e1er3=elr1+2
e1er4=elr1+2
e1er5=elr+300
e1er6=elr2+1
f1er1=f1n
e1er7=elr5+2
f1er2=f1n+600
f1er3=f1r2+2
f1er4=f1r1+2
f1er5=f1r+300
f1er6=f1r2+1
f1er7=f1r5+2
f1er8=f1r1+1
**NODE DEFINITION**

**ASSIGNING COORDINATES TO THE NODES**

**ASSIGNING NAMES TO EACH NODE**

*NSET, NSET = A
<an>,

*NSET, NSET = B
<brn>,

*NSET, NSET = C
<cn>,

*NSET, NSET = D
<dn>,

*NSET, NSET = E
<en>,

*NSET, NSET = F
<fn>,

*NSET, NSET = G
<gn>,

*NSET, NSET = A1
<a1n>,

*NSET, NSET = B1
<b1n>,

*NSET, NSET = C1
<c1n>,

*NSET, NSET = D1
<d1n>,

*NSET, NSET = E1
<e1n>,

*NSET, NSET = F1
<f1n>,

*NSET, NSET = G1
<g1n>,

*NSET, NSET = A2
<a2n>,

*NSET, NSET = B2
<b2n>,

*NSET, NSET = C2
<c2n>,

*NSET, NSET = D2
<d2n>,

*NSET, NSET = E2
<e2n>,

*NSET, NSET = F2
<f2n>,

*NSET, NSET = G2
<g2n>,

*NSET, NSET = A3
<a3n>,

*a_a1nod = 2*a_a1els
*a1_a2nod = 2*a1_a2els
*a3_a4nod = 2*a3_a4els

********************************

**ASSIGNING NAMES TO EACH NODE**

********************************

*a_bnod = 2*a_bels
*a_bdnod = a_bnod - 1
*b_cnod = 2*b_cels
*b_cdnod = b_cnod - 1
*c_dnod = 2*c_dels
*c_ddnod = c_dnod - 1
*d_enod = 2*d_eels
*d_ednod = d_enod - 1
*e_fnod = 2*e_fels
*e_fdnod = e_fnod - 1
*f_gnod = 2*f_gels
*f_gdnod = f_gnod - 1
*a_alnod = 2*a_a1els
*a1_a2nod = 2*a1_a2els
*a3_a4nod = 2*a3_a4els

********************************

**ASSIGNING DEFINITION**

**ASSIGNING COORDINATES TO THE NODES**

**NODE**

<an>, 0., 0.
<brn>, 0., <H1>
<cn>, 0., <H2>
<dn>, 0., <H3>
<en>, 0., <H4>
<fn>, 0., <H5>
<gn>, 0., <H>
<a1n>, <RS>, 0.
<br1n>, <RS>, <H1>
<cn1>, <RS>, <H2>
<dn1>, <RS>, <H3>
<en1>, <RS>, <H4>
<fn1>, <RS>, <H5>
<gn1>, <RS>, <H>
<a2n>, <R>, 0.
<br2n>, <R>, <H1>
<cn2>, <R>, <H2>
<dn2>, <R>, <H3>
<en2>, <R>, <H4>
<fn2>, <R>, <H5>
<gn2>, <R>, <H>
<a3n>, <W>, 0.
**GENERATING INTERMEDIATE NODES BETWEEN NODES DEFINE EARLIER**

*NFILL,NSET=A_B,BIAS=<a_bbias>,TWO STEP
A,B,<a_bnod>,1
*NFILL,NSET=B_C,BIAS=<b_cbias>,TWO STEP
  B,C,<b_cnod>,1
*NFILL,NSET=C_D,BIAS=<c_dbias>,TWO STEP
  C,D,<c_dnod>,1
*NFILL,NSET=D_E,BIAS=<d_ebias>,TWO STEP
  D,E,<d_enod>,1
*NFILL,NSET=E_F,BIAS=<e_fbias>,TWO STEP
  E,F,<e_fnod>,1
*NFILL,NSET=F_G,BIAS=<f_gbias>,TWO STEP
  F,G,<f_gnod>,1
*NFILL,NSET=A1_B1,BIAS=<a_bbias>,TWO STEP
A1,B1,<a_bnod>,1
*NFILL,NSET=B1_C1,BIAS=<b_cbias>,TWO STEP
B1,C1,<b_cnod>,1
*NFILL,NSET=C1_D1,BIAS=<c_dbias>,TWO STEP
C1,D1,<c_dnod>,1
*NFILL,NSET=D1_E1,BIAS=<d_ebias>,TWO STEP
D1,E1,<d_enod>,1
*NFILL,NSET=E1_F1,BIAS=<e_fbias>,TWO STEP
E1,F1,<e_fnod>,1
*NFILL,NSET=F1_G1,BIAS=<f_gbias>,TWO STEP
F1,G1,<f_gnod>,1
*NFILL,NSET=A2_B2,BIAS=<a_bbias>,TWO STEP
A2,B2,<a_bnod>,1
*NFILL,NSET=B2_C2,BIAS=<b_cbias>,TWO STEP
B2,C2,<b_cnod>,1
*NFILL,NSET=C2_D2,BIAS=<c_dbias>,TWO STEP
C2,D2,<c_dnod>,1
*NFILL,NSET=D2_E2,BIAS=<d_ebias>,TWO STEP
D2,E2,<d_enod>,1
*NFILL,NSET=E2_F2,BIAS=<e_fbias>,TWO STEP
E2,F2,<e_fnod>,1
*NFILL,NSET=F2_G2,BIAS=<f_gbias>,TWO STEP
F2,G2,<f_gnod>,1
*NFILL,NSET=A3_B3,BIAS=<a_bbias>,TWO STEP
A3,B3,<a_bnod>,1
*NFILL,NSET=B3_C3,BIAS=<b_cbias>,TWO STEP
B3,C3,<b_cnod>,1
*NFILL,NSET=C3_D3,BIAS=<c_dbias>,TWO STEP
C3,D3,<c_dnod>,1
*NFILL,NSET=D3_E3,BIAS=<d_ebias>,TWO STEP
D3,E3,<d_enod>,1
**GENERATING INTERNAL NODES IN HALF UNIT CELL**

*NFILL, NSET=E3_F3, BIAS=<e_fbias>, TWO STEP
E3, F3, <e_fnod>, 1

*NFILL, NSET=F3_G3, BIAS=<f_gbias>, TWO STEP
F3, G3, <f_gnod>, 1

*NFILL, NSET=A4_B4, BIAS=<a_bbias>, TWO STEP
A4, B4, <a_bnod>, 1

*NFILL, NSET=B4_C4, BIAS=<b_cbias>, TWO STEP
B4, C4, <b_cnod>, 1

*NFILL, NSET=C4_D4, BIAS=<c_dbias>, TWO STEP
C4, D4, <c_dnod>, 1

*NFILL, NSET=D4_E4, BIAS=<d_ebias>, TWO STEP
D4, E4, <d_enod>, 1

*NFILL, NSET=E4_F4, BIAS=<e_fbias>, TWO STEP
E4, F4, <e_fnod>, 1

**GENERATING INTERNAL NODES AWAY FROM THE EMBANKMENT**

*NFILL, BIAS=<a1_a2bias>, TWO STEP
A1_B1, A2_B2, <a1_a2nod>, 300
B1_C1, B2_C2, <a1_a2nod>, 300
C1_D1, C2_D2, <a1_a2nod>, 300
D1_E1, D2_E2, <a1_a2nod>, 300
E1_F1, E2_F2, <a1_a2nod>, 300
F1_G1, F2_G2, <a1_a2nod>, 300

**GENERATING NODE SETS**

*NSET, NSET=N1, GENERATE
<an>, <bnd>, 1

*NSET, NSET=N2, GENERATE
<bn>, <cnd>, 1

*NSET, NSET=N3, GENERATE
<cn>, <dnd>, 1

*NSET, NSET=N4, GENERATE
<dn>, <enod>, 1

*NSET, NSET=N5, GENERATE
<en>, <fnd>, 1

*NSET, NSET=N6, GENERATE
<aln>, <blnd>, 1

*NSET, NSET=N7, GENERATE
<b1n>, <c1nd>, 1
**GENERATING NODE SETS IN EACH SOIL LAYER FOR HALF UNIT CELL**

*NFILL,BIAS=<a1_a2bias>,TWO STEP,NSET=N_1
N6,N11,<a1_a2nod>,300
*NFILL,BIAS=<a1_a2bias>,TWO STEP,NSET=N_2
N7,N12,<a1_a2nod>,300
*NFILL,BIAS=<a1_a2bias>,TWO STEP,NSET=N_3
N8,N13,<a1_a2nod>,300
*NFILL,BIAS=<a1_a2bias>,TWO STEP,NSET=N_4
N9,N14,<a1_a2nod>,300
*NFILL,BIAS=<a1_a2bias>,TWO STEP,NSET=N_5

**GENERATING NODE SETS IN EACH SOIL LAYER FOR HALF UNIT CELL**

*NSET,NSET=N8,GENERATE
<c1n>,<d1nd>,1
*NSET,NSET=N9,GENERATE
<d1n>,<e1nd>,1
*NSET,NSET=N10,GENERATE
<e1n>,<f1nd>,1
*NSET,NSET=N11,GENERATE
<a2n>,<b2nd>,1
*NSET,NSET=N12,GENERATE
<b2n>,<c2nd>,1
*NSET,NSET=N13,GENERATE
<c2n>,<d2nd>,1
*NSET,NSET=N14,GENERATE
<d2n>,<e2nd>,1
*NSET,NSET=N15,GENERATE
<e2n>,<f2nd>,1
*NSET,NSET=N16,GENERATE
<a3n>,<b3nd>,1
*NSET,NSET=N17,GENERATE
<b3n>,<c3nd>,1
*NSET,NSET=N18,GENERATE
<c3n>,<d3nd>,1
*NSET,NSET=N19,GENERATE
<d3n>,<e3nd>,1
*NSET,NSET=N20,GENERATE
<e3n>,<f3nd>,1
*NSET,NSET=N21,GENERATE
<a4n>,<b4nd>,1
*NSET,NSET=N22,GENERATE
<b4n>,<c4nd>,1
*NSET,NSET=N23,GENERATE
<c4n>,<d4nd>,1
*NSET,NSET=N24,GENERATE
<d4n>,<e4nd>,1
*NSET,NSET=N25,GENERATE
<e4n>,<f4nd>,1

******************************************************************
**GENERATING NODE SETS IN EACH SOIL LAYER FOR HALF UNIT CELL**
******************************************************************

*NSET,NSET=N8,GENERATE
<c1n>,<d1nd>,1
*NSET,NSET=N9,GENERATE
<d1n>,<e1nd>,1
*NSET,NSET=N10,GENERATE
<e1n>,<f1nd>,1
*NSET,NSET=N11,GENERATE
<a2n>,<b2nd>,1
*NSET,NSET=N12,GENERATE
<b2n>,<c2nd>,1
*NSET,NSET=N13,GENERATE
<c2n>,<d2nd>,1
*NSET,NSET=N14,GENERATE
<d2n>,<e2nd>,1
*NSET,NSET=N15,GENERATE
<e2n>,<f2nd>,1
*NSET,NSET=N16,GENERATE
<a3n>,<b3nd>,1
*NSET,NSET=N17,GENERATE
<b3n>,<c3nd>,1
*NSET,NSET=N18,GENERATE
<c3n>,<d3nd>,1
*NSET,NSET=N19,GENERATE
<d3n>,<e3nd>,1
*NSET,NSET=N20,GENERATE
<e3n>,<f3nd>,1
*NSET,NSET=N21,GENERATE
<a4n>,<b4nd>,1
*NSET,NSET=N22,GENERATE
<b4n>,<c4nd>,1
*NSET,NSET=N23,GENERATE
<c4n>,<d4nd>,1
*NSET,NSET=N24,GENERATE
<d4n>,<e4nd>,1
*NSET,NSET=N25,GENERATE
<e4n>,<f4nd>,1
N10, N15, <a1_a2nod>, 300
*NFILL, BIAS=<a1_a2bias>, TWO_STEP, NSET=N_6
F1_G1, F2_G2, <a1_a2nod>, 300

**GENERATING NODE SETS IN EACH SOIL LAYER AWAY FROM THE EMBANKMENT
*******************************************************************************
*NFILL, BIAS=<a3_a4bias>, TWO_STEP, NSET=N_SOIL1
N16, N21, <a3_a4nod>, 300
*NFILL, BIAS=<a3_a4bias>, TWO_STEP, NSET=N_SOIL2
N17, N22, <a3_a4nod>, 300
*NFILL, BIAS=<a3_a4bias>, TWO_STEP, NSET=N_SOIL3
N18, N23, <a3_a4nod>, 300
*NFILL, BIAS=<a3_a4bias>, TWO_STEP, NSET=N_SOIL4
N19, N24, <a3_a4nod>, 300
*NFILL, BIAS=<a3_a4bias>, TWO_STEP, NSET=N_SOIL5
N20, N25, <a3_a4nod>, 300
*NFILL, BIAS=<a3_a4bias>, TWO_STEP, NSET=N_SOIL6
F3_G3, F4_G4, <a3_a4nod>, 300

******************************************************************************
**GENERATING NODE SETS ALONG SIDE BOUNDARIES
******************************************************************************
*NSET, NSET=N_LHS, GENERATE
<a4n>, <g4n>, 1
*NSET, NSET=N_RHS, GENERATE
<a4n>, <g4n>, 1

*******************************************************************************
**COPY THE NODES TO GENERATE NODES IN OTHER HALF OF UNIT CELL
*******************************************************************************
*NCOPY, OLD_SET=ND1, NEW_SET=N_D1, CHANGE_NUMBER=<R30>, SHIFT <R3>, 0., 0.
*NCOPY, OLD_SET=ND2, NEW_SET=N_D2, CHANGE_NUMBER=<R30>, SHIFT <R3>, 0., 0.
*NCOPY, OLD_SET=ND3, NEW_SET=N_D3, CHANGE_NUMBER=<R30>, SHIFT <R3>, 0., 0.
*NCOPY, OLD_SET=ND4, NEW_SET=N_D4, CHANGE_NUMBER=<R30>, SHIFT <R3>, 0., 0.
*NCOPY, OLD_SET=ND5, NEW_SET=N_D5, CHANGE_NUMBER=<R30>, SHIFT <R3>, 0., 0.
*NCOPY, OLD_SET=ND6, NEW_SET=N_D6, CHANGE_NUMBER=<R30>, SHIFT <R3>, 0.,
*NCOPY, OLD_SET=N_1, NEW_SET=N_111, CHANGE_NUMBER=<R26>, SHIFT <R2>, 0., 0.
*NCOPY, OLD_SET=N_2, NEW_SET=N_22, CHANGE_NUMBER=<R26>, SHIFT <R2>, 0., 0.
*NCOPY, OLD_SET=N_3, NEW_SET=N_33, CHANGE_NUMBER=<R26>, SHIFT <R2>, 0., 0.
*NCOPY, OLD_SET=N_4, NEW_SET=N_44, CHANGE_NUMBER=<R26>, SHIFT <R2>, 0., 0.
*NCOPY, OLD_SET=N_5, NEW_SET=N_55, CHANGE_NUMBER=<R26>, SHIFT <R2>, 0., 0.
**GROUPING THE NODES IN UNIT CELL FOR EACH SOIL LAYERS**

*NCOPY, OLD SET=N_6, NEW SET=N_66, CHANGE NUMBER=<R26>, SHIFT
<R2>, 0., 0.

*******************************************************************

**COPY THE NODES TO GENERATE NODES UNDER WHOLE EMBANKMENT**

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R5>, SHIFT
<S>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R6>, SHIFT
<S2>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R7>, SHIFT
<S3>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R8>, SHIFT
<S4>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R9>, SHIFT
<S5>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R10>, SHIFT
<S6>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R11>, SHIFT
<S7>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R12>, SHIFT
<S8>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R13>, SHIFT
<S9>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R14>, SHIFT
<S10>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R15>, SHIFT
<S11>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R16>, SHIFT
<S12>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R17>, SHIFT
<S13>, 0., 0.
*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R18>, SHIFT <S14>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R19>, SHIFT <S15>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R20>, SHIFT <S16>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R21>, SHIFT <S17>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R22>, SHIFT <S18>, 0., 0.

*NCOPY, OLD SET=NS11, NEW SET=N_SOIL1, CHANGE NUMBER=<R23>, SHIFT <S19>, 0., 0.

**------------------------------------------------------------------

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R5>, SHIFT <S>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R6>, SHIFT <S2>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R7>, SHIFT <S3>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R8>, SHIFT <S4>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R9>, SHIFT <S5>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R10>, SHIFT <S6>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R11>, SHIFT <S7>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R12>, SHIFT <S8>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R13>, SHIFT <S9>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R14>, SHIFT <S10>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R15>, SHIFT <S11>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R16>, SHIFT <S12>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R17>, SHIFT <S13>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R18>, SHIFT <S14>, 0., 0.
*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R19>, SHIFT <S15>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R20>, SHIFT <S16>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R21>, SHIFT <S17>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R22>, SHIFT <S18>, 0., 0.

*NCOPY, OLD SET=NS22, NEW SET=N_SOIL2, CHANGE NUMBER=<R23>, SHIFT <S19>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R5>, SHIFT <S>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R6>, SHIFT <S2>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R7>, SHIFT <S3>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R8>, SHIFT <S4>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R9>, SHIFT <S5>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R10>, SHIFT <S6>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R11>, SHIFT <S7>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R12>, SHIFT <S8>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R13>, SHIFT <S9>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R14>, SHIFT <S10>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R15>, SHIFT <S11>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R16>, SHIFT <S12>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R17>, SHIFT <S13>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R18>, SHIFT <S14>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R19>, SHIFT
<S15>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R20>, SHIFT <S16>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R21>, SHIFT <S17>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R22>, SHIFT <S18>, 0., 0.

*NCOPY, OLD SET=NS33, NEW SET=N_SOIL3, CHANGE NUMBER=<R23>, SHIFT <S19>, 0., 0.

**---------------------------------------------------------------

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R5>, SHIFT <S>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R6>, SHIFT <S2>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R7>, SHIFT <S3>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R8>, SHIFT <S4>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R9>, SHIFT <S5>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R10>, SHIFT <S6>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R11>, SHIFT <S7>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R12>, SHIFT <S8>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R13>, SHIFT <S9>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R14>, SHIFT <S10>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R15>, SHIFT <S11>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R16>, SHIFT <S12>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R17>, SHIFT <S13>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R18>, SHIFT <S14>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R19>, SHIFT <S15>, 0., 0.
*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R20>, SHIFT <S16>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R21>, SHIFT <S17>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R22>, SHIFT <S18>, 0., 0.

*NCOPY, OLD SET=NS44, NEW SET=N_SOIL4, CHANGE NUMBER=<R23>, SHIFT <S19>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R5>, SHIFT <S>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R6>, SHIFT <S2>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R7>, SHIFT <S3>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R8>, SHIFT <S4>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R9>, SHIFT <S5>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R10>, SHIFT <S6>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R11>, SHIFT <S7>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R12>, SHIFT <S8>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R13>, SHIFT <S9>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R14>, SHIFT <S10>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R15>, SHIFT <S11>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R16>, SHIFT <S12>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R17>, SHIFT <S13>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R18>, SHIFT <S14>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R19>, SHIFT <S15>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R20>, SHIFT <S16>, 0., 0.
*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R21>, SHIFT <S17>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R22>, SHIFT <S18>, 0., 0.

*NCOPY, OLD SET=NS55, NEW SET=N_SOIL5, CHANGE NUMBER=<R23>, SHIFT <S19>, 0., 0.

**--------------------------------------------------------------

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R5>, SHIFT <S>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R6>, SHIFT <S2>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R7>, SHIFT <S3>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R8>, SHIFT <S4>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R9>, SHIFT <S5>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R10>, SHIFT <S6>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R11>, SHIFT <S7>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R12>, SHIFT <S8>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R13>, SHIFT <S9>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R14>, SHIFT <S10>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R15>, SHIFT <S11>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R16>, SHIFT <S12>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R17>, SHIFT <S13>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R18>, SHIFT <S14>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R19>, SHIFT <S15>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R20>, SHIFT <S16>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R21>, SHIFT
*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R22>, SHIFT <S18>, 0., 0.

*NCOPY, OLD SET=NS66, NEW SET=N_SOIL6, CHANGE NUMBER=<R23>, SHIFT <S19>, 0., 0.

******************************************************************
**GROUPING THE NODES IN EACH SOIL LAYER FOR WHOLE FOUNDATION
******************************************************************
*NSET, NSET=N_SOIL1
NS11
*NSET, NSET=N_SOIL2
NS22
*NSET, NSET=N_SOIL3
NS33
*NSET, NSET=N_SOIL4
NS44
*NSET, NSET=N_SOIL5
NS55
*NSET, NSET=N_SOIL6
NS66
******************************************************************
**GROUPING THE NODES FOR WHOLE FOUNDATION
******************************************************************
*NSET, NSET=N_ALL
N_SOIL1, N_SOIL2, N_SOIL3, N_SOIL4, N_SOIL5, N_SOIL6
******************************************************************
**GENERATING THE NODE SETS ALONG TOP AND BOTTOM BOUNDARIES
******************************************************************
*NSET, NSET=N_BOT, GENERATE
<an>, <a4n>, 300
*NSET, NSET=N_POR_TOP1, GENERATE
<gn>, <g3n>, 600
*NSET, NSET=N_POR_TOP2, GENERATE
<g3nl>, <g4n>, 600
******************************************************************
**GENERATES THE NODES ALONG THE DRAIN BOUNDARIES
******************************************************************
*NSET, NSET=N_POR_DRAIN, GENERATE
<an>, <a6n>, <R5>
<an2>, <a6n2>, <R5>
<an3>, <a6n3>, <R5>
<an4>, <a6n4>, <R5>
<an5>, <a6n5>, <R5>
<an6>, <a6n6>, <R5>
<an7>, <a6n7>, <R5>
<an8>, <a6n8>, <R5>
<an9>, <a6n9>, <R5>
<an10>, <a6n10>, <R5>
<an11>, <a6n11>, <R5>
<an12>, <a6n12>, <R5>
<an13>, <a6n13>, <R5>
<an14>, <a6n14>, <R5>
<an15>, <a6n15>, <R5>
<an16>, <a6n16>, <R5>
<an17>, <a6n17>, <R5>
<an18>, <a6n18>, <R5>
<an19>, <a6n19>, <R5>
**ELEMENT DEFINITION**

*ELEMENT, TYPE=CPE8RP

*aer*, <aer1>, <aer2>, <aer3>, <aer4>, <aer5>, <aer6>, <aer7>, <aer8>

*a1er*, <a1er1>, <a1er2>, <a1er3>, <a1er4>, <a1er5>, <a1er6>, <a1er7>, <a1er8>

*a3er*, <a3er1>, <a3er2>, <a3er3>, <a3er4>, <a3er5>, <a3er6>, <a3er7>, <a3er8>

*bet*, <bet1>, <bet2>, <bet3>, <bet4>, <bet5>, <bet6>, <bet7>, <bet8>

*b1er*, <b1er1>, <b1er2>, <b1er3>, <b1er4>, <b1er5>, <b1er6>, <b1er7>, <b1er8>

*b3er*, <b3er1>, <b3er2>, <b3er3>, <b3er4>, <b3er5>, <b3er6>, <b3er7>, <b3er8>

*cet*, <cet1>, <cet2>, <cet3>, <cet4>, <cet5>, <cet6>, <cet7>, <cet8>

*c1er*, <c1er1>, <c1er2>, <c1er3>, <c1er4>, <c1er5>, <c1er6>, <c1er7>, <c1er8>

*c3er*, <c3er1>, <c3er2>, <c3er3>, <c3er4>, <c3er5>, <c3er6>, <c3er7>, <c3er8>

*det*, <det1>, <det2>, <det3>, <det4>, <det5>, <det6>, <det7>, <det8>

*d1er*, <d1er1>, <d1er2>, <d1er3>, <d1er4>, <d1er5>, <d1er6>, <d1er7>, <d1er8>

*d3er*, <d3er1>, <d3er2>, <d3er3>, <d3er4>, <d3er5>, <d3er6>, <d3er7>, <d3er8>

*eet*, <eet1>, <eet2>, <eet3>, <eet4>, <eet5>, <eet6>, <eet7>, <eet8>

*e1er*, <e1er1>, <e1er2>, <e1er3>, <e1er4>, <e1er5>, <e1er6>, <e1er7>, <e1er8>

*e3er*, <e3er1>, <e3er2>, <e3er3>, <e3er4>, <e3er5>, <e3er6>, <e3er7>, <e3er8>

*fet*, <fet1>, <fet2>, <fet3>, <fet4>, <fet5>, <fet6>, <fet7>, <fet8>

*f1er*, <f1er1>, <f1er2>, <f1er3>, <f1er4>, <f1er5>, <f1er6>, <f1er7>, <f1er8>

*f3er*, <f3er1>, <f3er2>, <f3er3>, <f3er4>, <f3er5>, <f3er6>, <f3er7>, <f3er8>

**GENERATION OF ELEMENTS AWAY FROM THE EMBANKMENT**

*ELGEN, ELSET=E_F1

<a3er>, <a_bels>, 2, 2, <a3_a4els>, 600, 600

*ELGEN, ELSET=E_F2

<b3er>, <b_cels>, 2, 2, <a3_a4els>, 600, 600

*ELGEN, ELSET=E_F3

<c3er>, <c_dels>, 2, 2, <a3_a4els>, 600, 600

*ELGEN, ELSET=E_F4

<d3er>, <d_eels>, 2, 2, <a3_a4els>, 600, 600

*ELGEN, ELSET=E_F5

<e3er>, <e_fels>, 2, 2, <a3_a4els>, 600, 600

*ELGEN, ELSET=E_F6

<f3er>, <f_gels>, 2, 2, <a3_a4els>, 600, 600

**GENERATION OF ELEMENTS WITHIN THE SMEAR ZONE AT ONE SIDE OF THE DRAIN**

*ELGEN, ELSET=E1_S11

<aer>, <a_bels>, 2, 2, <a_a1els>, 600, 600

**IF THERE ARE MANY ELEMENTS IN UP AND HORIZONTAL DIRECTION**

*ELGEN, ELSET=E1_S11

<aer>, <a_bels>, 2, 2, <a_a1els>, 600, 600

**IF THERE IS ONLY ONE ELEMENT IN UP AND HORIZONTAL DIRECTION**

*ELGEN, ELSET=E1_S11

<aer>, <a_bels>, 2, 2, <a_a1els>, 600, 600

*ELGEN, ELSET=E2_S21

<bet>, <b_cels>, 2, 2, <a_a1els>, 600, 600

*ELGEN, ELSET=E3_S31

<cet>, <c_dels>, 2, 2, <a_a1els>, 600, 600

*ELGEN, ELSET=E4_S41

<det>, <d_eels>, 2, 2, <a_a1els>, 600, 600

*ELGEN, ELSET=E5_S51

<det>, <d_eels>, 2, 2, <a_a1els>, 600, 600

**ELGEN, ELSET=E6_S61

**ELGEN, ELSET=E6_S61

*ELGEN, ELSET=E6_S61
**GENERATION OF ELEMENTS OUTSIDE THE SMEAR ZONE AT ONE SIDE OF THE DRAIN**

**ELGEN,ELSET=E1_S12**
**<a1er>,<a_bels>,2,2,<a1_a2els>,600,600**
*ELSET,ELSET=E1_S12<br>**<a1er>,
*ELGEN,ELSET=E2_S22<br><b1er>,<b_cels>,2,2,<a1_a2els>,600,600
*ELGEN,ELSET=E3_S32<br><c1er>,<c_dels>,2,2,<a1_a2els>,600,600
*ELGEN,ELSET=E4_S42<br><d1er>,<d_eels>,2,2,<a1_a2els>,600,600
*ELGEN,ELSET=E5_S52<br><e1er>,<e_fels>,2,2,<a1_a2els>,600,600
**ELGEN,ELSET=E6_S62<br>**<f1er>,<f_gels>,2,2,<a1_a2els>,600,600
*ELSET,ELSET=E6_S62<br><f1er>,

**GENERATE ELEMENTS BETWEEN TWO DRAINS**

*ELCOPY,OLD SET=E1_S11,NEWSET=E_S11,SHIFT NODES=<R30>, ELEMENT SHIFT=<R30>
*ELCOPY,OLD SET=E1_S12,NEWSET=E_S12,SHIFT NODES=<R26>, ELEMENT SHIFT=<R26>
*ELCOPY,OLD SET=E2_S21,NEWSET=E_S21,SHIFT NODES=<R30>, ELEMENT SHIFT=<R30>
*ELCOPY,OLD SET=E2_S22,NEWSET=E_S22,SHIFT NODES=<R26>, ELEMENT SHIFT=<R26>
*ELCOPY,OLD SET=E3_S31,NEWSET=E_S31,SHIFT NODES=<R30>, ELEMENT SHIFT=<R30>
*ELCOPY,OLD SET=E3_S32,NEWSET=E_S32,SHIFT NODES=<R26>, ELEMENT SHIFT=<R26>
*ELCOPY,OLD SET=E4_S41,NEWSET=E_S41,SHIFT NODES=<R30>, ELEMENT SHIFT=<R30>
*ELCOPY,OLD SET=E4_S42,NEWSET=E_S42,SHIFT NODES=<R26>, ELEMENT SHIFT=<R26>
*ELCOPY,OLD SET=E5_S51,NEWSET=E_S51,SHIFT NODES=<R30>, ELEMENT SHIFT=<R30>
*ELCOPY,OLD SET=E5_S52,NEWSET=E_S52,SHIFT NODES=<R26>, ELEMENT SHIFT=<R26>
*ELCOPY,OLD SET=E6_S61,NEWSET=E_S61,SHIFT NODES=<R30>, ELEMENT SHIFT=<R30>
*ELCOPY,OLD SET=E6_S62,NEWSET=E_S62,SHIFT NODES=<R26>, ELEMENT SHIFT=<R26>

**GROUPING ELEMENTS IN EACH SOIL LAYER BETWEEN TWO DRAINS**

*ELSET,ELSET=E_S11<br>E1_S11,
*ELSET,ELSET=E_S12<br>E1_S12,
*ELSET,ELSET=E_S21<br>E2_S21,
*ELSET,ELSET=E_S22<br>E2_S22,
*ELSET,ELSET=E_S31
E3_S31,
*ELSET,ELSET=E_S32
E3_S32,
*ELSET,ELSET=E_S41
E4_S41,
*ELSET,ELSET=E_S42
E4_S42,
*ELSET,ELSET=E_S51
E5_S51,
*ELSET,ELSET=E_S52
E5_S52,
*ELSET,ELSET=E_S61
E6_S61,
*ELSET,ELSET=E_S62
E6_S62,

***************************************************************
**GENERATE ELEMENTS UNDER THE EMBANKMENT
***************************************************************
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R5>,
ELEMENT SHIFT=<R5>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R6>,
ELEMENT SHIFT=<R6>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R7>,
ELEMENT SHIFT=<R7>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R8>,
ELEMENT SHIFT=<R8>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R9>,
ELEMENT SHIFT=<R9>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R10>,
ELEMENT SHIFT=<R10>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R11>,
ELEMENT SHIFT=<R11>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R12>,
ELEMENT SHIFT=<R12>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R13>,
ELEMENT SHIFT=<R13>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R14>,
ELEMENT SHIFT=<R14>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R15>,
ELEMENT SHIFT=<R15>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R16>,
ELEMENT SHIFT=<R16>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R17>,
ELEMENT SHIFT=<R17>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R18>,
ELEMENT SHIFT=<R18>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R19>,
ELEMENT SHIFT=<R19>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R20>,
ELEMENT SHIFT=<R20>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R21>,
ELEMENT SHIFT=<R21>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R22>,
ELEMENT SHIFT=<R22>
*ELCOPY,OLD SET=E_S11,NEWSET=E_SOIL11,SHIFT NODES=<R23>,
ELEMENT SHIFT=<R23>
*ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R4>,
ELEMENT SHIFT=<R4>

**-----------------------------------------------------------
*ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R5>,
ELEMENT SHIFT=<R5>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R6>,**
ELEMENT SHIFT=<R6>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R7>,**
ELEMENT SHIFT=<R7>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R8>,**
ELEMENT SHIFT=<R8>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R9>,**
ELEMENT SHIFT=<R9>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R10>,**
ELEMENT SHIFT=<R10>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R11>,**
ELEMENT SHIFT=<R11>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R12>,**
ELEMENT SHIFT=<R12>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R13>,**
ELEMENT SHIFT=<R13>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R14>,**
ELEMENT SHIFT=<R14>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R15>,**
ELEMENT SHIFT=<R15>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R16>,**
ELEMENT SHIFT=<R16>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R17>,**
ELEMENT SHIFT=<R17>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R18>,**
ELEMENT SHIFT=<R18>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R19>,**
ELEMENT SHIFT=<R19>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R20>,**
ELEMENT SHIFT=<R20>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R21>,**
ELEMENT SHIFT=<R21>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R22>,**
ELEMENT SHIFT=<R22>
**ELCOPY,OLD SET=E_S12,NEWSET=E_SOIL12,SHIFT NODES=<R23>,**
ELEMENT SHIFT=<R23>**--------------------------------------------------------------
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R5>,**
ELEMENT SHIFT=<R5>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R6>,**
ELEMENT SHIFT=<R6>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R7>,**
ELEMENT SHIFT=<R7>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R8>,**
ELEMENT SHIFT=<R8>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R9>,**
ELEMENT SHIFT=<R9>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R10>,**
ELEMENT SHIFT=<R10>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R11>,**
ELEMENT SHIFT=<R11>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R12>,**
ELEMENT SHIFT=<R12>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R13>,**
ELEMENT SHIFT=<R13>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R14>,**
ELEMENT SHIFT=<R14>
**ELCOPY,OLD SET=E_S21,NEWSET=E_SOIL21,SHIFT NODES=<R15>,**
ELEMENT SHIFT=<R15>
*ELCOPY, OLD SET=E_S21, NEWSET=E_SOIL21, SHIFT NODES=<R16>, ELEMENT SHIFT=<R16>
*ELCOPY, OLD SET=E_S21, NEWSET=E_SOIL21, SHIFT NODES=<R17>, ELEMENT SHIFT=<R17>
*ELCOPY, OLD SET=E_S21, NEWSET=E_SOIL21, SHIFT NODES=<R18>, ELEMENT SHIFT=<R18>
*ELCOPY, OLD SET=E_S21, NEWSET=E_SOIL21, SHIFT NODES=<R19>, ELEMENT SHIFT=<R19>
*ELCOPY, OLD SET=E_S21, NEWSET=E_SOIL21, SHIFT NODES=<R20>, ELEMENT SHIFT=<R20>
*ELCOPY, OLD SET=E_S21, NEWSET=E_SOIL21, SHIFT NODES=<R21>, ELEMENT SHIFT=<R21>
*ELCOPY, OLD SET=E_S21, NEWSET=E_SOIL21, SHIFT NODES=<R22>, ELEMENT SHIFT=<R22>
*ELCOPY, OLD SET=E2_S21, NEWSET=E_SOIL21, SHIFT NODES=<R23>, ELEMENT SHIFT=<R23>
*ELCOPY, OLD SET=E3_S21, NEWSET=E_SOIL22, SHIFT NODES=<R5>, ELEMENT SHIFT=<R5>
*ELCOPY, OLD SET=E3_S21, NEWSET=E_SOIL22, SHIFT NODES=<R6>, ELEMENT SHIFT=<R6>
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**GROUPING THE ELEMENTS IN EACH SOIL LAYER IN WHOLE FOUNDATION**

**GENERATE ELEMENT SETS ON TOP SURFACE**

**-----MATERIAL DEFINITION-----**

**ASSIGN MATERIAL PROPERTIES TO EACH ELEMENT SETS**
*SOLID SECTION, ELSET=E_SOIL21, MATERIAL=E_SOIL21
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*SOLID SECTION, ELSET=E_SOIL41, MATERIAL=E_SOIL41
*SOLID SECTION, ELSET=E_SOIL51, MATERIAL=E_SOIL51
*SOLID SECTION, ELSET=E_SOIL61, MATERIAL=E_SOIL61
*SOLID SECTION, ELSET=E_SOIL12, MATERIAL=E_SOIL12
*SOLID SECTION, ELSET=E_SOIL22, MATERIAL=E_SOIL22
*SOLID SECTION, ELSET=E_SOIL32, MATERIAL=E_SOIL32
*SOLID SECTION, ELSET=E_SOIL42, MATERIAL=E_SOIL42
*SOLID SECTION, ELSET=E_SOIL52, MATERIAL=E_SOIL52
*SOLID SECTION, ELSET=E_SOIL62, MATERIAL=E_SOIL62
*****************************************************************
**DEFINING MATERIAL PROPERTIES
*****************************************************************
*MATERIAL, NAME=E_SOIL11
*POROUS ELASTIC
0.01, 0.25
*CLAY PLASTICITY, INTERCEPT=1.63
0, 1, 1.4
*PERMEABILITY, TYPE=ISO, SPECIFIC=1.E+4
6.31E-10,
*MATERIAL, NAME=E_SOIL12
*POROUS ELASTIC
0.01, 0.25
*CLAY PLASTICITY, INTERCEPT=1.63
0, 1, 1.4
*PERMEABILITY, TYPE=ISO, SPECIFIC=1.E+4
1.8E-10,
*MATERIAL, NAME=E_SOIL21
*POROUS ELASTIC
0.03, 0.25
*CLAY PLASTICITY, INTERCEPT=2.94
0.3, 1, 1.4
*PERMEABILITY, SPECIFIC=1.E+4
2.67E-10,
*MATERIAL, NAME=E_SOIL22
*POROUS ELASTIC
0.03, 0.25
*CLAY PLASTICITY, INTERCEPT=2.94
0.3, 1, 1.4
*PERMEABILITY, TYPE=ISO, SPECIFIC=1.E+4
7.6E-10,
*MATERIAL, NAME=E_SOIL31
*POROUS ELASTIC
0.05, 0.25
*CLAY PLASTICITY, INTERCEPT=4.212
0.5, 1, 1.2
*PERMEABILITY, SPECIFIC=1.E+4
6.31E-10,
*MATERIAL, NAME=E_SOIL32
*POROUS ELASTIC
0.05, 0.25
*CLAY PLASTICITY, INTERCEPT=4.212
0.5, 1, 1.2
*PERMEABILITY, TYPE=ISO, SPECIFIC=1.E+4
1.8E-9,
*MATERIAL, NAME=E_SOIL41
*POROUS ELASTIC
0.08, 0.3
*CLAY PLASTICITY, INTERCEPT=5.27
0.73,1.0
*PERMEABILITY,SPECIFIC=1.E+4 1.33E-9,
*MATERIAL,NAME=E_SOIL42
*POROUS ELASTIC
0.08,0.3
*CLAY PLASTICITY,INTERCEPT=5.27 0.73,1.0
*PERMEABILITY,TYPE=ISO,SPECIFIC=1.E+4 3.8E-9,
*MATERIAL,NAME=E_SOIL51
*POROUS ELASTIC
0.03,0.3
*CLAY PLASTICITY,INTERCEPT=2.797 0.3,1.2
*PERMEABILITY,SPECIFIC=1.E+4 3.15E-9,
*MATERIAL,NAME=E_SOIL52
*POROUS ELASTIC
0.03,0.3
*CLAY PLASTICITY,INTERCEPT=2.797 0.3,1.2
*PERMEABILITY,TYPE=ISO,SPECIFIC=1.E+4 8.98E-9,
*MATERIAL,NAME=E_SOIL61
*POROUS ELASTIC
0.03,0.3
*CLAY PLASTICITY,INTERCEPT=2.797 0.3,1.2
*PERMEABILITY,TYPE=ISO,SPECIFIC=1.E+4 8.98E-9,
********************************************************************
**DEFINING NODE SETS FOR MODEL DATA OUTPUT REQUEST
********************************************************************
*NSET,NSET=N_OUT 39,1231,48031,48039
*ELSET,ELSET=E_OUT <aer>,
********************************************************************
**DEFINING THE VARIATION LOAD WITH TIME
********************************************************************
*AMPLITUDE,NAME=LD1 0.0,0.0,24192000.0,1.0
********************************************************************
**DEFINING THE INITIAL CONDITIONS
********************************************************************
*INITIAL CONDITIONS,TYPE=RATIO N_SOIL1,1.2 N_SOIL2,1.8 N_SOIL3,2.4 N_SOIL4,2.8 N_SOIL5,1.8 N_SOIL6,1.8
*INITIAL CONDITIONS, TYPE=STRESS, GEOSTATIC
E_SOIL11, -80.75E+3, 0., -64.75E+3, <H1>, 0.43
E_SOIL21, -64.75E+3, <H1>, -49.75E+3, <H2>, 0.46
E_SOIL31, -49.75E+3, <H2>, -39.75E+3, <H3>, 0.56
E_SOIL41, -39.75E+3, <H3>, -11.0E+3, <H4>, 0.71
E_SOIL51, -11.0E+3, <H4>, -8000., <H5>, 1.0
E_SOIL61, -8.0E+3, <H5>, -5000., <H>, 1.0
E_SOIL12, -80.75E+3, 0., -64.75E+3, <H1>, 0.43
E_SOIL22, -64.75E+3, <H1>, -49.75E+3, <H2>, 0.46
E_SOIL32, -49.75E+3, <H2>, -39.75E+3, <H3>, 0.56
E_SOIL42, -39.75E+3, <H3>, -11.0E+3, <H4>, 0.71
E_SOIL52, -11.0E+3, <H4>, -8000., <H5>, 1.0
E_SOIL62, -8.0E+3, <H5>, -5000., <H>, 1.0
*INITIAL CONDITIONS, TYPE=PORE PRESSURE
N_ALL, 145000., 0., 0., <H5>
********************************************************************
**DEFINING INITIAL BOUNDARY CONDITIONS
********************************************************************
*BOUNDARY
N_LHS, 1
N_BOT, 1, 2
N_POR_TOP1, 8
N_POR_TOP2, 8
N_RHS, 1
**********************************************************************
** HISTORY DATA
**********************************************************************
**STEP 1: INITIAL GEOSTATIC STATE
**********************************************************************
*STEP, NLGEOM
*GEOSTATIC
*DLOAD
E_SOIL11, BY, -8.0E+3
E_SOIL21, BY, -6.0E+3
E_SOIL31, BY, -5.0E+3
E_SOIL41, BY, -4.5E+3
E_SOIL51, BY, -6.0E+3
E_SOIL61, BY, -6.0E+3
E_SOIL12, BY, -8.0E+3
E_SOIL22, BY, -6.0E+3
E_SOIL32, BY, -5.0E+3
E_SOIL42, BY, -4.5E+3
E_SOIL52, BY, -6.0E+3
E_SOIL62, BY, -6.0E+3
*MONITOR, NODE=<g3n>, DOF=2
**OUTPUT, FIELD, FREQUENCY=10
**NODE OUTPUT, NSET=N_ALL
**U, POR
**OUTPUT, HISTORY, FREQUENCY=10
**NODE OUTPUT, NSET=N_OUT
**U, POR
**EL PRINT, ELSET=E_OUT, POSITION=CENTROIDAL
**S11, S22, POR
**NODE PRINT, NSET=N_SOIL6
**U, POR
**NODE PRINT, NSET=N_SOIL5
**U, POR
*NODE PRINT, NSET=N_OUT
U, POR
*CONTROLS, ANALYSIS=DISCONTINUOUS
*CONTROLS,PARAMETERS=FIELD,FIELD=PORE FLUID PRESSURE
0.01,1.0,10.0,,1.E-4
*END STEP
***************************************************************************
**STEP2-START CONSTRUCTION
***************************************************************************
*STEP,INC=500,NLGEOM
CONSOLIDATION OVER 8DAYS
*SOILS,CONSOLIDATION,UTOL=5000000.
86400.0,31536000.0,100.0,
*DLOAD,AMPLITUDE=LD1
E_TOP3,P3,73.125E+3
<g3el33>,P3,73.125E+3
<g3el32>,P3,70.875E+3
<g3el31>,P3,68.625E+3
<g3el30>,P3,66.375E+3
<g3el29>,P3,64.125E+3
<g3el28>,P3,61.875E+3
<g3el27>,P3,59.625E+3
<g3el26>,P3,57.375E+3
<g3el25>,P3,55.125E+3
<g3el24>,P3,52.875E+3
<g3el23>,P3,52.875E+3
<g3el22>,P3,50.625E+3
<g3el21>,P3,48.375E+3
<g3el20>,P3,46.125E+3
<g3el19>,P3,43.875E+3
<g3el18>,P3,41.625E+3
<g3el17>,P3,39.375E+3
<g3el16>,P3,37.125E+3
<g3el15>,P3,34.875E+3
<g3el14>,P3,32.625E+3
<g3el13>,P3,30.375E+3
<g3el12>,P3,28.125E+3
<g3el11>,P3,25.875E+3
<g3el10>,P3,23.625E+3
<g3el9>,P3,21.375E+3
<g3el8>,P3,19.125E+3
<g3el7>,P3,16.875E+3
<g3el6>,P3,14.625E+3
<g3el5>,P3,12.375E+3
<g3el4>,P3,10.125E+3
<g3el3>,P3,7.857E+3
<g3el2>,P3,5.625E+3
<g3el1>,P3,3.375E+3
<g3el>,P3,1.125E+3
**E_TOP1,P3,5.4E+3
***************************************************************************
**IF DRAINS ARE PROVIDED
*BOUNDARY,OP=MOD
N_POR_DRAIN,8,8,0.0
***************************************************************************
**if vacuum pressure is applied
**N_POR_TOP1,8,8,-50000.0
***************************************************************************
*END STEP