CENTRIFUGE AND NUMERICAL MODELLING OF
SAND COMPACTION PILE INSTALLATION

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SUMMARY

The installation of sand compaction pile (SCP) has been known to have a considerable impact on the surrounding soils. This research work focuses on evaluating the influence of sand compaction piling, particularly the resulting strength set-up in the adjacent clay. The study comprises both centrifuge experimental and numerical modelling.

The centrifuge tests were carried out to measure the changes in radial stresses and pore pressures in soft clays during and after the in-flight installation of sand compaction piles. It was noted that the measured peak increases in stress and pore pressure could be reasonably estimated by cavity expansion theory. Substantial strength improvements in the clay were observed after pile installation. The strength enhancement was considerably affected by consolidation effects, as well as the number of piles. For pile group installation, the dissipation of excess pore pressures between successive pile installations had a significant influence on the strength set-up effect.

The numerical analysis work in this study comprises two phases. The first phase was undertaken to validate the proposed numerical approach for modeling deep penetration problems involving consolidation effects. For this phase, the study problem was selected as the penetration of the cone penetrometer under various rates. Coupled consolidation finite element analyses were carried out to simulate the deep cone penetration using ABAQUS/Standard V6.6. A wide range of penetration rates was considered to cover the full spectrum of consolidation or drainage conditions.
As the penetration rate decreased, the transition from undrained to partially drained, and then to fully drained was clearly observed. The numerical results from the extremely fast and slow penetration, corresponding to the limiting undrained and drained conditions, compare favorably with various analytical and numerical solutions. The computed normalized backbone curve, which illustrates the effect of cone penetrate rate, was found to agree well with published centrifuge results. Using the hyperbolic curve fitting approach, a simplified procedure was proposed to derive the backbone curve for a soil with given strength and stiffness properties.

The second phase of the numerical study uses the deep penetration modeling techniques established in the first phase to carry out finite element analysis of sand compaction pile installation. Reasonable agreement was obtained between the numerical results and those obtained from the centrifuge experiments. By carrying out additional parametric studies, the numerical results provide a comprehensive information database which describes changes in the strains, stresses, pore pressures, and strengths during and after pile installation. More importantly, the extent and magnitude of the strength set-up effect may be defined and quantified by the computed strength improvement radial profiles. A logarithmic function was proposed to approximate these strength improvement profiles, which uses two fitting parameters that are correlated with the soil’s properties. This led to the development of a simple and practical means for predicting the long-term strength increase due to the sand compaction pile installation.
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### LIST OF SYMBOLS

- **A**: cross sectional areas of unit cell
- **A_c**: cross sectional areas of the clay within the unit cell
- **A_f**: Skempton pore pressure parameter at failure
- **A_s**: cross sectional areas of the sand pile within the unit cell
- **a**: smallest center-to-center spacing within the pile grid
- **a_c**: centrifuge acceleration level
- **a_s**: area replacement ratio (as = A_s/A)
- **B**: width of foundation
- **b, c, m**: hyperbolic constants
- **c’**: effective cohesion
- **c_v**: coefficient of consolidation of soil
- **D’**: soil effective constrained modulus
- **d**: diameter of the cone
- **d_b**: diameter of the cross bar of T-bar penetrometer
- **d_f**: equivalent nominal thickness of the wall
- **d_s**: diameter of the sand column
- **E_c**: Young’s modulus of the clay
- **E_s**: Young’s modulus of the sand
- **E_u**: undrained Young’s modulus of the soil
- **e**: void ratio
- **e_0**: reference void ratio
- **F**: applied external load
- **F_r**: reduction factor
- **F_s**: factor of safety.
- **G**: soil shear modulus
- **G_i**: initial soil shear modulus
- **G_z**: shear modulus at depth z
- **G_0**: shear modulus at the ground surface
- **H**: thickness of subsoil
Ir  rigidity index
Kc  coefficient of earth pressure in the clay
Kp  passive earth pressure coefficient of tributary clay
Ks  coefficient of earth pressure in the sand column
K0  coefficient of earth pressure at rest
k   coefficient of permeability
M   friction coefficient (M = 6 sin φ'/(3 - sin φ'))
mv  coefficient of volume change of clay
Nb  bar factor of the T-bar penetration
Nc  cone factor of the undrained penetration
Nd  cone factor of the drained penetration
Nα  bearing capacity factors for self-weight
Nβ  bearing capacity factors for cohesion
n   stress concentration ratio
P   the force per unit length acting on the cylinder,
Pf  maximum load on the composite ground
p'  mean effective normal stress
p'c  pre-compression pressure
p'o  initial mean effective stress
q   deviator stress
qc  cone tip resistance
q'(c  effective cone tip resistance
qnet  net cone resistance (qnet= qc- σvo)
qdrained  drained net cone resistance (qdrained= qnet-drained)
qf  ultimate bearing capacity
qref  reference (undrained) net cone resistance (qref= qnet-undrained)
qt  surcharge per unit area.
R   cone radius
Rc  radius of casing
Ri  isotropic overconsolidation ratio
\( R_l \)  radius of slip circle  
\( R_{hp}, Z_p \)  horizontal and vertical extent of plastic zone  
\( R_s \)  radius of sand compaction pile  
\( R_u \)  cavity radius.  
\( r \)  radial distance  
\( S \)  ground settlement  
\( S_0 \)  original ground settlement  
\( s \)  distance to the nearest drainage boundary  
\( s_u \)  undrained shear strength  
\( s_{u,i} \)  initial strength of the clay  
\( s_{u,l} \)  post-installation, post-consolidation undrained shear strength  
\( s_{uo} \)  cohesion of clay at surface  
\( s_u/q_t \)  ratio of undrained shear strength increase due to surcharge  
\( T \)  dimensionless time  
\( t \)  duration of the event  
\( U \)  degree of consolidation  
\( u \)  total pore pressure  
\( V \)  non-dimensional velocity  
\( v \)  penetration velocity  
\( W \)  weight of soil slice  
\( x \)  lever arm  
\( z \)  depth below surface  
\( \Delta \) initial stress anisotropy \((\Delta=(\sigma_{v0} - \sigma_{h0})/2s_u)\)  
\( \Delta l \)  arc of slip circle  
\( \Delta \sigma_c, \Delta u_c \) calculated total stress and pore pressure increases  
\( \Delta \sigma_h \)  difference of lateral pressure increases between sand column and clay  
\( \Delta \sigma_m, \Delta u_m \) measured peak total stress and pore pressure increases  
\( \Delta \sigma_r \)  total radial stress increase  
\( \Delta u \)  excess pore pressure  
\( \Lambda \) plastic volumetric strain ratio \((\Lambda= (\lambda-\kappa)/\lambda)\)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>M</td>
<td>friction coefficient ( (M = 6 \sin \phi'/(3 - \sin \phi')) )</td>
</tr>
<tr>
<td>( \phi' )</td>
<td>effective friction angle</td>
</tr>
<tr>
<td>( \phi_m )</td>
<td>average friction angle of composite ground ( (\phi_m = \tan^{-1}(\mu_s \tan \phi_s)) )</td>
</tr>
<tr>
<td>( \phi_s )</td>
<td>friction angle of sand</td>
</tr>
<tr>
<td>( \psi )</td>
<td>dilation angle</td>
</tr>
<tr>
<td>( \alpha )</td>
<td>inclination of the failure surface measured from horizontal plane</td>
</tr>
<tr>
<td>( \beta )</td>
<td>settlement reduction factor</td>
</tr>
<tr>
<td>( \gamma_c )</td>
<td>unit weight of clay</td>
</tr>
<tr>
<td>( \gamma_m )</td>
<td>average unit weight of composite ground ( (\gamma_m = \gamma_s a_s + \gamma_c (1 - a_s)) )</td>
</tr>
<tr>
<td>( \gamma_{\text{max}} )</td>
<td>maximum shear strain ( (\gamma_{\text{max}} = (\varepsilon_1 - \varepsilon_3)/2) )</td>
</tr>
<tr>
<td>( \gamma_s )</td>
<td>unit weight of sand pile</td>
</tr>
<tr>
<td>( \gamma' )</td>
<td>effective unit weight of soil</td>
</tr>
<tr>
<td>( \gamma_w )</td>
<td>unit weight of water</td>
</tr>
<tr>
<td>( \varepsilon_1, \varepsilon_3 )</td>
<td>major and minor principal strains</td>
</tr>
<tr>
<td>( \varepsilon_r )</td>
<td>radial strain</td>
</tr>
<tr>
<td>( \varepsilon_{ez} )</td>
<td>meridional shear strain</td>
</tr>
<tr>
<td>( \varepsilon'_p )</td>
<td>plastic volumetric strain</td>
</tr>
<tr>
<td>( \xi )</td>
<td>shear strength increment rate with depth</td>
</tr>
<tr>
<td>( \kappa )</td>
<td>re-compression index under isotropic loading</td>
</tr>
<tr>
<td>( \lambda )</td>
<td>compression index under isotropic loading</td>
</tr>
<tr>
<td>( \mu_c )</td>
<td>ratio of stress on clay to the average loading intensity</td>
</tr>
<tr>
<td>( \mu_s )</td>
<td>ratio of stress on sand pile to the average loading intensity</td>
</tr>
<tr>
<td>( \nu' )</td>
<td>effective Poisson’s ratio</td>
</tr>
<tr>
<td>( \nu_c )</td>
<td>Poisson’s ratio of clay</td>
</tr>
<tr>
<td>( \sigma )</td>
<td>average loading intensity,</td>
</tr>
<tr>
<td>( \sigma_c )</td>
<td>vertical stress on the clay</td>
</tr>
<tr>
<td>( \sigma_h )</td>
<td>lateral confining stress on the cylindrical surface of sand pile</td>
</tr>
<tr>
<td>( \sigma_{\text{h0}} )</td>
<td>in-situ total horizontal stress.</td>
</tr>
<tr>
<td>( \sigma_r )</td>
<td>total radial stress</td>
</tr>
<tr>
<td>( \sigma'_r )</td>
<td>effective radial stress</td>
</tr>
</tbody>
</table>
\( \sigma_s \)  vertical stress on the sand pile

\( \sigma_{v0} \)  in-situ total vertical stress

\( \sigma'_{v0} \)  in-situ effective vertical stress

\( \sigma_u \)  upper yield stress of clayey ground

\( \sigma_w \)  vertical load intensity applied by the overlying soil mass

\( \tau_{sc} \)  shear strength of the composite ground

\( \varphi \)  angle between the postulated failure surface and foundation

\( \omega \)  angular rotational speed of centrifuge
Chapter 1: INTRODUCTION

1.1 Background

Construction works in soft grounds often encounter problems originating from weak engineering properties of soft soils such as low bearing capacity, excessive settlements and ground movements. Various ground improvement methods are thereby developed and implemented to treat soft soil, one of which is the sand compaction pile (SCP) method.

The method of sand compaction pile improves weak soil strataums by introducing a number of well-compacted and large-diameter sand columns into soil, with the latter being substantially strengthened and reinforced. The sand compaction piling was originally developed to densify and improve the loose sandy ground (Kitazume, 2005). Its applications soon extended to soft clayey soil, where it found extensive usage in rapid and cost-effective bearing capacity improvement, stability enhancement and settlement reduction (e.g. Aboshi & Suematsu, 1985; Nakata et al., 1991; Kitazume, 2005). With its low-cost, rapid highly-automated installation operation and versatility of usages, the method of sand compaction pile has been practiced worldwide for the ground treatment, especially in East and Southeast Asia (e.g. Japan, Korea, Singapore, etc). In Japan, the sand compaction piling is a popular ground treatment method with wide applications in both on-land and near-shore projects such as constructions of building foundation, embankment, port and harbor facilities and sea revetment (e.g. Aboshi et al., 1979; Moroto & Poorooshasb, 1991; Kitazume, 2005). As Figure 1.1 indicates, the cumulative length of sand compaction
pile in Japan increased rapidly in the last several decades and reached up to 350 thousand kilometer in 2001 (Kitazume, 2005). In Singapore, sand compaction piles were adopted in land reclamation projects, such as those at Marina Bay, Tanjong Rhu and Tuas (e.g. Wei & Khoo, 1992; Wei et al., 1995). In addition, sand compaction piling was also used in the constructions of port and harbor facilities in Singapore. For instance, sand compaction piles with diameters of 2m and area replacement ratio of 70% were used in the Pasir Panjang container terminal project to improve the foundation system of caisson wharf structure (e.g. Ng et al., 1995).

Since its introduction in the 1950s (Murayama, 1957), the SCP construction technique and machinery has evolved and undergone significant advancement. During the 1950s and 1960s, the hammering compaction technique was adopted for the construction of the compacted sand column. It was subsequently phased out and replaced by the vibro-compaction technique, which is much more energy-efficient and environmentally-friendly. The appearance of auto-control execution system in the 1980s undoubtedly expedited the construction speed and enhanced its capability to accommodate variation in soil properties. The advancing construction technique also helped to extend use of sand compaction pile from on-land to near-shore constructions and allow for greater improvement depth. The maximum reported improvement depth was 70m (Kitazume, 2005).

The most commonly practiced construction method of SCP is the “compozer method” (Aboshi & Suematsu, 1985). Figures 1.2 and 1.3 illustrate the typical equipment for on-land and near-shore constructions using the compozer method,
respectively. The construction procedure of the compozer method consists of several steps as depicted in Figure 1.4. The steel casing is first positioned to the prescribed locations, collimation of which in marine construction is assisted by the transit apparatus, optical finder, or GPS. Under excitation from the vibro-hammer, the casing pipe is then driven downwards into the ground. Sand is continually in-filled into the casing pipe during its penetration. In case of stiff soil layer, compressed air may be used to assist in the penetration. After arriving at the desired depth, the casing pipe is hoisted upwards by certain height to discharge and feed sand into the ground. Afterwards, the casing pipe is partially re-driven downwards to squash and compact the discharged sand, which also enlarges the diameter of sand column. The above procedure of withdrawal followed by partially re-driving is repeated up to the soil surface. At the end, a well-compacted sand column with a diameter larger than the casing pile is constructed in soil (e.g. Aboshi & Suematsu, 1985; Kitazume, 2005). Due to the use of vibro-hammer, the preceding execution procedure is inevitably accompanied with some noise and disturbance, which may restrict the use of SCP construction in urban areas. To mitigate the noise problem, a non-vibratory compaction technique was recently developed in Japan (Tsuboi et al., 2003). As shown in Figure 1.5, the non-vibratory compaction technique utilizes a rotary motor, instead of vibro-hammer, to facilitate penetration and withdrawal. The use of rotary motor can substantially reduce the amount of noise and vibration generated during SCP construction and retain favorable compaction efficacy.
1.2 The “Set-up” effect in soil

The SCP-induced ground improvement effect stems from two sources: the reinforcement effect due to the presence of stiff, compacted sand columns and the so-called “set-up” effect in the surrounding clay (Asaoka et al., 1994b; Lee et al., 2001; Guetif et al., 2007). As in the case of displacement pile installation (Randolph & Wroth, 1979), the set-up effect or the set-up of soil’s shear strength due to the SCP installation is a displacement- and consolidation-related phenomenon. The SCP construction, as described earlier, includes the intrusion of casing pipe and formation of compacted sand column. A considerable amount of excess pore pressure is generated in soil during the SCP installation process where soil is squeezed and displaced to accommodate the intrusion of casing and compaction of sand. The subsequent dissipation of excess pore pressure leads to an increase in effective stress and thereby the strength of the soil. A considerable set-up of shear strength in soil can be attained by the end of consolidation (Asaoka et al., 1994b). The set-up effect in clay has been verified by a wealth of field measurements (Enokido et al., 1973; Aboshi et al., 1979; Yagyu et al., 1991; Matsuda et al., 1997). Enokido et al. (1973) reported the unconfined compressive strengths of clay were almost double its initial strength values when measured about 45 days after the SCP installation. Aboshi et al. (1979) also reported the time-relevant strength changes in the surrounding soil after the SCP installation, with as much as 70% strength gain recorded around one month after the pile driving. Yagyu et al. (1991) observed a substantial (approximately 80%) strength build-up in the clay about 10 months after the SCP installation in Maizuru Port (Japan),
which was attributed by Asaoka et al. (1994b) predominantly to the clay’s set-up after sand pile installation. Apart from the field observation, geotechnical centrifuge test results (Lee et al., 2001 & 2004; Juneja, 2002; Ng, 2003; Weber et al., 2005 & 2009) also suggested that the SCP installation would exert significant influence on the soil bed. Similar conclusions were drawn by numerical (Farias et al., 2005; Guetif et al., 2007) and theoretical analyses (Asaoka et al., 1994b; Lee et al., 2004).

While the set-up effect has been alluded to or highlighted by field, experimental, numerical and theoretical studies, this phenomenon is still not normally considered in engineering design of SCP (e.g. Aboshi et al., 1979; Sogabe, 1981; Aboshi & Suematsu, 1985). The strength set-up in clay is generally ignored in the current design methodologies. For instance, in Aboshi et al.’s (1979) design framework, consideration was only given to the sand piles itself, the overall shear strength of the composite ground being taken to be some weighted average of that of the sand pile and the in-situ strength of the soft clay. This often led to rather conservative design wherein relatively high area replacement ratios up to 80% were used (e.g. Kitazume, 2005). Partly because of this, SCP has not found wider usage in other types of construction and in other countries such as the UK. Hight (2002), for instance, suggested that, if significant improvement could be achieved with area replacement ratios of about 10%, then SCP would be much more viable economically.

The main reason for this probably lies in the fact that much remains unknown with respect to the quantum of improvement and the various factors affecting it. For this reason, it is difficult to establish clear design methodology which takes this set-up
1.3 Research scope and objective

The objective of this study is to build upon previous studies, especially that of Lee et al. (2004), clarify the factors affecting the amount of improvement due to the set-up effect and develop some basic framework to account for them in the engineering design. The present research includes both centrifuge experimental study and finite element analysis. The experimental works were designed to investigate the changes in soil’s stress states during SCP installation and evaluate the SCP-induced strength enhancement in soil. The finite element analyses were conducted with the aim of modelling sand pile installation in the most realistic way feasible with current numerical technologies. As part of the validation process using a related coupled-flow problem, the build-up and dissipation of excess pore pressure during cone penetration was studied, since this problem has been investigated extensively experimentally (e.g. Randolph & Hope, 2004; Kim, 2005, Chung et al., 2006, Kim et al., 2008). The overall scope of this study encompasses the following aspects:

i) Stress and pore pressure changes in the kaolin clay during SCP installation.

ii) Post-construction dissipation of excess pore pressure and its effect on the shear strength of the soft clay around the installed SCP.

iii) Validation of coupled-flow finite element analysis with large deformation and sliding contact mechanism, using the cone penetration problem.

iv) Extension of the numerical techniques used in the cone penetration
problem to simulate the short-term and long-term effects due to SCP installation.

v) Use of centrifuge model data obtained in this study and those obtained by Juneja (2002) to benchmark numerical analyses of SCP installation process.

vi) Numerical parametric studies on the effects of various factors on the state of the surrounding soft clay during and after SCP installation, with the objective of developing a simplified procedure for defining the quantum and extent of improvement around a single SCP.
Figure 1.1  Cumulative length of sand compaction piles constructed (after Kitazume, 2005).

Figure 1.2  Execution equipment for SCP on-land construction (after Kitazume, 2005).
Figure 1.3 Execution equipment for the SCP off-shore construction (after Kitazume, 2005).

Figure 1.4 Compozer method of the SCP installation (after Aboshi & Suematsu, 1985).
Figure 1.5 Non-vibratory SCP installation (after Tsuboi et al., 2003).
Chapter 2: LITERATURE REVIEW

2.1 Introduction

As a versatile ground improvement method, the sand compaction pile (SCP) method has been widely applied in the ground improvement of soft clayey or loose sandy soil, to increase bearing capacity, reduce settlement, enhance stability and even prevent liquefaction (for loose silty or sandy deposits) in seismic areas. This chapter presents a literature review on the design principles for sand piles and sand columns and previous studies. The first part of the chapter is a review of existing design methodologies to evaluate the bearing capacity, settlement, and stability of the SCP-improved ground. The second part is a review of several representative cases of reported field investigation reported. The third and last part reviews previous experimental and numerical research works relevant to sand compaction piles.

2.2 Design methodology for the SCP-treated ground

The mechanical performance of the SCP-improved soft clay ground (or the “composite ground”) is fairly complex. It is influenced by a number of factors which, as summarized by Terashi et al. (1991b), include:

i) the shear strength of sand piles as well as the strength profile of the soft clay,

ii) the area replacement ratio “a_s” which is defined as the ratio of the area occupied by sand piles to the overall area of the improved ground,

iii) the geometric conditions such as the ratio of the width of improved area over the width of foundation,
iv) the ratio of the length of sand piles over the thickness of soft soil layer, and
v) the external loading conditions (e.g. loading rate, load eccentricity and inclination).

A number of design approaches have been proposed in the literature to evaluate the bearing capacity, settlement and stability of the composite ground. Due to the complexity of the problem, most of these methods are developed on semi-empirical or empirical basis, instead of first principle analysis. The following sections give an overview of various SCP design methodologies.

### 2.2.1 Bearing capacity evaluation

#### 2.2.1.1 Unit cell approach

When subjected to widespread load, the composite ground can be viewed as an assemblage of identical cells and analyzed using the unit cell approach. As shown in Figure 2.1, each cell is made up of a single SCP column surrounded by its “tributary” clay. The mechanical behavior of cell is assumed to be representative of the composite ground. It therefore simplifies the analysis of the whole soil domain into one soil unit. Within the unit cell, applied load is collectively taken by the stiff sand column and the soft tributary clay.

\[
F = \sigma A = \sigma_s A_s + \sigma_c A_c \tag{2.1}
\]

\[
\sigma = \sigma_s a_s + \sigma_c \left(1 - a_s \right) \tag{2.2}
\]

where \( F \) is the applied external load ,

\( \sigma \) the average loading intensity,
A, A_c, and A_s are the cross-sectional areas of unit cell, as well as those of the clay and the sand pile within the unit cell,

\( \sigma_c \) and \( \sigma_s \) the vertical stress on the clay and sand pile respectively, and

a_s the area replacement ratio, which is defined as \( A_s / A \).

Owing to the stiffness disparity, stress concentration is expected to be present in the unit cell, with more stress on the stiff sand column and less stress on the clay. The ratio of stress acting on the sand column, \( \sigma_s \), over the stress on the clay, \( \sigma_c \), is defined as stress concentration ratio, \( n \). Stress equilibrium and stability within the unit cell (Murayama, 1962; Aboshi & Suematsu, 1985) leads to:

\[
\sigma_h \geq \frac{1 - \sin \phi \sigma_s}{1 + \sin \phi \sigma_s} \sigma_s \tag{2.3}
\]

\[
\sigma_h \leq \sigma_c + \sigma_u \tag{2.4}
\]

where \( \sigma_h \) is the lateral confining stress on the cylindrical surface of sand pile,

\( \phi \) the internal friction angle of sand, and

\( \sigma_u \) the upper yield stress of clayey ground.

If one further assumes that the stress conditions of clay and sand in the unit cell are fully passive and active, respectively, the combination of above Equations 2.3 and 2.4 produces the stress concentration ratio \( n \) as follows:

\[
n = \frac{\sigma_s}{\sigma_c} = \frac{1 + \sin \phi \sigma_s}{1 - \sin \phi \sigma_s} \frac{1 + \sigma_u}{\sigma_c} \tag{2.5}
\]

The ultimate bearing capacity is therefore evaluated as:

\[
P_r = q_r A = \frac{1 + \sin \phi \sigma_s}{(n - 1) + (n + 1) \sin \phi} (A_s n + A_c) \tag{2.6}
\]
wherein $P_f$ is the maximum load on the composite ground and $q_f$ is the ultimate bearing capacity.

As Equation 2.7 indicates, the bearing capacity of composite ground can be estimated based on the soil and sand strength parameters ($\sigma_u$ and $\phi_s$) and stress concentration ratio $n$. The stress concentration ratio $n$, in the above equation, usually needs to be determined empirically or based on the field measurement data. Aboshi & Suematsu (1985) suggested a reasonable range of 4 ~ 7, from experimental data and field measurements.

### 2.2.1.2 Sliding failure approach

Another approach for ultimate bearing capacity estimation of the composite ground is based on the sliding failure mode, as depicted in Figure 2.2 (e.g. Aboshi et al., 1979; Kitazume, 2005). Again, load acting on composite ground is shared by both clay and sand pile, giving:

\[
\sigma_s = \mu_s \sigma = n\sigma / [1 + (n-1)a_s] \tag{2.8}
\]

\[
\sigma_c = \mu_c \sigma = \sigma / [1 + (n-1)a_s] \tag{2.9}
\]

in which $\mu_s$ and $\mu_c$ are the ratios of stress on sand pile and clay to the average loading intensity, respectively. The shear strength of composite ground can be estimated by taking the weighted-average of the strength of the soft soil and sand with respect to the area replacement ratio (Aboshi et al., 1979):

\[
\tau_{sc} = (1-a_s)s_s + a_s (\mu_s \sigma + \gamma_s z) \tan \phi_s \cos^2 \alpha \tag{2.10}
\]
in which $\tau_{sc}$ denotes the shear strength of the composite ground,

$\gamma_s$ the unit weight of sand pile,

$z$ the depth below surface

$\alpha$ the inclination of the failure surface measured from horizontal plane, and

$s_u$ the undrained shear strength of clay.

In cases where load is gradually applied on the composite ground in stages (e.g. the construction of embankment), Aboshi & Suematsu (1985) suggested taking into account the additional shear strength increase due to the application of surcharge as follows:

$$ s_u = s_{u-i} + (\mu_c \sigma)U(s_u / q_i) $$

(2.11)

where $s_{u-i}$ is the initial strength of the clay,

$U$ the degree of consolidation, and

$s_u/q_i$ the ratio of undrained shear strength increase due to surcharge.

Knowing the shear strength of composite ground (Equations 2.10 and 2.11), the bearing capacity the slip circle analysis can be given by (Kitazume, 2005):

$$ P_i = \frac{R_i \sum (\tau_{sc} \Delta l)}{x} \cdot \frac{1}{F_s} $$

(2.12)

where $x$ is horizontal distance of load to the centre of rotation as shown in Figure 2.2,

$\Delta l$ the arc of slip circle,

$R_i$ the radius of slip circle, and

$F_s$ the factor of safety.
2.2.1.3 General shear failure approach

In scenarios wherein the ground is improved by short end-bearing sand piles, general shear failure (as depicted in Figure 2.3) is likely to be the controlling failure mode of the composite ground. Sogabe (1981) estimated the bearing capacity of composite ground failed by general shear failure, by invoking Terzaghi's bearing capacity theory:

\[
q_f = \frac{1}{F_s} \left( \frac{1}{2} a_s \cdot B \cdot \gamma_s \cdot N_\alpha + (1 - a_s) \cdot s_u \cdot N_\beta \right)
\]

where \( B \) is the width of foundation,

\( \gamma_s \) the unit weight of sand pile, and

\( N_\alpha \) and \( N_\beta \) the bearing capacity factors for self-weight and cohesion, respectively.

Barksdale & Bachus (1983) also developed another approach to assess the bearing capacity of composite ground based on general shear failure, as illustrated in Figure 2.4. In this simplified mechanism, the failure surface is represented by two straight rupture surfaces. Analyzing the force equilibrium of the wedge formed by the two straight rupture surfaces produced the ultimate bearing capacity in the following forms:

\[
q_f = 2(1 - a_s) s_u \tan \phi + \frac{1}{2} \gamma_c B \tan^3 \phi + 2s_u \tan \phi
\]

\[
\phi = 45^\circ + \tan^{-1} \left( \frac{\mu_s a_s \tan \phi_s}{2} \right)
\]

where \( \gamma_c \) is the unit weight of clay,

\( \mu_s \) the ratio of stress on sand pile to the average loading intensity,
\( \phi_s \), angle of friction of the sand, and
\( \varphi \), the angle between the postulated failure surface and foundation as shown in Figure 2.4.

### 2.2.1.4 Bulging failure approach

For granular columns (e.g. SCP) extended to the underlying firm stratum, there is a tendency for columns to bulge and mobilize passive earth pressure from the surrounding clay (Greenwood, 1970). Bulging, or local, failure of columns may control the bearing capacity of the composite ground as shown in Figure 2.5. In such situations, Greenwood (1970) proposed the following relationship for the estimation of later earth pressure resisting the bulging column:

\[
\sigma_h = (\gamma_c z + q_t)K_p + 2s_u \sqrt{K_p}
\]

where \( K_p \) is the passive earth pressure coefficient of tributary clay and \( q_t \) is the surcharge per unit area.

If we further assume that the column is fully active, the bearing capacity of column can be calculated and expressed as follows:

\[
\sigma_s = \sigma_h \frac{1 + \sin \phi_s}{1 - \sin \phi_s}
\]

(2.17)

\[
\sigma_s = (\gamma_c zK_p + 2s_u \sqrt{K_p} + q_tK_p) \frac{1 + \sin \phi_s}{1 - \sin \phi_s}
\]

(2.18)

### 2.2.1.5 Cavity expansion approach

Using the analogy between the bulging of granular column and the expansion of
cylindrical cavity, Hughes & Withers (1974) introduced the elasto-plastic cavity expansion theory into analyzing the bulging failure of granular column. The limiting internal pressure on the expanding cavity was related to the radial stress of the soil around bulging granular column. Hughes & Withers (1974) formulated the horizontal (or radial) stress $\sigma_h$ of the soil with respect to the in-situ total horizontal stress $\sigma_{h0}$ and undrained shear strength of soil $s_u$:

$$\sigma_h = \sigma_{h0} + 4s_u \quad (2.19)$$

The ultimate bearing capacity of sand column can then be estimated by

$$\sigma_v = \frac{\sigma_h \left( 1 + \sin \phi_s \right)}{1 - \sin \phi_s} \quad (2.20)$$

Another similar solution was proposed by Brauns (1978), who re-casted the total horizontal stress $\sigma_h$ as the function of Vesic’s rigidity index $I_r$ (Vesic, 1972):

$$\sigma_h = \sigma_{h0} + (1 + \ln I_r)s_u \quad (2.21)$$

Bhandari (1983) commented that Hughes & Withers’s method may probably underestimate the ultimate load of granular columns. Hansbo (1994) suggested increasing the value of multiplier for $s_u$ in Equation 2.19 from 4 to 5 based on field data:

$$\sigma_h = \sigma_{h0} + 5s_u \quad (2.22)$$

2.2.2 Settlement analysis

2.2.2.1 Aboshi & Suematsu’s Equilibrium method
Aboshi & Suematsu (1985) proposed a simple approach (named “equilibrium method”) to estimate the (long-term) settlement of the composite ground. It was assumed that both the sand column and the surrounding clay settled by the same amount when loaded. The settlement of composite ground $S$ can be calculated as the product of original ground settlement and the settlement reduction factor:

$$S = S_0 \beta$$  \hspace{1cm} (2.23)

where $S_0$ is the original (unimproved) ground without improvement and $\beta$ is the settlement reduction factor. The settlement reduction factor $\beta$ can be deduced from the stress reduction mechanism, and the original ground settlement $S_0$ can be readily determined by using the classic one-dimensional consolidation calculation:

$$\beta = \frac{1}{1 + (n-1)a_s}$$  \hspace{1cm} (2.24)

$$S_0 = m_v \cdot \sigma \cdot H$$  \hspace{1cm} (2.25)

where $m_v$ is the coefficient of volume change of clay and $H$ is the thickness of subsoil.

**2.2.2.2 Baumann & Bauer’s Elastic method**

The elastic method for settlement analysis of composite ground were proposed and developed by Baumann & Bauer (1974) and Priebe (1976 & 1995). Baumann & Bauer (1974) assumed both intervening soil and granular columns experienced the same amount of settlement under loading. The sand column was assumed to be incompressible, deforming with no volume change. Hence the settlement or vertical
shortening of column could be directly linked with its radial deformation which could be estimated by the elastic cavity expansion theory. By doing so, Baumann & Bauer (1974) arrived at the following relation for the estimation of stress concentration:

$$\frac{\sigma_s}{\sigma_c} = \left[ 1 + \frac{E_c K_s}{E_s} \ln \left( \frac{4(A_s + A_c)}{\pi d_s^2} \right) \right]$$

(2.26)

where $K_s$ and $K_c$ are the coefficients of earth pressure in the sand column and clay, respectively,

d_s the diameter of the sand column, and

$E_s$ and $E_c$ the Young’s modulus of the sand and clay, respectively.

The ground settlement, $S$, was solved and expressed in the following correlation.

$$S = \frac{\Delta \sigma_h}{E_s} H \ln \left( \frac{4(A_s + A_c)}{\pi d_s^2} \right)$$

(2.27)

where $\Delta \sigma_h$ is the difference of lateral pressure increases between sand column and clay, i.e. $\Delta \sigma_h = \sigma_s K_s - \sigma_c K_c$.

A similar relation was derived by Priebe (1976) on the following assumptions:

i) the column extends to the underlying rigid layer;

ii) the column deforms with constant volume;

iii) the self-weight of the column and soil can be neglected;
iv) the shearing of column material takes place from the beginning while the surrounding soil reacts elastically;

v) the coefficient of earth pressure is 1.0.

Based on the theory of elasticity, Priebe obtained the following expression for the settlement reduction factor $\beta$:

$$\frac{1}{\beta} = 1 + a_1 \left[ 0.5 + f(v_c, a_s) \right]$$

$$K_{ns} = \tan^2 (45^\circ - \phi_s / 2)$$

in which $v_c$ is the Poisson’s ratio of clay. Solutions of Equations 2.28 for the Poisson’s ratio of 0.33 were plotted as a design chart (Figure 2.6). Priebe (1995) subsequently refined the preceding solutions by considering the compressibility of granular column and self-weight of clay and granular material.

### 2.2.2.3 Goughnour’s Elasto-plastic method

Goughnour (1983) correlated the bulging of granular column with the particle-to-particle slippage (or plastic deformation). It was believed that, when the composite ground is loaded, load distribution between the column and soil is governed by the complex interaction between the column and soil. On one hand, if
the vertical load is relatively small as compared with the confining pressure provided by the surrounding soil, plastic flow within the column does not occur. If, on the other hand, the load becomes sufficiently large, the strength of the column material is exceeded and particle-to-particle slippage takes place which is manifested as the column bulging (Goughnour, 1983). Since the overburden stress and radial confining pressure experienced by the column are functions of depth, the capability of granular column to sustain the load without generating the particle-to-particle slippage is also related to the depth. This implies that plastic deformation is initiated from the soil surface where the confining pressure is minimum and progressively develops downwards. To incorporate the depth-dependent characteristic, Goughnour (1983) discretized the unit cell into successive disc-shaped soil elements throughout the thickness. Each element was first analyzed by assuming the column material to be rigid-plastic and incompressible. The shortening or settlement of column was calculated by linking with the volumetric compression of tributary soil. The analysis was then repeated by assuming the column material to be linear elastic. The greater strain solution obtained from preceding plastic and elastic analysis was taken as the strain of element. The column material was deemed to yield if the plastic calculation gave the larger strain. For all elements at different depths, the above calculation procedure needed to be repeated, the sum of which produced the overall settlement of composite ground.
2.2.2.4 Van Impe & De Beer’s Granular wall method

Van Impe & De Beer (1983) studied the settlement performance of the composite ground using a granular wall method. The granular wall method replaces the discrete granular columns with the continuous wall with equivalent sectional area, as shown in Figure 2.7. The equivalent nominal thickness of the wall, \( d_r \), was calculated using the relation:

\[
d_r = \pi d_s^2 / 4a
\]  

(2.31)

in which \( a \) is the smallest center-to-center spacing. The settlement of granular wall was then solved assuming plane strain conditions. Van Impe & De Beer also neglected the shear stress between the granular wall and adjacent clay. As with preceding methods, granular wall method also assumed identical settlement experienced by both clay and granular column. Manipulating the compatibility and equilibrium equations, Van Impe & De Beer attained complex correlations for the settlement estimation which is abbreviated as follows:

\[
S = S_0 \beta
\]  

(2.32)

\[
\beta = f(a_s, \phi_s, \nu_c, \sigma, E_c)
\]  

(2.33)

While the value of settlement reduction factor \( \beta \) can be analytically calculated, it is more convenient to be expressed in graphic form as shown in Figure 2.8.
2.2.3 Stability analysis

The stability of SCP improved ground is usually evaluated by the slip circle analysis, as illustrated in Figure 2.9 (Aboshi et al., 1991; Kitazume, 2005). If Fellenius or Bishop’s slice method is used, the factor of safety $F_s$ is given:

$$F_s = \frac{R \cdot \sum (\tau_s \cdot \Delta l)}{\sum (W \cdot x)} \tag{2.34}$$

in which $W$ is the weight of soil slice. The application of slice method to the composite ground requires the proper estimation of its shear strength ($s_c$). As summarized by Kitazume (2005), there are several approaches to evaluate the shear strength ($s_c$ of the composite ground in the stability analysis, as shown in Equations 2.35 - 2.37.

$$\tau_{sc} = (1 - a_s)(s_{uo} + \xi z + \mu_c \cdot \sigma_w \cdot U \cdot s_a / p) + a_s (\mu_s \sigma_w + \gamma_z z) \tan \phi_s \cos^2 \alpha \tag{2.35}$$

$$\tau_{sc} = (1 - a_s)(s_{uo} + \xi z) + a_s \mu_s (\sigma_w + \gamma_m z) \tan \phi_s \cos^2 \alpha \tag{2.36}$$

$$\tau_{sc} = (\sigma_w + \gamma_m z) \tan \phi_m \cos^2 \alpha \tag{2.37}$$

where $s_{uo}$ is the cohesion of clay at surface,

$\xi$ the shear strength increment rate with depth,

$z$ the depth below surface,

$\sigma_w$ the vertical load intensity at ground surface due to weight of overlying soil mass,

$U$ the degree of consolidation,

$s_u/q_k$ the ratio of undrained shear strength increase due to the surcharge applied by the overlying soil mass,
\(\alpha\) the inclination of the failure surface measured from horizontal plane,

\(\gamma_m\) the average unit weight of composite ground, \(\gamma_m = \gamma_s a_s + \gamma_c (1 - a_s)\), and

\(\phi_m\) the average friction angle of composite ground, \(\phi_m = \tan^{-1} (\mu_s a_s \tan \phi_s)\).

There are similarities and differences between the preceding three approaches. As can be seen, Equations 2.35 and 2.36 base the strength of composite ground on the shear resistance components of both clay and sand. Equation 2.37, in contrast, derives the shear strength of the composite ground merely from the strength component of sand, which is therefore more applicable to cases with high replacement ratios. Equation 2.35 further presumess that clay and sand pile carry their own weights individually, but sustain the overlying soil mass collectively based on the stress concentration ratio. The strength increase arising from the gradually applied surcharge is accounted for in Equation 2.35 as well. Equation 2.36, on the other hand, does not distinguish between the internal load (self-weights of clay and sand) and external load (weight of overlying soil mass). The strength increase due to surcharge is also neglected. Kitazume (2005) highlighted the first approach (Equation 2.35) is most popular in the design practice for the on-land or offshore SCP constructions.

2.3 Research investigation on the sand compaction piling

2.3.1 Field studies

This section presents several representative cases of field investigation on the sand compaction piling. As will be described, the first two cases were carried out to
explore the behavior, that is bearing capacity, settlement and stability, of SCP-treated ground. The remaining two cases were undertaken to assess the applicability and efficiency of new SCP filling material and novel SCP construction technique.

A field investigation was conducted at Ebetsu in Hokkaido (Japan) to assess the efficacy of SCP in enhancing the slope stability of embankment (Aboshi & Suematsu, 1985; Kitazume, 2005). The ground condition of the test site was characterized with thick soft peaty clay with high water content and compressibility. The test site was divided into three parts, two of which were treated with two different ground improvement methods i.e. sand compaction pile and sand drain with steel sheet reinforcement. The third part was left untreated. Three full-scale embankments were subsequently constructed over the three parts, as shown in Figures 2.10a - c. Figure 2.11 illustrates the construction process of embankments and the corresponding settlement time histories. The embankment on the unimproved ground (denoted with “NT” in the graph) failed with crack forming in all direction and substantial horizontal deformation when its height reached 3.5m. On the other hand, embankments on the SCP improved ground (labeled “SCP” in the graph) and the sand drain treated ground (labeled “SD+RF” in the graph) were successfully constructed to a target height of 8m. The embankment settlements on the former (i.e. SCP-treated subsoil) were observed to be much less than those on the latter. In addition, the stress concentration mechanism on SCP-treated ground was examined by measuring the earth pressures acting on the sand compaction piles and adjacent clay as shown in Figure 2.12.
Yagyu et al. (1991) reported one full scale loading test which was carried out at the Maizuru Port (Japan) to evaluate the bearing capacity of the composite ground and also explore its failure mechanism under the gravity caisson load, as shown in Figure 2.13. The test site (with thick soft clay deposit strengthened by SCPs at an area replacement ratio of 25%) was loaded by several superstructures in two stages. The first stage involved subjecting the SCP-improved ground to load exerted by three concrete caissons which were partially filled by water. The resulting load intensity on the ground surface was 29.4 kPa, which was maintained for 10 months to allow for the dissipation of excess pore pressure. After that, the second stage loading was started by first replacing the water in the caisson with the nickel slag. Subsequently, three water tanks were placed on top of the caissons, into which water was rapidly pumped until the improved ground failed. Measurements included settlement, horizontal displacement, earth pressure, pore water pressures and the applied load. The test ground was failed by sliding failure as the second stage loading was built up to 103.9 kPa. A nearly circular slip surface was observed from the post-test soil investigation, which is marked in black in Figure 2.14. Based on the experimental measurements and back-figured results, Yagyu et al. (1991) recommended a stress concentration ratio of 3 for the design of SCP. Apart from the above, the influence of SCP driving and subsequent loading on the soil’s strength conditions was investigated in their tests. Yagyu et al. (1991) noticed an approximately 30% reduction in shear strength of the clay immediately after the SCP installation. Substantial strength recovery and increase was recorded in the following months. A
strength gain of approximate 80% was registered before the application of second stage load.

Kitazume et al. (1988) and Kitazume (2005) reported a field construction test to examine the suitability of copper slag sand as SCP filling material. The field test was carried out at Uno Port in Okayama Prefecture (Japan), where SCPs with an area replacement ratio of 0.7 were utilized to improve the soft alluvial clay on the front and rear of the sheet pile wall as shown in Figure 2.15. For the purpose of comparison, SCPs were constructed with two distinct filling materials namely the marine sand and the copper slag. After the SCP installation, exploratory boring with standard penetration test (SPT) was carried out to examine the strength of compacted columns of marine sand and copper slag. The SPT N-values measured from columns made of both marine sand and copper slag are plotted in Figure 2.16. As the graph suggests, the SPT N-values of the copper slag turned out to be slightly larger than or at least comparable with those of the marine sand. Field test results in combination with some laboratory data finally confirmed the applicability and effectiveness of the copper slag being an alternative SCP filling material.

Kitazume (2005) described another field investigation carried out at Matsusaka Port in Mie Prefecture (Japan) intent to assess the efficacy of a novel SCP execution method viz. non-vibratory or static SCP method. As explained in Chapter 1, the non-vibratory sand compaction pile installation, different from the conventional vibro-compaction technique, manipulates the casing pipe and compacts the sand column by means of rotation rather than vibration, with less adverse environmental
impacts to the surrounding areas. This field investigation was undertaken to evaluate its efficacy in reducing vibration and noise and ascertain its compaction effectiveness. As shown in Figure 2.17, field measurements of noise and vibration were performed at various distances (10m, 30m, 50m, 100m and 200m) away from the construction locations where the non-vibratory construction took places. The measured data clearly indicated a substantial decrease in the level of vibration and noise at various distances as compared with the vibro-compaction SCP execution. As illustrated in Figure 2.18, the post-execution standard penetration test (SPT) further revealed that the static SCP construction could yield similar ground improvement effects as the convention method.

2.3.2 Reduced-scale 1g-model tests

Apart from the preceding full-scale field tests, a good number of reduced-scale model tests were reported in the literature, which were conducted either at 1g or high-g environment. Owing to the errors in scaling the stresses between prototype and model, the results of reduced-scale model at 1g could not be readily upscaled to prototype values. Instead, they were usually utilized to illustrate the failure mechanism of granular columns and shed light on the stress concentration mechanisms within the unit cell. In this section, a brief review of 1g small-scale model tests is presented, while more detailed discussion will be given to the centrifuge small-scale model testing in the subsequent sections.
Muir Wood et al. (2000) reported small-scale model tests conducted at 1g to study the pile interaction (i.e. group effect) within the granular column group. A number of granular columns were installed inside the over-consolidated kaolin clay beds, which was subsequently loaded by rigid circular footing. It was noticed that the failure modes of granular columns in the group was closely related to their relative position in the group. Columns located underneath the center of footing tended to fail at greater depths in the form of bulging. In contrast, columns located far away from the center, towards the edge of the footing, failed mainly by shearing. This was attributed to the difference in lateral constraints at different locations within the pile group. In the central area, strong lateral constraints imposed by the surrounding piles allowed load to be transferred to great depths, leading to bulging at large depths. The weaker lateral constraint and severe displacement discontinuity in areas close to the edge explained the shear failure of the columns at these areas. Hence there was a tendency for the stiffness to increase towards the center of pile group, which, as Muir Wood et al. (2000) suggested, should be considered in the design practice. In addition, the experimental data of stress measurements also indicated the most heavily loaded columns within the group were those near the footing’s mid-radius.

Kim et al. (2004) used a modified triaxial cell (Figure 2.19) to examine the interaction between sand columns and clay in the SCP-treated ground. As illustrated in the graph, a cylindrical cell measuring 550mm in height and 500mm in diameter was built to house miniature sand piles and clay with various area replacement ratios. Uniform, flexible loading was imposed on the top surface of the improved ground via
the compressed air. Several pressure cells were placed in the sand column and clay at different locations throughout the sample’s thickness. Displacement gauges were installed at the similar locations to measure the soil movements. The tests were carried out at 1g. The stress measurement from the experiments indicated an increase of stress concentration ratio with the depth. In response to the uniform, flexible loading, sand and clay were observed to settle differently. The differential settlement could be lessened by the increasing area replacement ratio.

Kim et al. (2006) also reported another series of experiments to investigate the stress concentration mechanism within the unit cell. Figure 2.20 illustrates the test set-up, where one sand column is placed at the center of the cylindrical container, surrounded by clay. Load was applied on the top of the improved ground through the loading piston (i.e. rigid loading). Kim et al. (2006) noticed that the settlement reduction factor seemed independent of the applied stress. The stress concentration ratio was strongly related with the area replacement ratio and the relative density of sand column. In addition, the stress concentration ratios measured from the tests were comparable to the predictions.

2.3.3 Centrifuge model tests

It has been well recognized that the soil behaviour is closely related to its stress state and stress history. The results of small-scale model tests can be quantitatively correlated with the prototype behaviours only when the former is subjected to a stress
state and history similar to the latter. This can be accomplished by carrying out the small-scale model testing in the high-g field simulated by centrifugal acceleration. By subjecting the small-scale model to a high-g field, the stress level of model can be scaled to that of the prototype in a self-consistent manner.

The following sections give a comprehensive review of the previous centrifuge studies concerning the sand compaction piles. The discussions are carried out in two parts, divided by the two distinct SCP installation manners viz. the 1g SCP installation and in-flight SCP installation. In 1g SCP installation, SCP(s) are installed in the laboratory, prior to centrifuging, by either pouring or tamping sand into the pre-bored cylindrical hole (e.g. Almeida et. Al, 1985; Al-Khafaji, 1996; Al-Khafaji & Craig, 2000) or inserting frozen sand columns into the pre-formed hole (Terashi et al., 1991a & b; Kitazume et al., 1996; Rahman et al., 2000a&b; Nakamura et al., 2006). In contrast, the in-flight SCP installation models more accurately the real SCP execution procedure and forms the compacted sand columns while centrifuge is rotating (Ng et al., 1998, Lee et al, 2001 & 2004; Juneja, 2002; Ng, 2003; Daramalinggam, 2004; Weber et al., 2009).

2.3.3.1 Centrifuge testing with the simplified SCP installation

Centrifuge experiments were reported by Terashi et al. (1991a & b) to explore the bearing capacity behaviour of the soft clayey ground improved by sand compaction piles, subjected to various combinations of vertical and horizontal loads. Figure 2.21
illustrates the experimental set-up for the centrifuge testing, where the improved ground was tested at the centrifuge acceleration level of 50g. To simulate the loading condition for near-shore structures (e.g. the breakwater), the vertical load was imposed before the application of horizontal load. The vertical load component was applied in two stages: first stage drained loading provided by the self-weight of model caisson which was followed by the second stage undrained loading by rapidly lowering the water level. The horizontal load component was then applied by means of the loading jack mounted on the container (as illustrated in Figure 2.21). Figure 2.22 presents various combinations of vertical and horizontal load components measured upon the failure of SCP-improved ground, which indicated the interdependence between vertical and horizontal load capacities. It suggested that the largest horizontal load capacity was captured when the vertical load intensity was approximately half of its maximum. Back-analysis using slip circle method with an assumed stress concentration ratio of 3 (as denoted with lines in the graph) were found to agree well with the measured data. In addition, photographs were frequently taken during tests, where the displacement vector diagrams of soil under the different loading conditions could be attained as shown in Figures 2.23a and b. Terashi et al. (199b) analyzed various experimental results and postulated that yielding of the SCP-improved ground was triggered by the yielding of the sand pile.

It may be worth mentioning that model SCPs in Terashi et al’s (1991a & b) experiments were prepared and installed in the laboratory under normal gravity (1g), using the so-called “frozen pile method”. The frozen pile method, devised by
Kimura (1983), prepares and installs SCP in several steps. Sand was firstly poured into water-filled tubes, which was subsequently densified by vibration. It was then frozen to produce the frozen sand column. The frozen sand column was next demoulded from the tube, inserted into pre-formed holes in the clay bed and eventually left for gradual thawing. The entire installation process took place at 1g environment before centrifuge testing. The frozen pile method are widely used in Japan to prepare the model SCP-improved ground (Terashi et al., 1991a & b; Kitazume et al., 1996; Rahman et al., 2000a & b; Nakamura et al., 2006).

Rahman et al. (2000a & b) conducted a series of centrifuge tests to examine the stability of SCP-improved soft ground subjected to the gravity caisson and backfill loading. Their experimental set-up is schematically illustrated in Figure 2.24. The loading procedure in the tests was devised to be consistent with the realistic caisson construction procedure. Loading was conducted at 100g by first in-flight filling the empty model gravity caisson with ballast liquid (water or zinc chloride solution) to achieve the target load levels. After filling the caisson, consolidation was allowed for 10 minutes (equivalent to 10 weeks or so in the prototype scale) before the placement of backfill material. Zircon sand was rained down in-flight to the rear of gravity caisson to simulate the stage construction of backfill. The primary focus of the test was to examine the various factors influencing the stability of the improved ground loaded by the caisson and backfilled sand. These included the area replacement ratio, the SCP improvement width and the weight of caisson. It was found that widening the SCP-improved area towards the fill was effective in
improving the stability of the ground. Increasing the caisson weight could decrease the lateral displacement of caisson and also to some extent enhance the stability of the ground during backfilling. The experimental results also verified that a high area replacement ratio was conducive to reducing the settlement of the caisson, which was to be expected.

Centrifuge model tests at the acceleration levels of 105g was carried out by Al-Khafaji & Craig (2000), which were meant to examine the settlement performance of an oil storage tank foundation strengthened by the sand columns. The experimental set-up was depicted in Figure 2.25. A circular model tank with a diameter of 325mm was placed on the top of a circular foundation area (measuring 380mm in diameter), where as many as 572 model sand columns of 10mm diameter were installed. The SCP installation was performed at 1g by pouring and vibrating sand into pre-bored holes. The model tank was in-flight loaded by pumping water from the storage tank until the tank base pressure reached 40kPa. A comparison was made between the experimental observations and analysis results of Priebe (1995). It was found that the latter tended to overstate the ground improvement. One possible explanation was the lack of consideration of the three-dimensional effect in the analysis. Al-Khafaji & Craig (2000) recognized there were limitations in their model preparation where multiple sand columns were prepared and installed at 1g. This method, as they highlighted, might result in lower than ideal model stiffness.

More recently, centrifuge testing has been performed to study the mechanic behaviour of the ground strengthened by the partially penetrated SCPs, at the Port and
Airport Research institute, Japan (Nakamura et al., 2006; Takahashi et al., 2006). As commented by Nakamura et al. (2006), the partially penetrated or floating type SCP, which is not driven to the firm soil stratum but instead terminated in thick soft clay deposit, is inevitable in some scenarios, where, for instance, the depth of soft clay deposit extends beyond the capacity of SCP installer. The mechanical characteristics of the ground improved by the partially penetrated SCPs, however, are not well understood. Nakamura et al. (2006) carried out centrifuge model tests in order to study the failure pattern and deformation of partially-penetrated-SCP-improved ground subjected to caisson and backfill loading. Figure 2.26 shows their experimental set-up, which was in essence similar to the previous one reported by Rahman et al. (2000a) (Figure 2.24). A total of five experiments were conducted, which had various penetration depths of SCP (ranging from the floating to fixed type SCP) and different improved areas. It was found that the failure pattern of the improved ground was closely associated with the penetration depth of SCP. Figures 2.27a & b present the recorded displacement vectors diagrams of two soil grounds; one improved by the fixed type pile(Figure 2.27a) and the other with floating type pile (Figure 2.27b). As can be seen, the ground improved by the floating type SCP tended to fail with the slip surface passing underneath the improved area, while ground with the fixed type SCP tended to fail with large lateral deformation near the ground surface. In addition, the experimental results also showed that the bearing capacity was influenced by the SCP penetration depth in cases where it was shallow and less than the critical depth.
2.3.3.2 Centrifuge testing with in-flight SCP installation

As described in Chapter 1, in the field, SCPs are usually formed by first driving a cylindrical casing pipe into the soft ground. The casing pipe filled with sand is then repeatedly withdrawn and partially re-driven to discharge and compact sand. As soil is not removed from the ground before the SCP installation, the insertion of the casing and the subsequent injection of sand in the ground will lead to a cavity-expansion type displacement and thereby the strength set-up in soft soil (Lee et al., 2001). The “frozen pile” method and Al-Khafaji & Craig’s pluviation method are 1g “replacement” methods. Stress state changes and the strength set-up in the soft soil (i.e. the “set-up” effect explained in Chapter 1), which may occur in the field, are not captured in these 1g replacement methods. Lee et al. (1996) reported the development of an experimental device to install the model sand piles at 1g which involved displacing the surrounding soil. Further improvements were subsequently made to this apparatus so that it could install SCPs at high-g, that is while centrifuge was spinning (Ng et al., 1998; Lee et al., 2001). A detailed description of this experimental apparatus, namely the SCP in-flight installer, will be presented in Chapter 3. Using the SCP in-flight installer, several centrifuge experimental works were undertaken (Lee et al, 2001 & 2004; Juneja 2002, Ng 2003; Daramalinggam 2004).

Lee et al. (2001) and Ng (2003) reported a series of centrifuge model tests conducted to study the effects of SCP installation method on the mechanical behaviour of the composite ground. Three different SCP installation methods were
adopted and compared, namely the frozen pile method, the 1-g displacement method and the high-g displacement method (i.e. the in-flight installation). The SCP improved grounds produced by different installation methods were all tested under embankment loading. It was found that both displacement methods conferred higher strength and stiffness to the improved grounds as compared with the frozen pile method. This was attributed to the fact that the frozen pile method did not cause any displacements and increase in lateral stress onto the soil surrounding the sand column. Instead, the frozen sand pile shrunk upon thawing, which would probably lead to the reduction in lateral stress of soil thus softening the surrounding soft clay. The softened clay may even flow around the SCPs under embankment loading, which explained the wavy pattern deformation observed in the experiment (Lee et al., 2001). On the other hand, the displacement methods, in particular the in-flight installation, caused cavity expansion displacements to the surrounding clay, which gave rise to the set-up effect and led to strength increase and integrity enhancement of the improved ground. In addition, Lee et al. (2001) also postulated that lock-in stress in the SCPs installed by displacement methods also contributed to the high strength and stiffness of the improved ground.

Daramalinggam (2004) conducted a research study which examined the bearing capacity increase induced by the SCPs installation. Soil samples made of the remoulded Singapore marine clay and strengthened by SCPs at various replacement ratios were loaded by model gravity caisson (Figure 2.28). During tests, SCPs were installed in-flight and loading was also applied in-flight by in-filling the
model caisson with zinc chloride solution. The experimental observations verified the effectiveness of SCPs in improving the soft ground by increasing its bearing capacity and reducing its settlement. It was also concluded that, apart from the strength increase in clay, design methodology should also consider the strength increase of the sand pile itself due to the increase in confining stress in the surrounding clay. If this is not accounted for, the ground improvement effect is still probably underestimated.

To examine the stress state changes in soil during SCP installation, Juneja (2002) and Lee et al. (2004) carried out a series of centrifuge model tests at 70g model gravity. The model SCPs were installed in-flight in soft Singapore marine clay. Pore pressure and total radial stress were measured in the vicinity of the pile. The measured changes in stress and pore pressure were compared with Vesic’s (1972) plane strain cavity expansion predictions which suggested that the former could be well predicted at large depths. Nevertheless, substantial deviation was noticed at shallow depths, as shown in Figure 2.29. The plane strain cavity expansion calculation tended to overestimate the pore pressure and total stress build-up near surface. The observed discrepancy was attributed to the free surface effect (or soil heave near surface), which rendered the plane strain assumption invalid at shallow depths. To cater for the influence of the free surface, Lee et al. (2004) proposed a modified cavity expansion theory, which was developed by merging two limit situations namely the plane strain and plane stress situations. By fitting the experimental data, a semi-empirical correlation was established which took into
account the vertical soil movement as well as the resulting volume loss at shallow depths. As Figure 2.30 shows, the modified semi-empirical cavity expansion theory yields close predictions of total stress and pore pressure increase as compared with the experimental results. Lee et al. (2004) also pointed out that the cumulative total stress incensement arising from multiple piles in a grid might be reasonably estimated by superimposing the increment induced by each pile installation. Nonetheless, this superposition rule may not be applicable to the cumulative pore pressure increments. One likely reason is that the shear-induced component of excess pore pressure does not increase linearly with deviator stress.

2.3.4 Numerical analysis

In parallel with field investigation and laboratorial experimental study, numerical works were also undertaken to elucidate the mechanic behaviour of SCP-improved ground as well as the complex mechanisms involved in the SCP installation.

Asaoka et al. (1994a) analyzed the bearing capacity behaviour of SCP-improved soft ground using a limit analysis that took into account soil-water coupling. Their study was focused primarily on the various factors influencing the bearing capacity of the composite ground including the effect of the stiffness and roughness of the footing, the drainage condition of sand during loading and the consolidation situation of clay. They found that when the composite ground supported a rigid-rough footing, drained condition of sand produced more significant
stress concentration on sand piles yielding greater bearing capacity. By contrast, when the embankment-like flexible load was considered, the undrained condition of sand generated higher bearing capacity than the drained situations.

Finite element study was carried out by Takahashi et al. (2006) to investigate the stability of the SCP-improved ground subjected to the caisson and backfill loading. One objective of their study was to explore the failure mechanism of soft ground improved by partially penetrated (or floating type) piles. Numerical analysis results indicated that the failure pattern of the improved ground under backfill loading was closely related with the penetration depth of piles. In situations with fully penetrated (or fixed type) piles, the improved ground tended to fail with bending of sand piles, as depicted in Figure 2.31a. On the other hand, in the case of partially penetrated piles (Figure 2.31b), the failure mode changed to slip failure as the penetration depth became less than a certain value i.e. the critical depth. The critical depth in their calculation conditions was around 9m based on the numerical results.

Most reported numerical analyses are concerned with the mechanical performance, failure mode and stress concentration mechanism of SCP treated ground, two examples of which were described above. SCPs in the numerical model were invariably pre-placed before the start of simulation. In other words, the installation process of SCP itself was not considered in the modelling. There are a few exceptions, where the SCP execution was modelled by various means. The prime interest of these analyses was to assess the impact of SCP installation to the surrounding soil.
Farias et al. (2005) reported a finite element analysis performed to evaluate the extent and effectiveness of densification induced by the SCP installation in the granular soil. The construction process of SCP was simulated sequentially as shown in Figure 2.32. In consistency with the field SCP installation, Farias et al. (2005) modeled the execution procedure of SCP in two stages namely “casing driving” and “compaction”. At the first stage (casing driving), uniform vertical displacements were imposed to the nodes at the bottom of the casing, while the nodes at the periphery were constrained in the horizontal direction (Figure 2.32a). The magnitudes of vertical nodal displacements were specified to be equal to the height of the corresponding elements, and had to be incremented in small steps. Element inside the casing should be progressively deactivated on step-by-step basis. The above procedures was carried on until the pre-determined the depth was reached. What followed was the second stage: the compaction of sand. Element deactivated in the first stage would be sequentially re-activated from bottom upwards, which corresponded to the withdrawal of casing. The re-activation was performed layer by layer. Following the reactivation of certain layer of elements, vertical force was immediately applied to the top nodes of the reactivated element layer to simulate the hammer load. The applied vertical force was removed before the calculation proceeded to next layer. The above procedure was repeated until the ground surface was reached. The numerical analysis revealed that the densification area due to pile installation could spread to five diameters both from the pile centerline and pile tip. Farias et al. (2005) also noticed that the densification effectiveness was closely linked
with the initial condition of soil. The looser the initial soil was, the more substantial was the densification (Farias et al., 2005).

Guetif et al. (2007) idealized the vibro-compaction of granular column as the expansion of a cylindrical cavity within soft clay. A numerical procedure, named “dummy material” was developed to simulate the column construction. As shown in Figure 2.33, a cylindrical cavity of 0.25m radius was first introduced along axis of symmetry, where a dummy material, fictitious and soft elastic material, was assigned. The pre-existing cavity filled with weak dummy material was meant to represent the presence of vibro-probe in soil before the commencement of vibro-compaction. Subsequently, the periphery of cylindrically cavity was expanded radially outwards until it reached the target column radius of 0.55m. The preceding expansion of cavity modelled the formation of granular columns. Following the expansion, the dummy material in the cavity was replaced with the real column material, which was followed by the post-installation consolidation analysis. The axisymmetric numerical modeling was conducted using the commercial software Plaxis, with 15-noded triangular finite elements. Both soft clay and column material was approximated by the elastic-perfectly plastic Mohr-Coulomb model. The main objective of their analysis was to estimate the improvement of soil’s Young modulus due to the installation of vibro-compacted granular column. Numerical results indicated that soil improvement took place within the radial distance up to six times the column radius.
Won (2002) reported an attempt to numerically simulate the complete SCP installation in a rational manner. His initial intention was to carry out a coupled consolidation analysis on the platform of ABAQUS/Standard. It was later given up due to a great deal of numerical difficulties. Instead, a total stress formulation in ABAQUS/Explicit was used together with contact modelling and adaptive meshing. The simulation considered both the stages of casing penetration and SCP formation. The casing was driven into the soil; the soil-casing interaction being accounted for by the contact algorithm. The casing was modelled as a deformable body, which was subsequently enlarged at the second stage to simulate the SCP formation. Won’s (2002) research attempts demonstrated the potential of various numerical tools, many of which were used in this study, as described in a later chapter. However, difficulties encountered by Won (2002) reflected the significant challenges that needed to be overcome in modelling a problem like SCP installation, which involved large deformation and sliding contact mechanism with effective stress formulation and coupled flow.

2.4 Knowledge gaps and outstanding issues

As described in Chapter 1, the phenomenon of “set-up” in clay due to SCP installation has been verified by numerous field measurements (Enokido et al., 1973; Aboshi et al., 1979; Yagyu et al., 1991; Matsuda et al., 1997). Nevertheless, the post-installation strength build-up in clay is not accounted for in the aforementioned design methodologies (e.g. unit cell approach, sliding failure approach, general shear failure
approach, etc). To date, there exist no guidelines on the extent and magnitude of the improvement in strength, if any, in the surrounding soft ground. One important reason is that there remain some uncertainties about the governing mechanism of set-up effect and significance of its resulting strength increase, which actually drove a lot of research efforts in both experimental and numerical fields.

Set-up cannot be examined in geotechnical centrifuge experiments wherein sand compaction pile models are prepared by frozen pile method (Kitazume et al., 1996; Rahman et al., 2000a & b; Nakamura et al., 2006) or pouring or tamping sand into the pre-bored cylindrical hole (e.g. Almeida et. al 1985; Al-Khafaji, 1996; Al-Khafaji & Craig, 2000). Up to the present, there have been only a few centrifuge experiments (Lee et al., 2001 & 2004; Juneja, 2002; Ng, 2003) which investigated the set-up effect by carrying out in-flight SCP installation. As discussed earlier, these tests focused mainly on the stress and pore pressure changes during the SCP installation, which is one aspect of the set-up phenomenon. There is another important aspect i.e. soil’s strength change due to the SCP installation that needs to be explored. In addition, previous tests were invariably conducted with Singapore marine clay. As the set-up effect is presumably associated with the sensitivity of the clay to remoulding (Juneja, 2002), there is need to further examine it in other types of soft soil.

It is expected that numerical modelling can provide reliable and versatile platform to systematically and comprehensively examine the set-up effect. However, previous attempts were frustrated by the difficulties encountered in simulating
complete SCP construction process in a coupled consolidation analysis (Won, 2002). Difficulties arise out of three major sources: i) large deformation and strain of soil in the course of casing insertion and subsequent sand column formation; ii) material nonlinearity due to the elasto-plastic behaviour of soil; iii) complex interfacial behaviour which exists between soil-casing as well as soil-SCP interaction. Given the complexity of the problem, the modelling of SCP construction reported in the literature was simplified in various approaches such as removing and re-activating elements (Farias et al., 2005) and adopting “Dummy material” numerical procedure (Guetif et al., 2007). However, these approaches cannot capture many of the salient features involved in SCP installation such as soil-casing and soil-SCP interaction, progressive top-down casing penetration and bottom-up SCP formation.

This thesis deals specifically with the improvement in strength in SCP-improved ground under various conditions. The study was undertaken using both centrifuge and numerical modelling. The centrifuge model tests and results described in Chapters 3 and 4 were designed to explore the relation between pore pressure generation and dissipation and the consequent strength enhancement. These centrifuge results, together with those obtained by Juneja (2002) will be used in benchmarking the numerical analysis. The techniques and validation of the numerical analysis using the problem of a penetrometer as a precursor to the SCP problem, are discussed in Chapter 5. The comparison of the numerical results with centrifuge data and further parametric studies on the SCP-induced strength
improvement are presented in Chapters 6. Final conclusions are summarized in Chapter 7.
Figure 2.1 Unit cell concept (after Aboshi & Suematsu, 1985).

Figure 2.2 Sliding failure of the composite ground (after Kitazume, 2005).
Figure 2.3  General shear failure of the composite ground (after Kitazume, 2005).

Figure 2.4  Shear failure of the composite ground (after Barksdale & Bachus, 1983).

Figure 2.5  Bulging failure of the composite ground (after Greenwood, 1970).
Figure 2.6 Design chart for estimation of settlement improvement (after Priebe, 1995).

Figure 2.7 Replacement of granular columns with continuous walls (after Van Impe & De Beer, 1983).
Figure 2.8  Design chart for estimation of settlement reduction (after Van Impe & De Beer 1983).

Figure 2.9  Slope stability analysis of the composite ground (after Aboshi et al., 1991).
Figure 2.10  Cross-section of embankment founded on (a) untreated ground; (b) ground treated by the sand drain with steel sheet reinforcement; (c) ground treated by the sand compaction pile (after Aboshi & Suematsu 1985).
Figure 2.11 Settlement histories of embankments (after Aboshi & Suematsu, 1985).

Figure 2.12 Vertical earth pressures measured in the SCP-improved ground (after Aboshi & Suematsu, 1985).
Figure 2.13 Set-up of full scale test at the Maizuru Port (after Yagyu et al., 1991).

Figure 2.14 Failure surface obtained from the post-test investigation (after Yagyu et al., 1991).
Figure 2.15  Sectional view of construction site at the Uno Port (after Kitazume, 2005).

Figure 2.16  SPT N-value with depth (after Kitazume, 2005).
Figure 2.17  Vibration and noise levels at various distances away from the construction site: (a) vibration level; (b) noise level (after Kitazume, 2005).

Figure 2.18  Comparison of vibro and non-vibro method in terms of compaction efficacy (after Kitazume, 2005).
Figure 2.19  Schematic illustration of experimental set-up (after Kim et al., 2004).

Figure 2.20  Unit cell test device (after Kim et al., 2006).
Figure 2.21  Schematic illustration of centrifuge experimental set-up (after Terashi et al., 1991a).

Figure 2.22  The combination of horizontal and vertical loads upon failure (after Terashi et al., 1991b).
Figure 2.23  Displacement vectors of the SCP-improved ground under (a) vertical loading; (b) inclined loading conditions (after Terashi et al., 1991a).

Figure 2.24  Schematic illustration of centrifuge experimental set-up (after Rahman et al., 2000a).
Figure 2.25  Schematic illustration of centrifuge experimental set-up (after Al-Khafaji & Craig, 2000).

Figure 2.26  Centrifuge experimental set-up: (a) photo; (b) schematic illustration (after Nakamura et al., 2006).

Figure 2.27  Displacement vectors of the ground improved by (a) fixed type SCPs; (b) floating type SCPs (after Nakamura et al., 2006).
Figure 2.28 Schematic illustration of centrifuge experimental set-up (after Daramalinggam, 2004).

Figure 2.29 Variation of measured to calculated increase in total horizontal stress and excess pore pressure ratio (post-installation) (after Lee et al., 2004).
Figure 2.30  Variation of measured stress and excess pore pressure against calculated values inferred from modified cavity expansion theory: (a) peak jack-in; (b) post-installation (after Lee et al., 2004).

Figure 2.31  Failure behavior of ground improved by (a) fixed of SCPs; (b) floating type of SCPs (after Takahashi et. al., 2006).
Figure 2.32  Idealized SCP installation process for FEM implementation (after Farias et al. 2005).

Figure 2.33  Numerical procedure named “Dummy material” for column installation: (a) model of improved soil; (b) modelling column expansion; (c) discretized improved soil (after Guetif et al. 2007).
Chapter 3: CENTRIFUGE EXPERIMENTAL PROCEDURE

3.1 Fundamentals of centrifuge modelling

It is well recognized that the mechanical behaviour of soil is a function of its stress history and current stress levels. Hence, maintaining stress similarity between model and prototype is a crucial consideration when carrying out physical modelling of geotechnical activities. The inherent limitation of conventional small-scale model tests performed under 1g condition is the incorrect in-situ stress field in the model domain, which may be much lower than that in the prototype. This discrepancy may be minimized using the geotechnical centrifuge, which undertakes the small-scale modelling of static and dynamic soil-related activities in an elevated acceleration field to achieve stress similarity with the prototype.

In the geotechnical centrifuge, soil models placed at the end of a centrifuge arm are accelerated and exposed to an inertial radial acceleration field, which is similar to a gravitational acceleration field but of much higher magnitude (Taylor, 1995). The inertial acceleration field in the centrifuge can be derived as follows:

\[ a_c = N \cdot g = \omega^2 r \]  \hspace{1cm} (3.1)

where \( a_c \) is the centrifuge acceleration level, customarily expressed as \( N \) times the Earth’s gravity \( g \), with \( N \) being the gravity scale factor,

\( \omega \) the angular rotational speed of centrifuge, and

\( r \) the radius to any element in the soil model.
Subjecting a 1/N-th scale soil model to a centrifuge acceleration field of N g can then produce a vertical stress field in the model similar to what is present in prototype.

Table 3.1 summarizes the principal scaling rules to correlate the model performance with the prototype behaviour. They are developed via dimensional analysis and well documented in the literature (Schofield, 1980 & 1988; Taylor, 1995).

3.2 Centrifuge experimental set-up

The present centrifuge model testing was carried out at the National University of Singapore (NUS) Geotechnical Centrifuge Laboratory. The geotechnical centrifuge in NUS is a rotational-arm type with two swing platforms symmetrically hung at the ends of both arms. It has a radius of approximately 2 m (after the swing platform is swung up), a capacity of 40 g-tonnes and a maximum centrifugal acceleration of 200g. The working area of the swing platform is approximately 750 mm x 700 mm, and the maximum allowable model headroom is 1295mm. More details concerning the NUS geotechnical centrifuge can be found in Lee et al. (1991).

Figure 3.1 shows the centrifuge experimental set-up of this study. The present sand compaction pile (SCP) installations utilize the “in-flight SCP installer” to install model SCPs during flight. Developed by Ng et al. (1998), this system has been acknowledged as an advanced and appropriate modelling apparatus for SCP research (Al-Khafaji & Craig, 2000; Weber et al., 2005). As part of this study,
further improvements were made to optimize its performance and enhance its stability. Its capabilities were further enhanced with the addition of an in-flight strength acquisition tool, namely the T-bar penetrometer, which enables the evaluation of strength changes induced by SCP installation.

In the following sections, the in-flight SCP installer will first be briefly described, as more detailed information is available in Ng et al. (1998). The improvement works to the installation system undertaken in this study will be highlighted afterwards.

3.2.1 NUS in-flight SCP installation system

The in-flight SCP installer is made up of three main components: (a) the hydraulic system for the sand delivery and injection, (b) the stepper-motor-driven carriage (X-Y table) for in-flight shifting of the installer, and (c) the control/power supply system for controlling and powering the stepper motor on the X-Y table.

With reference to Figure 3.1 and 3.2, the hydraulic system consists of i) a 1.5 horsepower (HP) on-board hydraulic power pack which is mounted on the centrifuge arm to drive a miniature hydraulic motor, ii) a miniature hydraulic motor with a maximum rotation speed of 5000 rpm which drives an Archimedes’ screw (Figure 3.2), iii) an Archimedes’ screw slotted inside the hopper-casing assembly which rotates to deliver and inject sand into the soil during operation, and iv) a hydraulic cylinder for pushing down and retracting the hopper-casing assembly (Ng, 2003).
During the test, the rotating Archimedes’ screw discharges sand from the casing-hopper assembly and injects it into the soil deposit to form the sand column.

The preceding hopper-casing assembly, together with the hydraulic cylinder, are mounted onto the moveable stepper-motor-driven carriage (X-Y table). Driven by a pair of stepper motors, the X-Y table can freely travel in the horizontal plane within the available range for in-flight positioning of the hopper-casing assembly to the target locations. The movement of the X-Y table in the horizontal plane is tracked by a pair of potentiometers which operate in two horizontally, mutually perpendicular directions (i.e. X and Y directions). The power supply/control system provides the DC power and control signals required for operating the stepper motors. It includes two DC power packs and one signal generation unit.

3.2.2 Further modification to the in-flight SCP installer

At the start of the present study, some problems were encountered during the operation of the existing in-flight installation system. To resolve these problems and improve the performance of the in-flight SCP installer, several modifications and improvement were made to the system, as described below:

i) The previous design of the Archimedes’ screw gave rise to several problems which affected the smooth conduct of the tests and the reliability of the results. For one, tremendous heat was generated during its rotation, as sand was ground against the steel casing tube by the screw. This heat resulted in some
drying of the clay around the casing, which sometimes led to cracking of the clay bed (Ng, 2003). In addition, the significant friction between the sand, screw and casing also caused occasional jamming of the Archimedes’ screw during SCP installation, which could affect the integrity of the installed pile (Daramalinggam, 2004). Besides, the Archimedes’ screw itself also wore out rapidly due to the severe friction, and hence required frequent replacements. When many piles had to be installed in one single test, the rapid wearing of the screw made it challenging to ensure a uniform quality of SCPs.

To resolve the aforementioned problems, several remedial measures had been attempted in the past. Ng (2003) made use of a miniature water-spraying system to quench the heated casing and surrounding soil during the installation, so as to forestall cracking of the clay bed. Daramalinggam (2004) suggested reversing the direction of screw rotation periodically to prevent the jamming. However, all these precautionary measures could not fully eradicate the problems, since they did not deal directly with the root cause - the Archimedes’ screw. In the present study, a miniature auger-like screw was designed to replace the Archimedes’ screw. The thread of this miniature auger, as shown on Figure 3.3, is different from that of Archimedes’ screw. It can feed sand at a much higher rate and higher energy efficiency, with substantially less friction generated. As a result, the friction-induced heat is very much lower compared to the earlier installer. More importantly, the
likelihood of jamming is very much reduced, and there is no need for frequent screw replacement due to frictional wear and tear. These features lead to improved integrity and quality of the installed SCPs.

ii) As mentioned earlier, the control signal and power for the stepper motors are provided by a control/power supply system. The previous control system comprised a digital-analogue-converter and feedback control software. While this computer-aided control system had the capability of positioning the X-Y installer with a high level of accuracy, the control software would occasionally fail to send out any control signals to the stepper motors. This likely occurred due to the degradation of the feedback signals from the motors. As the control system was located in the control room outside the centrifuge enclosure, any communication between the software and the stepper motors had to pass through the centrifuge slip rings. The quality of the feedback signals received by the software might have been degraded after the centrifuge had been in continuous operation for several hours, due to carbon deposition on the slip rings. To address this problem, a new signal generator was designed and assembled, as shown on Figure 3.4. This compact hand controller replaced the previous computer-based control system and fully catered for the functions of the previous system, without the need for any feedback signals.
3.3 Centrifuge experimental procedure

The experimental procedure in this study consisted of three phases: (i) sample preparation, (ii) in-flight SCP installation under a 50-g acceleration field, and (iii) post-installation in-flight strength profiling. The following sections provide the details of these three stages.

3.3.1 Sample preparation

The soil model was housed in a strong box measuring 520mm in length, 260mm in width and 250mm in depth. A 15 mm thick sand layer was placed at the base of the strong box, in which two perforated tubes were embedded to facilitate drainage during soil consolidation. The top of this sand layer was covered by a sheet of geotextile which served as a filter between the soil and underlying sand.

The soft clay used in this study was constituted from remoulded Malaysian kaolin clay. The physical properties of Malaysian kaolin clay has been reported by Goh (2003) and described in Table 3.2. The soil specimen was prepared from kaolin clay slurry, which was obtained by mixing and remoulding kaolin clay at a water content of 1.5 times the liquid limit, corresponding to a water content of about 120%. The mixing was carried out in a vacuum chamber to remove any air bubble and to achieve full saturation. The de-aired clay slurry was slowly poured into the strongbox, the inner walls of which were greased to minimize friction.
of clay slurry was performed under a layer of water to prevent air trapped during operation.

The clay slurry in the strong box was then allowed to consolidate on the laboratory floor under its self-weight, followed by the application of an overburden surcharge (Figure 3.5). The overburden surcharge was applied via the progressive placement of dead weights up to about 6.5 kPa. The 1-g consolidation took about 5 days to complete, upon which the strong box was opened on the rear side. De-aired pore pressure transducers (PPTs) and waterproofed total stress transducers (TSTs) were inserted through the side of the clay bed at prescribed locations. The container was then reassembled and transferred to the centrifuge for self-weight consolidation under a 50-g gravitational field. It was decided to place the transducers in the soil before, rather than after, the high-g consolidation so as to minimize disturbance to the consolidated soil sample. Furthermore, it was likely that any strength loss experienced by the soil due to transducer placement would be recovered during the high-g consolidation process.

During the 50-g self-weight consolidation, the readings from the pore pressure transducers embedded in the clay were used to monitor the dissipation of excess pore pressures. It took approximately 6 hours to achieve an average degree of consolidation of 95%.

The above consolidation sequence, involving both 1-g and 50-g centrifugal acceleration field, eventually produced a 100mm thick soil sample comprising a top
20 mm layer of overconsolidated crust overlying 80 mm of normally consolidated clay.

### 3.3.2 SCP In-flight installation

Upon completing the 50-g self-weight consolidation, the centrifuge was spun down for mounting the in-flight SCP installation system. The X-Y table was bolted onto the strongbox and was then used to position the hopper-casing assembly to a pre-determined reference point on the clay surface whose X and Y coordinates were measured by the pair of potentiometers. The subsequent determination of SCP positions during the test was made relative to the coordinates of this reference point.

Silica sand ($D_{50}=0.68$ mm) was added to the sand storage unit on the X-Y table. After mounting the in-flight SCP installation system, the centrifuge was swung up to 50-g again. Prior to the commencement of SCP installation, the soil specimen was re-consolidated for about 4 hours until the excess pore pressures were almost fully dissipated.

The SCP installation was then carried out in two phases: casing jack-in and casing withdrawal. Using the X-Y table, the casing was first moved to its target installation position above the soil surface. It was then jacked into the clay at a rate of approximately 10.5 mm/s. Simultaneously, the auger slotted inside the hopper-casing assembly was triggered to rotate, thus discharging a small amount of sand into the clay to prevent the latter from clogging the rapidly advancing casing.
Upon reaching the bottom of the clay bed, the hopper-casing assembly was immediately withdrawn at a rate of about 7.7 mm/s. During casing retraction, sand was still being continually injected into the clay from the casing tip by the rotating auger. (The withdrawal rate of casing was calibrated a priori to yield a SCP of diameter 20 mm, as shown in Figure 3.6.) The installation ended with the hopper-casing assembly being fully withdrawn out of the soil deposit. Overall, the average time taken for the installation of one pile was approximately 23s. In experiments involving multiple piles installation, the above jack-in and retraction procedure was repeated for the hopper-casing assembly at different locations along the soil surface, as controlled by the X-Y table.

3.3.3 In-flight shear strength profiling

In this study, strength changes in the soil due to SCP installation were evaluated using the in-flight undrained shear strength profiling test. Strength measurements were obtained during flight by means of a T-bar penetrometer. As depicted on Figure 3.1, the T-bar penetrometer, the details of which will be described shortly, was also attached onto the movable X-Y table. It could therefore be freely moved and positioned to any desired location for strength acquisition. Upon reaching the target position, the T-bar peneterometer was pushed into the clay bed at a constant rate of about 3 mm/s by a hydraulic cylinder. During its penetration, the resistance acting on the bar was continuously measured. By means of a calibration constant, the
measured resistances were then converted to yield the undrained shear strength profile of the soil.

3.4 Instrumentation in centrifuge testing

Four types of instrumentation were adopted in this experimental study. These are: (i) linear-motion potentiometers, (ii) pore pressure transducers (PPT), total stress transducers (TST), and (iv) T-bar penetrometer. Details of the instruments are provided in the following subsections.

3.4.1 Linear-motion potentiometer

A pair of linear-motion potentiometers, with stroke lengths of 200mm (Model: Midori LP-200F) and 250mm (Model: Midori LP-250F) respectively, were installed on the X-Y table to measure its X and Y coordinates. In addition, the jack-in and retraction of the casing was monitored by another potentiometer (Model: Midori LP-300F) with a stroke length of 300mm aligned in the vertical direction. Similarly, the motion of the T-bar penetrometer was monitored by another 300mm-stroke potentiometer.

3.4.2 Pore pressure transducer (PPT)

Pore pressure generation and dissipation in the tests were measured by pore pressure transducers (PPTs). The Druck PDCR 81 model transducers were used in this study, with a sensitivity of around 0.008kPa. These instruments were periodically
calibrated using the Druck Digital Pressure indicator. Before installation, care was taken to fully saturate the porous stones connected to each transducer, by applying vacuum suction over its tip. This minimized the possibility of air bubbles trapped inside the transducer tip, which might adversely affect its reading.

### 3.4.3 Total stress transducer (TST)

To measure total stresses inside the soil specimens, miniature total stress transducers (TSTs) were utilized in this study. The Entran miniature flat-line EPL-D12 transducers were used, as shown on Figure 3.7. The performance of miniature total stress transducers in soft clay under centrifuge environment was studied by Lee et al. (2002). It was found that the complex interaction between the soil and transducer’s active diaphragm made it challenging to measure the normal stress accurately. Also, particular attention should be paid to the transducer calibration, which should be performed in an environment close to the actual testing situation.

In this research, calibration of the total stress transducers was undertaken in normally consolidated kaolin clay, within the high-g centrifuge environment. The experimental set-up for the calibration exercise is schematically shown on Figure 3.8. Each total stress transducer was placed near the bottom of a large cylindrical container of diameter 500mm. The use of a large-diameter cylindrical container minimized boundary effects from the wall (Lee et al., 2002). Prior to its placement, the stress transducer was waterproofed by a thin (approximately 0.3mm) silicone rubber coating.
Its rear side was also mounted on a 6mm wide x 12mm long x 0.4mm thick aluminum backing plate. According to Juneja (2003), the use of a small backing plate helped improve the performance of a total stress transducer embedded in clay.

During calibration, the active diaphragm of the transducer was oriented to face upward, in order to measure the vertical normal stress i.e. overburden stress. As shown on Figure 3.8, the transducer rested on a 20mm thick drainage sand layer. De-aired kaolin slurry with a water content of 120% was then poured into the container above the drainage sand layer. During the pour, a pore pressure transducer was installed roughly at mid-depth of the kaolin clay layer to monitor consolidation progress during the test. When pouring was completed, a layer of water was added above the clay surface. It should be noted that the mass of kaolin slurry, as well as the overlying water, was pre-determined prior to the test, and were used to estimate the overburden stress acting on the total stress transducer during the test.

The effects of loading and unloading in the calibration test were achieved via in-flight stepwise changes in the g-level applied to the centrifuge soil model. Following the procedure proposed by Lee et al. (2002), the calibration test was undertaken as follows:

i) Swing centrifuge up to 50g to allow the soil specimen to consolidate to 95% consolidation.

ii) Reduce the g-level in five stages from 50g to 40g to 30g to 20g to 10g to 1g, each of which was maintained for around 2 minutes.
iii) Increase the g-level in five stages from 1g to 10g to 20g to 30g to 40g to 50g, each of which was maintained for 2 minutes.

iv) Reduce the g-level in one step to 1g, which was then followed by increasing the g-level in five steps from 1g to 10g to 20g to 30g to 40g to 50g again. Each increment lasted about 5 minutes.

With the pre-determined mass of kaolin slurry plus water, and the known cross-sectional area of container, the overburden stress acting on the total stress transducer at any given g-level could be readily calculated to obtain the applied stress on the transducer. Figure 3.9 presents the measured total stress against the applied stress during the prescribed loading and unloading events. The measured stress was calculated from the signal output of the stress transducer by using its fluid calibration factor. As shown on the figure, the measured stress were somewhat higher than the applied values, especially at high stress levels. If the ratio of the measured stress to the applied stress is defined as the registration ratio R (Weiler & Kulhawy, 1982), the overall registration ratio R of the transducer shown on Figure 3.9 would be approximately 1.03. Furthermore, comparing the stress measurements during the loading and unloading phases revealed some hysteresis between the loading and unloading paths. Nevertheless, considering that the response of a total stress transducer in soil may be affected by many factors such as temperature, soil type, effective stress and pore pressure level, etc, the overall performance of the total stress transducer (Entran EPL-D12) shown on Figure 3.9 was felt to be quite reasonable. The over-registration was not excessive, and hence was unlikely to significantly affect
the accuracy of stress measurements. The repeatability, despite some hysteresis, was also quite satisfactory, thus assuring the consistency of test results.

Apart from the vertical stress calibration, the performance of the total stress transducer subject to lateral loading in soil was evaluated as well. The experimental arrangement was analogous to the preceding vertical tests, except that the stress cell was oriented to face the horizontal direction for measuring the lateral stress. As before, the centrifugal g-level was accelerated in steps of 10g up to 50g, while the duration of each increment was protracted to 1.5 hours to achieve at least 90% degree of consolidation. Figure 3.10 presents the measured stresses plotted against the applied lateral stresses. The applied lateral stresses were derived from the overburden stress multiplied by the at-rest earth pressure coefficient $K_0$. The $K_0$ coefficient in this study was estimated using Jaky’s (1944) formula for a normally-consolidated soil. For kaolin clay with a friction angle $\phi$ of 23° (Goh, 2003), this yields $K_0 = 1 - \sin \phi \approx 0.6$. As can be seen in Figure 3.10, the measured stresses compared favorably with the applied stresses, with a registration ratio of about 0.96. This was slightly lower than that obtained from the vertical calibration test, but still close to unity.

### 3.4.4 T-bar penetrometer

As described earlier, the T-bar penetrometer was utilized to acquire soil strength data before and after SCP installation. The T-bar penetrometer, developed by Stewart &
Randolph (1991), is a useful site investigation tool, particularly suited for centrifuge testing. In comparison with conventional in-situ strength devices such as the vane shear apparatus, the bar penetrometer has the advantage that it gives continuous and direct measure of the undrained shear strength (Stewart & Randolph, 1991). Figure 3.11 shows the T-bar penetrometer adopted in the present study. It is made up of a cylindrical cross bar (5mm diameter and 25mm long) which connects perpendicularly with a vertical shaft forming a “T” shape. The vertical shaft is tapered down near the joint to minimize the projected area on the cross bar and facilitate soil flow around the bar during penetration. A highly sensitive load cell is attached to the end of the vertical shaft right behind the cross bar to measure the penetration resistance acting on the bar.

The bar penetrometer can be categorized as a “full-flow” probe (Einav & Randolph, 2005), that is, during penetration, soil flows around the bar surface and close behind the bar. Full-flow penetrometers like the T-bar are characterized by a well-defined failure mechanism and, more importantly, a sound theoretical basis to correlate the measured penetration resistance with the shear strength of soil. Stewart & Randolph (1991) evaluated the penetration resistance of T-bars using the plasticity solution for the limiting pressure on an infinitely long cylinder moving laterally through cohesive soil. The plasticity solution (Randolph & Houlsby, 1984) relating the limiting force acting on a cylinder to the undrained shear strength of soil is as follows:
\[
\frac{P}{s_u d_b} = N_b
\]  

(3.2)

where \(P\) is the force per unit length acting on the cylinder,  

\(s_u\) the undrained shear strength of soil,  

\(d_b\) the bar diameter, and  

\(N_b\) the bar factor.

The bar factor \(N_b\) varies within a small range from 9 to 12, as the roughness of the cylindrical surface changes from perfectly smooth to completely rough. Following Stewart & Randolph (1991), an intermediately value of 10.5 was adopted in the present study, which was also recommended by Randolph & Houlsby (1984) for general use. Given the small variation in the bar factor, the error resulting from the adopted value of 10.5 is typically less than 13% as calculated by Stewart & Randolph (1991).

### 3.4.5 Data acquisition systems for instruments

Analogue signal outputs generated by the preceding potentiometers, pore pressure transducers and total stress transducers were transmitted from the centrifuge enclosure to the control room containing the data acquisition system. The signal output from the pore pressure and total stress transducers were subjected to low-pass filters to remove the electrical noise with frequency exceeding 10Hz. The filtered output were subsequently amplified a hundredfold. These amplified transducer signals, together
with the potentiometer signal, were converted into 12-bit digital signals and recorded using the Dasylab computer software.
Table 3.1  Centrifuge scaling rules (after Schofield, 1980 & 1988; Taylor, 1995)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model/ Prototype</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear dimension</td>
<td>1/N</td>
</tr>
<tr>
<td>Area</td>
<td>1/N²</td>
</tr>
<tr>
<td>Volume</td>
<td>1/N³</td>
</tr>
<tr>
<td>Density</td>
<td>1</td>
</tr>
<tr>
<td>Mass</td>
<td>1/N³</td>
</tr>
<tr>
<td>Displacement</td>
<td>1/N</td>
</tr>
<tr>
<td>Acceleration</td>
<td>N</td>
</tr>
<tr>
<td>Velocity</td>
<td>1</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
</tr>
<tr>
<td>Force</td>
<td>1/N²</td>
</tr>
<tr>
<td>Time (seepage)</td>
<td>1/N²</td>
</tr>
</tbody>
</table>

Table 3.2  Properties of the kaolin clay (after Goh, 2003, Purwana et al. 2005)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit, %</td>
<td>80</td>
</tr>
<tr>
<td>Plastic Limit, %</td>
<td>35</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.60</td>
</tr>
<tr>
<td>Compression Index</td>
<td>0.562</td>
</tr>
<tr>
<td>Swelling Index</td>
<td>0.122</td>
</tr>
<tr>
<td>Bulk Unit Weight, kN/m³</td>
<td>16.39</td>
</tr>
<tr>
<td>Friction angle, °</td>
<td>23</td>
</tr>
<tr>
<td>Permeability, m/s</td>
<td>2.0×10⁻⁸</td>
</tr>
</tbody>
</table>
Figure 3.1  Centrifuge test set-up for SCP installation (a) front view & (b) side view.

Figure 3.2  Miniature hydraulic motor, Archimedes’ screw and sand hopper casing assembly (after Daramalinggam, 2004).
Figure 3.3  Miniature auger and Archimedes’ screw.

Figure 3.4  The control / power supply system of X-Y table.
Figure 3.5  1-g consolidation test set-up.

Figure 3.6  SCP produced by in-flight installer.
Figure 3.7  Entran EPL-D12 stress transducer (after Juneja, 2002).

Figure 3.8  Layout for total stress transducer (TST) calibration test in the fully saturated, normally consolidated clay.
Figure 3.9  Measured versus applied vertical stress (in vertical calibration test).

Figure 3.10  Measured versus applied lateral stress (in lateral calibration test).
Figure 3.11  T-bar penetrometer.
Chapter 4: CENTRIFUGE MODEL TESTING: RESULTS AND ANALYSIS

4.1 Introduction

As discussed in Chapter 2, the effects of sand compaction piling on stress and strength changes in Singapore marine clay have been reported by Juneja (2002) and Lee et al. (2004). Singapore marine clay has a very low coefficient of permeability of about $5 \times 10^{-10}$ m/s, which may explain why Juneja’s (2002) centrifuge data showed virtually no significant dissipation of excess pore pressure during the installation of pile groups (up to 9 sand piles). In practice, there are soft soils that are more permeable than the Singapore marine clay used by Juneja (2002). For instance, Ariake clay in Japan is reported to have a coefficient of permeability up to $5 \times 10^{-9}$ m/s (Tanaka et al., 2001); this is about 10 times higher than that of Singapore marine clay. One would surmise that, for soils with higher permeability, the consolidation behaviour during sand compaction piling, and thus the amount of set-up, is likely to be different. While the undrained assumption is likely to hold for a single pile installation due to its rapid installation rate of about 0.7 m/min (Kitazume, 2005), substantial dissipation of pore pressure would probably occur when installing multiple piles to form a pile group. The effect of such dissipation on the subsequent soil behaviour in the pile vicinity is one of the key areas of study in this research. In this chapter, the details and results from a series of centrifuge experiments carried out to study this phenomenon are presented and discussed.

The centrifuge experiments performed in this study were carried out to
i) further examine the undrained response during single pile installation and

ii) study the influence of consolidation during single and multiple pile installations.

In the latter, dissipation of excess pore pressure was allowed during the intervals between installations of adjacent piles in a group. Table 4.1 shows the two types of centrifuge modelling tests carried out in this research. The Type I centrifuge test was conducted to investigate the changes in total stress and pore pressures in the surrounding soft soil during the undrained installation of a single pile. The Type II tests were designed mainly to study the effects of consolidation on the set-up of soil strength during multiple pile installations, which includes quantifying the shear strength changes induced by SCP installations.

Table 4.1 summarizes the key features of the Type I and II tests. Total stress and pore pressure variations during single pile installation were measured in the Type I test, while the Type II tests were focused on strength measurements using the in-flight penetrometer. Hence, in the latter tests, total stress and pore pressures were not measured as the transducers could not be embedded without incurring significant risk of damage.

It should be noted that the current centrifuge experiments differed from previous studies (Juneja, 2002; Ng, 2003; Lee et al., 2004) in several important aspects, although similar experimental apparatus were adopted. First and foremost, kaolin clay was used in this study instead of Singapore marine clay. The higher
coefficient of permeability (around 2x10^{-8} m/s) associated with kaolin clay allows for significant dissipation of excess pore pressure within the time intervals between successive in-flight SCP installations. On the other hand, casing jack-in and withdrawal rates utilized in this study were sufficiently fast to achieve undrained response during single or individual pile installation in the kaolin clay. The jack-in and withdrawal rates used herein were about 10.5 mm/s and 7.7 mm/s respectively in model scale, which is about 7 times faster than the corresponding rates adopted by Juneja (2002) and Lee et al. (2004).

In line with the faster installation rates, some changes were made to the in-flight SCP installation procedure. A small amount of sand was injected into the clay during casing penetration to prevent soil from clogging the rapidly advancing casing. This differs from the previous experiments, where sand was injected into the soil only during casing retraction. As will be discussed shortly, such a change may have an unintended but significant effect on the subsequent measured response. Apart from this, the present experiments were performed under a gravity field of 50g to simulate piles of 1.0 m prototype diameter installed in a 5 m thick kaolin clay bed. In contrast, the previous tests were carried out at 80g with piles of 1.4 m diameter installed in a 8.4 m thick marine clay layer.
4.2 Results and discussion of Type I tests

4.2.1 Stress and pore pressure variations during SCP installation

In Test S1, a 20 mm diameter pile was installed in a 100 mm thick kaolin clay bed under a centrifugal acceleration of 50g, which, in the prototype context, is equivalent to installing a 1.0 m diameter pile into a 5 m thick soil layer. As illustrated in Figure 4.1a, the instrumentation comprised three total stress transducers (T1, T2 and T3) and pore pressure transducers (P1, P2 and P3). T1, T2 and T3 were placed at a distance of approximately 1.5 pile diameters away from the SCP central line, at depths of approximately 30mm, 55mm and 80mm (i.e. prototype-equivalent 1.5m, 2.75m and 4m), beneath the ground surface. All three total stress transducers were oriented to measure the total radial stress. The pore pressure transducers, P1, P2 and P3, were located on the other side of the pile, at roughly the same stand-off and depths as T1, T2 and T3, respectively. Figure 4.1b shows the prototype equivalent of the model layout. For ease of description, all dimensions, rates and time in the subsequent sections will be given in prototype scale unless otherwise specified.

Figure 4.2 shows the variation with time of the total radial stress ($\sigma_r$), measured by T1, T2 and T3 during the pile installation. The corresponding pore pressure (u) measurements from P1, P2 and P3 are shown on Figure 4.3. The initial values of total stress and pore pressure before pile installation were assumed to comply with the $K_0$-condition and hydrostatic pressure respectively. The graphs are plotted with respect to the prototype time. As described earlier, the in-flight installation process of each pile comprises two phases: (a) the jack-in, followed by (b)
the subsequent withdrawal of the casing. These two phases are demarcated by the dash line in the graphs. The depth of the casing tip at different times is also shown on the graph (via the triangle symbols). The prototype rates of penetration and withdrawal are approximately 0.2 mm/s and 0.15 mm/s.

As can be seen from Figure 4.2, a gradual build-up in total radial stress was measured by each transducer as the casing was jacked into the ground. The measured stresses attained their peak values as the tip of the casing approached the depth of the corresponding transducers, with the larger depth registering higher peak stresses. The peaking of total stress during the casing jack-in process was also observed by Juneja (2002) and Ng (2003), and was attributed to the combined effects of cavity expansion as well as the stress bulb induced by the jack-in of the casing tube. Lee et al. (2004) also noted this peaking of the total stress and reported that it was consistent with Levadoux & Baligh’s (1980) strain path solution which showed a stress bulb around the tip of the penetrator. The higher peak stresses at greater depths is due to depth-increasing initial stresses and undrained shear strengths, and is consistent with cavity expansion theories (e.g. Vesic, 1972). The post-peak decrease in total stress behind the casing tip is similar to the stress relief phenomenon which occurs during pile driving (Coop & Wroth, 1989). It is also consistent with the passage of the stress bulb in Levadoux and Baligh’s (1980) strain path solution. As soon as the casing reached the desired depth, in this case the bottom of the 5 m thick kaolin clay layer, it was immediately retracted. A sudden drop in radial stress was registered simultaneously by each of the three total stress transducers at the start of
casing withdrawal, as shown on Figure 4.2. After this sudden drop, the measured total stresses bottomed out and showed some increase as the casing was retracted and sand was injected. The increase in radial stress during casing retraction may be attributed to additional cavity expansion caused by the injection of sand.

The initial sudden drop in radial stress at the start of casing withdrawal could be due to several reasons, among which are the instantaneous contraction of the cavity around the casing tube and the reversal of the friction force at the casing-soil interface. As indicated earlier, sand was continuously introduced into the clay during the rapid casing jack-in phase to prevent clogging. As a result, a thin layer of sand was likely to have been deposited around the casing tube by the end of the jack-in process. Upon sudden reversal of the casing movement, such a sand layer may densify, causing the cylindrical soil cavity to contract and the radial stress to drop. Another possible cause is the frictional interaction at the casing-soil interface, whereby the casing tube tend to drag the adjacent soil downward during jack-in phase, and upward during the withdrawal phase. The abrupt reversal of friction force may give rise to the part of the observed sudden stress drop. Theses postulates will be further discussed and examined via numerical studies in Chapter 6.

As shown on Figure 4.3, the pore pressure histories exhibit somewhat different trends from the radial stresses. The transducer P1 located nearest to the ground surface was the first to register an increase in pore pressure during the casing jack-in. This pressure attained a peak value, after which it started to decrease with continuing casing penetration. On the other hand, P2 showed a more gradual rise in pore
pressure, which attained a plateau just before the end of jack-in. The pore pressure measured by P3 increased continuously up to the end of the jack-in process. During casing withdrawal, the measured pore pressures were relatively constant, with P2 registering a slight increase, while P3 showed a slight decrease towards the end of the installation process.

To examine if the post-peak decrease in pore pressure during jack-in (P1) and withdrawal (P3) is attributable to the dissipation of excess pore pressure, a simple dimensionless time calculation was performed. It is noted that transducers P1 and P3 are situated closer to drainage boundaries than P2. For example, P1 was placed about 1.5 m beneath the ground surface, which is a drainage boundary. During jack-in, the casing was impermeable and therefore could not be considered a drainage boundary. Transducer P3 was located about 1 m above the bottom sand layer and approximately 1 m from the boundary of the sand compaction pile formed during casing withdrawal. The coefficient of consolidation of this kaolin clay is about 40m²/year (Purwana et al., 2005). As a rough indicator of the drainage condition, we can define a dimensionless time T using the relationship

\[ T = \frac{c_v t}{s^2} \]  

(4.1)

in which \( c_v \) is the coefficient of consolidation, \( t \) is the duration of the event (in this case, either jack-in or retraction) and \( s \) is the distance to the nearest drainage boundary.
As Table 4.2 shows, the estimated values of T corresponding to the jack-in and retraction stages of a single pile are relatively small, when interpreted in terms of the point consolidation at the P1, P2 and P3 locations. This suggests that the pore pressure dissipation at these locations was not significant during these processes. Alternatively, for the casing penetration phase, the consolidation behaviour may be evaluated using the non-dimensional penetration velocity (Randolph & Hope, 2004):

$$V = \frac{v.d}{c_v}$$

in which $V$ is non-dimensional velocity, $v$ is the penetration velocity and $d$ is the penetrometer or pile diameter. Although the concept of non-dimensional velocity was originally used by Randolph & Hope (2004) for evaluating the degree of consolidation during cone penetrometer push-in, it may also be applied to the casing jack-in process, as both are inherently deep penetration problems. As discussed in Randolph & Hope (2004) and Chapter 5 of this dissertation, fully undrained cone response is expected for non-dimensional velocities that are greater than about 30. In the present study, the computed non-dimensional velocity $V$ associated with the casing penetration is about 107, which suggests that the process is sufficiently fast to achieve undrained response.

The preceding calculations suggest that there was no significant excess pore pressure dissipation during the course of the single pile installation. As such, the installation event of a single pile was likely to be fully undrained. However, such undrained situation may not apply to multiple pile installation because of the
additional time needed for re-positioning the installer and the shorter drainage paths due to the already-installed piles serving as vertical drains.

4.2.2 Comparison with previous centrifuge studies

As described in Chapter 2, Juneja (2002) performed a series of centrifuge tests to study the influence of SCP installation on the adjacent soft clay. The 1.4 m diameter pile was installed in-flight into remoulded and reconsolidated Singapore marine clay. Changes in total radial stresses and pore pressures were examined in the vicinity of the pile. Figure 4.4 presents the typical pore pressure and total radial stress histories measured by Juneja during SCP installation. The reported total stress and pore pressure data were recorded by total stress transducer Ti and pore pressure transducer Pi at the prototype equivalent depths of 2.8 m and 4.2 m respectively. As summarized in Table 4.3, both transducers were placed at a distance of 1.4 m from the pile’s central axis, which is equivalent to 1 pile diameter away.

As can be seen, the peaking of measured total stress and pore pressure during jack-in also occurred as the casing tip approached the depths of the embedded transducers. The phenomenon of stress relief behind the casing tip was similarly manifested as rapid decreases in the measured total stress and pore pressure after peaking. However, a quick comparison with the present experimental observations (Figures 4.2 and 4.3) shows that the measured responses are not quite similar. These may arise partly out of the aforementioned differences in experimental configurations,
such as the thickness of the soft soil layer, the duration and speed of casing jack-in and withdrawal, and the casing and pile size.

A more appropriate comparison between Juneja’s and the present experiments may be made by plotting the normalized stress increase (which is the stress increment normalized by the corresponding vertical effective stress) against the prototype penetration depth, instead of time. This approach allows the two sets of results to be readily compared on a common basis, and avoids the complexities of considering thickness and rate differences when interpreting the results on a time scale. Figure 4.5 shows the normalized total stress increase $T_i$ and pore pressure $P_i$ during the pile installation, plotted against the prototype penetration depth. Also included on these graphs are data from the T2 and P3 transducers of the present study. It should be noted that the present study involves a smaller penetration depth of 5m, compared to 8.4m in Juneja’s tests. This accounts for the break in the measured responses of T2 and P3 between 5m and 8.4m.

Figure 4.5a shows favourable agreement between the total stress measurements of T2 and $T_i$. The comparison is also quite favourable for the pore pressure responses, P3 and $P_i$, as shown on Figure 4.5b. As indicated on the graphs, the prototype equivalent locations of T2 and $T_i$ were quite close, as were P3 and $P_i$. This suggests that, overall, there is good consistency between the measurements obtained from the present and previous experimental studies. It is noted that $T_i$, unlike T2, registered no sudden, steep decline upon the commencement of casing withdrawal (Figure 4.5a). As explained earlier, the sudden drop in radial stress
observed in the present study may have been caused by cavity contraction. Such contraction may occur at the instant of casing reversal, due to densification of the thin sand layer surrounding the casing that was formed during jack-in. Since sand was not discharged during the casing jack-in phase in Juneja’s tests, the clay was in direct contact with the casing; hence, reversal-induced contraction effects, if any, would be minimal, which may explain the absence of any sudden change in the radial stress.

Figures 4.5a and 4.5b also include two additional sets of data (Tii and Pii) from other tests carried out by Juneja. The transducer Tii, which measured the total radial stress, was located at the same depth and radial distance from the pile as Ti (Table 4.3). The pore pressure transducer Pii was placed at a depth of 4.7 m and a radial distance of 2 m from the pile. (Details of transducer positions are summarized in Table 4.3.) As shown on Figure 4.5a, the radial stress responses of Ti and Tii are quite similar, and are generally consistent with the T2 measurements of the present study. It is noted that the pore pressure responses of Pi and Pii are also quite similar, despite being at different radial distances of 1.4 m and 2 m respectively. This may be due to the fact that, in terms of the model scale, the two locations are separated by only 8 mm, which is a relatively small distance compared to the size of the pore pressure transducers.

The preceding comparison shows that, for the installation of a single pile, the overall trends in the measured total stresses and pore pressures of the present study are reasonably consistent with those reported by Juneja (2002).
4.2.3 Initial strength state of clay bed

As described in Chapter 3, the undrained shear strength $s_u$ of the in-situ clay was measured in-flight after the clay bed had been normally consolidated under a 50-g gravity field and prior to SCP installation. Undrained strength profiles were typically measured at different locations using the T-bar penetrometer. Figure 4.6 plots the representative strength profiles, labelled Po-1 to Po-4, measured from such in-flight probes. The measured strengths profiles are quite similar, which indicates that the T-bar measurement is repeatable and the clay bed is reasonably uniform. In the same figure, the mean strength profile P-O is obtained by averaging the undrained shear strengths from Po-1 to Po-4 at each depth. This mean profile is representative of the initial strength condition of the soil, and will be adopted in the subsequent discussions.

Before undergoing self-weight consolidation under the 50-g gravitational field, the soil was first pre-loaded at 1-g under a pressure of 6.5 kPa. As a result, the top 1 m or so of the soil layer is likely to be over-consolidated, this being also reflected in the strength profile. Beneath this overconsolidated layer, soil strength generally increases linearly, at a rate of approximately 1.72 kPa/m. For the kaolin clay with an effective unit weight of about 6.39 kN/m$^3$ (Table 4.4), the strength variation can be approximated as:

$$s_u = 0.269\sigma'_{x_0}$$  \hspace{1cm} (4.3)
in which $\sigma'_{v0}$ is the in-situ effective vertical stress. Alternatively, the shear strength can also be estimated using the following relationship developed by Wroth (1984) with the Cam-clay model:

$$s_u = \frac{M}{2} p' \left( \frac{R_i}{2} \right)^{\Lambda}$$  \hspace{1cm} (4.4)

where $M$ is the friction coefficient, $p'$ is the mean effective stress, and $R_i$ is the isotropic overconsolidation ratio. $\Lambda$ is the plastic volumetric strain ratio, evaluated as $(\lambda - \kappa)/\lambda$, wherein $\lambda$ and $\kappa$ are the slopes of the isotropic consolidation line and the swelling line in the $e$-$\ln p'$ space. The key Cam-clay parameters of kaolin clay are summarized in Table 4.4 (Goh, 2003). Substituting these parameters into Equation 4.4, the calculated value of $s_u/p'$ is 0.33.

As the present kaolin clay is normally-consolidated (except for the top 1 m or so), the earth pressure coefficient at rest can be estimated using $K_0 = 1 + \sin \phi'$ (Jaky, 1944). Equation 4.4 can then be further expressed in terms of $\sigma'_{v0}$:

$$s_u = 0.244 \sigma'_{v0}$$  \hspace{1cm} (4.5)

Using the Cam-clay parameters, the estimated $s_u/\sigma'_{v0}$ ratio of 0.244 is quite close to the value measured in the centrifuge model, i.e. 0.269. In addition, the shear modulus $G$ of the clay can be estimated using the Cam-clay parameters (Wood, 1990) as:

$$G = \frac{\sqrt{3(1-2\nu)(1+\epsilon)p'}}{2(1+\nu)\kappa}$$  \hspace{1cm} (4.6)
where \( e \) and \( \nu' \) are the void ratio and effective Poisson’s ratio of the soil respectively. For a typical void ratio of 1.7, the calculated \( G/p' \) ratio is 19.6. Equations 4.4 and 4.6 can be combined to obtain \( G/s_u = 59 \). The corresponding \( E_u/s_u \) ratio is therefore approximately 180, which is within the typical range expected of a soft clay.

### 4.2.4 Comparison of measured radial stress and pore pressures with analytical solutions

In this section, the measured peak total stress and pore pressure increase during single pile installation will be compared with analytical predictions from various cavity expansion calculations.

#### 4.2.4.1 Vesic’s (1972) cavity expansion theory

Vesic’s (1972) plane strain cavity expansion theory assumed that the soil is a linearly elastic, perfectly plastic material which obeys the Mohr-Coulomb failure criteria. The initial stress state of the soil mass was assumed to be isotropic. As the cavity expands, soil yielding initiates from the cavity wall and propagates radially outwards. Beyond the plastic zone, the soil remains elastic, as shown on Figure 4.7. For undrained cavity expansion, the radius of the plastic annular zone \( R_p \) can be expressed as

\[
R_p = R_u \sqrt{\frac{G}{s_u}} \tag{4.7}
\]
where \( R_u \) is the cavity radius. The total radial stress increment \( \Delta \sigma_r \) in the plastic zone can be calculated using the following relationship:

\[
\Delta \sigma_r = s_u \left( \ln\left(\frac{G}{s_u}\right) + 1 \right) - 2s_u \ln\left(\frac{r}{R_u}\right)
\]  

(4.8)

where \( r \) is radial distance. Following Skempton’s excess pore pressure derivation, Vesic (1972) predicted the excess pore pressure \( \Delta u \) in the plastic zone as

\[
\Delta u = [0.578(3A_f - 1) + 2 \ln\left(\frac{R}{r}\right)]s_u
\]

(4.9)

where \( A_f \) is Skempton’s pore pressure parameter at failure.

Figures 4.8a-c compare the measured peak total stresses and pore pressure increases with the analytical predictions from Vesic’s cavity expansion theory for three \( E_u/s_u \) ratios of 150, 200 and 250. (Recall that previous calculations indicate the \( E_u/s_u \) ratio of kaolin is approximately 180, which provides reference for choosing \( E_u/s_u \) ranging from 150 to 250.) The horizontal axis in Figure 4.8 represents the measured peak total stress and pore pressure increases (\( \Delta \sigma_m, \Delta u_m \)). The vertical axis shows the calculated total stress and pore pressure increases (\( \Delta \sigma_c, \Delta u_c \)) using Equations 4.8 and 4.9. In the calculations, the casing jack-in process was approximated by the expansion of a cylindrical cavity to the casing diameter, while casing withdrawal and sand pile formation was approximated by further cavity expansion to the sand compaction pile diameter. As cavity expansion calculations could not depict the complete histories of total stress and pore pressure changes, comparisons were made only with the peak values of total stress and pore pressure increases measured during
casing jack-in and withdrawal. Overall, there is reasonable agreement between the measured and calculated values, with the results for $E_u/s_u$ ratios of 200 and 250 (Figures 4.8b and c) comparing quite favourably with the experimental measurements. In particular, the pore pressure increases are fairly well predicted by cavity expansion solutions, as shown on Figure 4.8c.

Generally, the discrepancies are more significant with respect to total stresses (compared to pore pressures), with the cavity expansion predictions underestimating total stress increases during casing jack-in, and overestimating them during casing withdrawal. The discrepancies also appear to be more significant at larger depths. One possible explanation for this phenomenon is the violation of the isotropic initial stress condition. As stated earlier, Vesic’s cavity expansion theory assumed that the initial stress state was isotropic. However, for a normally or lightly overconsolidated soil, the initial stress state is likely not isotropic. The violation of this isotropic stress state assumption may affect the accuracy of stress predictions more than pore pressure predictions, which may account, in part, for why the deviation of total stress predictions from measured values is more significant than the excess pore pressures. Another possible contributing factor is the effect of soil heaving, during installation, which would violate the plane strain assumption. The effect of vertical soil movement on plane strain cavity expansion will be further discussed in the next section.
4.2.4.2 Lee et al.’s (2004) cavity expansion theory

As described above, the presence of soil heaving may be one reason for the discrepancy between plane strain cavity expansion predictions and the measured stresses. Randolph & Wroth (1979) accounted for this vertical soil movement by introducing an adjustment parameter to the original cavity expansion calculations. Lee et al. (2004) noted that, during SCP installation, the deviation of the measured stress increase from cavity expansion predictions occurred not only in the vertical direction, but also in the radial direction. Randolph & Wroth’s “leakage of soil” concept cannot account fully for such deviations in the radial direction. To account for the effect of free surface on plane strain cavity expansion, Lee et al. (2004) proposed modified cavity expansion expressions.

Lee et al. (2004) noted that, for soils at shallow depths, the assumption of constant vertical stress may be more applicable than the plane strain condition, which holds true for greater depths. By merging the two limits imposed by the constant vertical stress and plane strain conditions, they were able to establish a semi-empirical relationship which fitted their centrifuge test data much better. In their modified cavity expansion theory, soil is similarly assumed to follow Mohr-Coulomb failure criteria and subjected to the isotropic initial stress field prior to the expansion of cavity. The radius $R_p$ of plastic zone, however, is re-expressed as follows:

$$R_p = R_u \sqrt{\frac{G}{\sigma_u}} \exp(1 - \alpha_i)$$  \hspace{1cm} (4.10)
\[ \alpha_1 = \frac{0.15 \left( \frac{z}{R_u} \right)^2}{1 + 0.15 \left( \frac{z}{R_u} \right)^2} \]  

(4.11)

where \( \alpha_1 \) is a fitted parameter allowing for the transition from the constant vertical stress condition to the plane strain condition by varying from 0 to 1, and \( z \) is the depth of interest. Equation 4.10 converges to the plane strain scenario as the depth \( z \) increases. The radial stress increase due to cavity expansion is recast as follows

\[ \Delta \sigma_r = \left( \frac{z}{r} \right)^{3.33} \left\{ \alpha_1 \cdot s_u \left[ \ln \left( \frac{G}{s_u} \right) - 1 \right] - 2s_u \left[ \ln \left( \frac{r}{R_u} \right) - 1 \right] \right\} \]  

(4.12)

The excess pore pressure \( \Delta u \) is then given by

\[ \Delta u = \left( \frac{z}{r} \right)^{3.33} \left\{ 0.578 \left( 3A_r - 1 \right) + \frac{2}{3} \left[ 1 - 2 \ln \left( \frac{r}{R_u} \right) \right] + \alpha_1 \left[ \ln \left( \frac{G}{s_u} \right) - \frac{2}{3} \left( 1 + \ln \left( \frac{r}{R_u} \right) \right) \right] \right\} s_u \]  

(4.13)

Figures 4.9a-c show the calculated vs experimental total stress and pore pressure increases for three \( E_u/s_u \) ratios of 150, 200 and 250. The results suggest that the modified cavity expansion approach does not improve the accuracy of total stress predictions. This is to be expected. In Lee et al.’s (2004) study, the computed stress and pore pressure increments are generally higher than the measured values. Accounting for the proximity to the ground surface has the effect of lowering the computed values thereby improving the agreement. In this case, the computed values are generally lower than the measured values. Thus, using Lee et al.’s (2004)
will, in fact, accentuate the discrepancy. Hence, in addition to vertical soil movement, there may be other factors which significantly influence the accuracy of the predictions.

4.2.4.3 Cao et al.’s (2001) cavity expansion theory

Apart from Mohr-Coulomb based cavity expansion theories, cylindrical cavity expansion calculations incorporating the modified Cam-clay model (Collins & Yu 1996; Cao et al., 2001) were also carried out. Adopting similar assumptions of plane strain and isotropic initial stress states, Cao et al. (2001) proposed an approximate closed-form solution for undrained cavity expansion in modified Cam-clay. As the cavity expands, the total radial stress increase $\Delta \sigma_r$ in the plastic zone is expressed as follows:

$$\Delta \sigma_r = M p'_0 \left( \frac{R_i}{2} \right)^2 \left[ \sqrt{\frac{3}{2}} + \frac{2}{3} \frac{\sqrt{3}}{3} \ln \left( \frac{R_p}{r} \right) \right]$$

where $p'_0$ is the initial mean effective stress, $M$ is the friction constant, $R_i$ is the isotropic overconsolidation ratio, $\lambda$ is the plastic volumetric strain ratio, $r$ is the radial distance and $R_p$ is the radius of the plastic annular zone.

The excess pore pressure is evaluated using the following relationship

$$\Delta u = \frac{2\sqrt{3}}{3} M p'_0 \left( \frac{R}{2} \right)^{\lambda} \ln \left( \frac{R_p}{r} \right) + p'_0 \left[ 1 - \left( \frac{R}{2} \right)^{\lambda} \right]$$

The calculations were carried out using the kaolin Cam-clay parameters summarized in Table 4.4. Figure 4.10 shows the comparison between the calculated values and
experimental measurements. Like the cavity expansion methods considered earlier (Vesic, 1972; Lee et al., 2004), the agreement is quite favourable for the pore pressure increases, but less so for the total stress increases.

4.2.5 Summary of the single pile installation

During the installation of sand compaction piles, considerable changes in the radial stresses and pore pressures were observed during casing jack-in and subsequent withdrawal. In the preceding sections, comparisons were made between the measured experimental data and predictions from various cavity expansion theories (Vesic, 1972; Lee et al., 2004; Cao et al., 2001). In general, cavity expansion calculations produced good estimates for the pore pressure increases observed during SCP installation, while predictions of total stress increases are comparatively less satisfactory. This may be partly attributed to the initial isotropic stress state assumed in cavity expansion theories, which may not be the case in a normally or lightly over-consolidated soil.

Furthermore, the physical processes and mechanisms involved in SCP installation are obviously much more complex than the idealized phenomenon of cylindrical cavity expansion. The complex nature of the problem, coupled with the limitation of cavity expansion theories, highlights the need for more realistic modelling of the installation process that simulates the casing jack-in and subsequent
withdrawal. Such an approach, using the finite element method with advanced contact features and large deformation capabilities, is introduced in the Chapter 6.

4.3 Results and discussion of Type II Tests

4.3.1 Overview of tests

As stated at the beginning of this chapter, the second group of centrifuge tests (Type II) was carried out to evaluate and quantify time-dependent shear strength changes in the clay surrounding the installed sand compaction pile. They were designed to investigate how shear strength set-up was affected by consolidation during and after SCP installation. By controlling the time intervals between consecutive pile installations, significant consolidation was allowed to take place between the installation of consecutive piles within a group. The installation of the individual pile remained largely undrained as before.

A total of five tests, T1 to T5, were carried out in this tests series, the details of which are tabulated in Table 4.1. Tests were carried out under a model gravity field of 50g with the piles installed in-flight. Shear strength measurements were also carried out in-flight by means of the T-bar penetrometer, as described in Chapter 3. As summarized in Table 4.1, the main differences amongst these five tests are:

(i) the number of installed piles,

(ii) the time, after installation of the last pile, at which strength profiling is performed, and
(iii) the installation interval between consecutive piles.

The pile layouts and the strength probe locations for the different tests are shown on Figures 4.11a-c (in model scale).

A single pile was installed in-flight in test T1 (Figure 4.11a). Strength measurements were carried out at points A and B located on opposite sides of the pile, with radial distances of 30mm (i.e. prototype 1.5 m) to the pile axis. Strength profiling at these two points was carried out at different times. Point A was profiled in the “short” term, which was 2 to 3 minutes (i.e. prototype 3 - 5 days) after the pile installation. The strength profile at Point B was measured in the “long” term, which was more than 45mins (i.e. prototype 78 days) after the pile installation. The terms, “short” and “long”, used here are defined with respect to the time required for the dissipation of excess pore pressures at the given radial distance. As Figure 4.12 shows, the transducer located at different depths of the clay layer indicated that excess pore pressures generated by the single pile installation would take approximately 35 mins (i.e. prototype 60 days) to fully dissipate at a radial distance of 30mm. Hence, for strength measurements made in the short term, which is 2 to 3 mins after installation, the bulk of the excess pore pressures generated would still be present. On the other hand, the long-term strength measurements were carried out more than 45 minutes after pile installation; this corresponds to the condition in which the excess pore pressure would have fully dissipated.

In tests T2 and T3, two piles were installed, with a centre-to-centre spacing of 60 mm (i.e. prototype 3 m), as shown on Figure 4.11b. Strength measurements were
performed at the midpoint between the two piles, labelled C in the graph. While the pile layout is similar in T2 and T3, there is a difference in the installation procedure. In T2, the piles were installed consecutively, with only a brief pause of approximately 2 minutes (i.e. prototype 3 days) between installations; this duration is about the minimum time required to re-position the SCP installer in-flight. Such an installation sequence is hereafter referred to as “consecutive installation”. From the preceding discussion on short and long term conditions associated with test T1, it is expected that, during the consecutive installation of T2, significant excess pore pressures (generated by the first pile) were present at the midpoint between the piles prior to the installation of the second pile. In contrast, the piles in T3 were installed with an interval of more than 45 minutes (i.e. prototype 78 days) in between; this representing a delay or pause in the construction. By doing so, full dissipation of the excess pore pressures at the midpoint between the piles would have occurred during the interval between installations. This installation procedure is hereafter referred to as “separate installation”. For both T2 and T3, strength was measured more than 45 minutes after the second pile was installed, at the mid-point C in Figure 4.11b. This represents the long-term strength was at the midpoint between the two piles.

In tests T4 and T5, the four piles were laid out in a square, with centre-to-centre spacings of 60 mm (i.e. prototype 3m), as shown on Figure 4.11c. In T4, the four piles were installed consecutively, with only brief pauses of about 2 minutes (i.e. prototype 3 days) between adjacent pile installations. On the other hand, the four piles in T5 were installed separately, with intervals of at least 45
minutes (i.e. prototype 78 days) between successive installations. In both tests, strength profiling was carried out at the midpoint of the four piles (point D in the graph) more than 45 minutes after the installation of the fourth pile.

The above tests were designed to study the effect of two key factors on the soil strength set-up phenomenon, namely consolidation and pile group effect. In the following section, the experimental results will be presented to illustrate the set-up phenomenon due to single and multiple pile installations. The results will be further interpreted and discussed with respect to consolidation and pile group effects. As before, in the subsequent discussion on the experimental results, only the prototype dimensions will be adopted for the sake of clearance.

4.3.2 Consolidation effect

In test T1, points A and B were both located at a distance of 1.5 times the pile diameter from the pile axis. However, the strength measurements at these locations were taken at different times. At point A, the strength profile was measured shortly after the pile installation, before any significant dissipation of excess pore pressures at this location. On the other hand, the profile at point B reflected the long-term, post-consolidation strength of the clay. Figure 4.13 plots the measured strength profiles at points A and B, which are labelled “P-A” and “P-B” respectively. In addition, the initial in-situ strength profile of the clay bed (discussed in Section 4.2.3) is also shown on the graph, labelled “P-O”. It is clear that the shear strengths
associated with the P-A and P-B profiles are greater than the initial values indicated by the P-O profile. In fact, the strength values from P-B are almost doubled those from P-O.

For depths greater than 1.5 m, the strengths associated with P-B are noticeably greater than those of P-A. Given that both points are equidistant from the pile axis, the discrepancy may be attributed to the different times at which the strengths were measured. That is, P-A and P-B represent the short-term and long-term strength profiles, respectively, at a distance of 1.5 times pile diameter from the pile. It is a result of excess pore pressure dissipation. With time, the excess pore pressures dissipate while the effective stresses increase, leading to the higher strengths.

Figure 4.14 plots the strength profiles, labelled “P-C(T2)” and “P-C(T3)”, obtained at point C in tests T2 and T3 respectively. For reference, the initial strength profile, P-O, is also included on the graph. As described earlier, the two piles were installed consecutively in test T2, and separately in test T3. In both cases, the long-term strengths were measured after the installation of the second pile. As shown on the graph, the long-term shear strengths of P-C(T3) are larger than those of P-C(T2). Since both profiles are measured at a point midway between the two piles, the different strengths are likely to result from the “consecutive” vs “separate” installation procedures. In the consecutive installation of test T2, no significant dissipation of excess pore pressures was permitted during the short interval between the first and second pile installations. As a result, the midpoint C experienced no substantial strength gain prior to the installation of the second pile. The increase in
strength shown by the profile P-C(T2) is mainly due to set-up effects caused by the second pile installation, measured after a sufficiently long time has elapsed. On the other hand, the separate installation procedure adopted in test T3 allowed for almost complete dissipation of excess pore pressures at the midpoint C before the second pile installation. As a result, there is significant strength gain at this point prior to the second pile installation. The dissipation of additional pore pressures generated by the second pile installation introduced further set-up effects, resulting in the measured strength profile P-C(T3) shown on the Figure 4.14.

Figure 4.15 shows the measured strength profiles from tests T4 and T5 at the mid-points D. In both tests, four piles were installed in a square configuration. The piles in test T4 were installed consecutively, with only short intervals between successive installations for adjusting the X-Y table. On the other hand, the piles in T5 were installed separately, with a long interval between successive installations to allow complete dissipation of excess pore pressures generated by any one pile. As shown on the graph, the long-term strengths of P-D(T5) are substantially higher than those measured of P-D(T4). The profile P-D(T5) corresponds to the long-term strength condition when the piles are installed separately, while the profile P-D(T4) corresponds to the long-term condition when the piles are installed consecutively. The results are consistent with the measurements reported for tests T1 to T3 in Figures 4.13 and 4.14.

The preceding experimental results highlight the importance of allowing for excess pore pressure dissipation between successive installations of piles in a group.
In cases where full consolidation was allowed to take place between successive pile installations, the measured strength gains of the soil were noted to be more significant. This may be attributed to the interaction between cavity expansion and consolidation. The dissipation of excess pore pressure makes soil stiffer before the subsequent pile installation. The interaction between pore pressure dissipation and cavity expansion leads to a multiplier effect which may play an important role for the ground improvement effect arising from SCP installation.

4.3.3 Pile group effect

Besides consolidation, the effect of multiple pile installation on the long term development of the soil strength was also examined in this study. As illustrated on Figure 4.11, several configurations were considered in this study: (a) a single pile, (b) a 2-pile group, and (c) a 2×2 pile group. In tests T1, T3 and T5, full consolidation was achieved at a radial distance of 1.5 m or 1.5 times pile diameter after the installation of each pile, before proceeding with the subsequent installation. The effect of installing a second pile on the long-term shear strength at a point located 1.5 times pile diameter from both piles, is shown on Figure 4.16. In this figure, the profiles P-B (from test T1) and P-C(T3) (from test T3) correspond to the measured long-term strengths due to the one and two pile installations respectively. It is clear that the installation of the second pile caused an increase in the long-term shear strength.
On the same figure, the strength profile P-D(T5) shows the additional strength increase after the installation of the third and fourth piles. As described earlier, point D is located at the centre of a 2×2 pile group. It is observed that P-D(T5) shows an even greater strength increase, compared to the P-B and P-C(T3). Hence, the results indicate that the installation of additional piles would further enhance the long-term strength of the soil located between the piles.

The same conclusion can be drawn by comparing the results from tests T2 and T4, in which successive piles were installed consecutively. Figure 4.17 plots the strength profiles P-C(T2) and P-D(T4), from tests T2 (2-pile group) and T4 (2×2 pile group) respectively, measured long-term after the installation of the last pile. The long-term strengths of P-D(T4) are consistently larger than those of P-C(T2), indicating, as before, that the pile group effect is beneficial in enhancing the long-term soil strengths.

### 4.3.4 Conclusion remarks

Centrifuge experiments were carried out to investigate and quantify the phenomenon of shear strength set-up associated with the installation of single and multiple sand compaction piles. The emphasis of this study was placed on examining the consolidation and pile group effects. Two distinct installation sequences were utilized to study the consolidation influence. The first involved consecutive installation of successive piles, in which the time interval between installations was
kept to the bare minimum required for adjusting the X-Y table, hence ensuring little, if any, excess pore pressure dissipation. In the second sequence, the piles are installed separately, with a time interval of at least 45 minutes (i.e. prototype 78 days) between successive installations to permit full dissipation of excess pore pressures at a radial distance of 30 mm (i.e. prototype 1.5 m). It was noted that, for the same radial distance of 30 mm at which measurements were made, the piles installed separately exhibited more significant increases in the long-term strength. This may be attributed to the dissipation of excess pore pressures permitted during the intervals between pile installations, which introduced more significant set-up effects contributing to strength gains. This has implications for field practice, and suggests that the set-up phenomenon may be optimized by allowing for timely excess pore pressure dissipation between successive pile installations. This may be achieved in practice by adjusting the construction speed or the time intervals between successive pile installations.

Apart from consolidation, the pile group effect is another factor that influences the set-up of soil strengths. The experimental data indicates that multiple pile installation results in a beneficial increase in the soil strengths measured between the piles.
Table 4.1  Centrifuge models test details

<table>
<thead>
<tr>
<th>Test group</th>
<th>Type I</th>
<th>Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test identifier</td>
<td>S1</td>
<td>T1</td>
</tr>
<tr>
<td>No. of installed SCP(s)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Test measurement</td>
<td>$\Delta \sigma_r &amp; \Delta u$</td>
<td>$s_u$</td>
</tr>
<tr>
<td>Strength profiling points</td>
<td>Null</td>
<td>A</td>
</tr>
<tr>
<td>Strength profiling time after the installation of last pile (mins)</td>
<td>Null</td>
<td>2-3</td>
</tr>
<tr>
<td>Installation sequence</td>
<td>Null</td>
<td>Null</td>
</tr>
</tbody>
</table>

$\Delta \sigma_r \& \Delta u$ - total radial stress increase and excess pore pressure  
$s_u$ - undrained shear strength  
A, B, C, D - strength acquisition locations (see Figure 4.11)  
Con - consecutive installation  
Sep - separate installation

Table 4.2  Estimation of dimensionless time $T$

<table>
<thead>
<tr>
<th>Transducer</th>
<th>Casing Jack-in</th>
<th>Casing Withdrawal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$s^*$ (m)</td>
<td>$T$</td>
</tr>
<tr>
<td>P1</td>
<td>1.5</td>
<td>0.013</td>
</tr>
<tr>
<td>P2</td>
<td>2.25</td>
<td>0.006</td>
</tr>
<tr>
<td>P3</td>
<td>1</td>
<td>0.03</td>
</tr>
</tbody>
</table>

* $s$ is the distance to the nearest drainage boundary, in model dimensions.

Table 4.3  Summary of Juneja’s (2002) centrifuge experimental information from selected tests.

<table>
<thead>
<tr>
<th>Data &amp; Transducer identifier referred to in this chapter</th>
<th>Transducer type</th>
<th>Transducer position</th>
<th>Transducer &amp; Test identifier, as referred to in Juneja (2002)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ti</td>
<td>TST</td>
<td>2.8</td>
<td>1.4</td>
</tr>
<tr>
<td>Pi</td>
<td>PPT</td>
<td>4.2</td>
<td>1.4</td>
</tr>
<tr>
<td>Tii</td>
<td>TST</td>
<td>2.8</td>
<td>1.4</td>
</tr>
<tr>
<td>Pii</td>
<td>PPT</td>
<td>4.7</td>
<td>1.98</td>
</tr>
</tbody>
</table>

* Tests T2, T4 and T5 were all conducted at 70g. The casing jack-in and withdrawal rates (in model scale) are 2.14 mm/s and 0.5 mm/s, 1.94 mm/s and 1.0 mm/s, 1.85 mm/s and 0.71 mm/s at T2, T4, T5, respectively.
Table 4.4  Cam-clay properties of kaolin clay (Goh, 2003)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma' ) (kN/m(^3))</td>
<td>6.39</td>
</tr>
<tr>
<td>( k ) (m/s)</td>
<td>( 2.0 \times 10^{-8} )</td>
</tr>
<tr>
<td>( M )</td>
<td>0.9</td>
</tr>
<tr>
<td>( \kappa )</td>
<td>0.053</td>
</tr>
<tr>
<td>( \lambda )</td>
<td>0.244</td>
</tr>
<tr>
<td>( \nu' )</td>
<td>0.33</td>
</tr>
<tr>
<td>( \Gamma )</td>
<td>3.221</td>
</tr>
</tbody>
</table>
Figure 4.1 Sketches of centrifuge experimental set-up: (a) in model scale; (b) in prototype scale.

Figure 4.2 Measured total stress variation during the SCP installation of test S1.
Figure 4.3  Measured pore pressure variation during the SCP installation of test S1.

<table>
<thead>
<tr>
<th>Type</th>
<th>Depth (m)</th>
<th>Radial distance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ti</td>
<td>TST</td>
<td>2.8</td>
</tr>
<tr>
<td>Pi</td>
<td>PPT</td>
<td>4.2</td>
</tr>
</tbody>
</table>

Figure 4.4  Total lateral stress and pore pressure changes during SCP installation (after Juneja, 2002).
Figure 4.5 Comparison of the present and previous test results: (a) total stress increase, (b) excess pore pressure.
Figure 4.6  Initial strength state of soil bed.

Figure 4.7  Expansion of cylindrical cavity in an infinite medium.

R_e - Radius of cavity
R_p - Radius of plastic zone
Figure 4.8 Comparisons between measured total stresses and pore pressure increases ($\Delta\sigma_m, \Delta u_m$) and calculated values ($\Delta\sigma_c, \Delta u_c$) from solutions by Vesic (1972) (a) $E_u/s_u=150$, (b) $E_u/s_u=200$, (c) $E_u/s_u=250$. 
Figure 4.9  Comparisons between measured total stresses and pore pressure increases ($\Delta \sigma_m, \Delta u_m$) and calculated ones ($\Delta \sigma_c, \Delta u_c$) from solutions by Lee et al. (2004) (a) $E_u/s_u=150$, (b) $E_u/s_u=200$, (c) $E_u/s_u=250$. 

\[ \Delta \sigma_c, \Delta u_c \text{ (kPa)} \]

Depth (m): 1.5 2.75 4

$\Delta \sigma$ (jack-in) $\Delta \sigma$ (withdrawal) $\Delta u$ (jack-in) $\Delta u$ (withdrawal)
Figure 4.10  Comparisons between measured total stresses and pore pressure increases ($\Delta \sigma_m, \Delta u_m$) and calculated ones ($\Delta \sigma_c, \Delta u_c$) from solutions by Cao et al. (2001).
Figure 4.11 SCP layout and strength acquisition point positions of test (a) T1; (b) T2 and T3; (c) T4 and T5.
Figure 4.12 Dissipation of excess pore pressure after SCP installation, observed at 30mm radial distance from SCP axis, at different depths of 100mm-thick soil sample.

Figure 4.13 Centrifuge measured strength profiles: P-O, P-A, and P-B.
Figure 4.14  Centrifuge measured strength profiles: P-O, P-C(T2), and P-C(T3).

Figure 4.15  Centrifuge measured strength profiles: P-O, P-D(T4), and P-D(T5).
Figure 4.16  Centrifuge measured strength profiles: P-O, P-B, P-C(T3), and P-D(T5).

Figure 4.17  Centrifuge measured strength profiles: P-O, P-C(T2), and P-D(T4).
Chapter 5: NUMERICAL STUDY OF CONE PENETRATION RATE EFFECTS

5.1 Introduction

In Chapters 3 and 4, the centrifuge experimental details and results of the short and long-term soil response associated with single and multiple sand compaction pile (SCP) installation were presented and discussed. The next part of this study focuses on the numerical analyses that were carried out to model the SCP installation, and its effect on the surrounding soil. This is a highly-complex penetration problem that involves not only large deformations and strains, but also highly nonlinear and time-dependent soil behaviour. Before embarking on the modelling of the SCP problem, a detailed study was first conducted to investigate the feasibility of using the finite element software, ABAQUS, to analyze deep penetration problems involving coupled-consolidation soil response. For this purpose, the cone penetration test was selected as the study problem.

As will be discussed shortly, the numerical modelling of the cone penetration test is not new. Various studies on this topic have been reported in the literature, most of which treated the problem as either drained or undrained. Thus far, the effects of consolidation have not been rigorously considered in the numerical modelling of such problems. The present numerical analysis focuses on modelling the build-up and dissipation of excess pore pressure during cone penetration. More specifically, the effects associated with different rates of penetration are examined numerically. Experimental studies which study the cone penetration rate effects have
been reported by Randolph & Hope (2004) and Kim et al. (2008). The results from these studies are used to validate the proposed numerical model.

In this chapter, the numerical modelling of the cone penetration test, incorporating coupled consolidation effects, will be covered in detail. By varying the rate of penetration, the results will be interpreted and discussed with respect to the fully drained, the fully undrained and the intermediate drainage conditions. The materials in this chapter serve as validation of the proposed methodology for modelling deep penetration problem. More importantly, it provides the basis for the numerical modelling of sand compaction pile installation, which will be covered in detail in Chapter 6.

5.2 Literature review

The cone penetration test (CPT) is a popular in-situ procedure for identifying soil stratigraphy and correlating soil properties. Since its inception, the features of this instrument have evolved significantly, from the earlier mechanical models to the current electric ones. In particular, the piezoecone penetrometer, which allows pore pressures in the soil to be measured simultaneously as the probe advances, is widely used in current practice.

The piezoecone provides continuous measurements of cone resistance and pore pressure along its passage, which are useful for the determination of the engineering properties of soil. The cone tip resistance, in particular, offers valuable information which could be utilized to assess the strength characteristics of soil. Correlations
between the cone tip resistance and the shear strength of soil are typically derived using one of the following theoretical basis: (a) bearing capacity theory, (b) cavity expansion method, (c) strain path approach. Earlier research (Meyerhof, 1961; Sokolovskii, 1965) treated the cone penetration process as a bearing capacity problem. In this approach, the cone tip resistance was analogous to the bearing capacity of a deep foundation, and was simply evaluated as the collapse load of a deep circular foundation in soil, which could be solved by limit equilibrium or slip-line methods (Yu & Mitchell, 1998). The second approach is based on the analogy between cavity expansion and the cone penetration process, in which the cone tip resistance was correlated to the limiting pressure obtained from cavity expansion theories (Ladanyi & Johnson, 1974; Salgado et al., 1997; Yu, 2000). In the third approach, Levadoux & Baligh (1980) regarded the cone penetration process as a steady state problem which was analyzed using the strain path method. This method analyzes the penetration mechanism as a strain-controlled problem to produce the strain field in the soil surrounding the penetrating cone.

The limitations and advantages of the foregoing theoretical approaches are discussed in detail by Yu & Mitchell (1998). Given the large deformation and strains involved, coupled with strong material nonlinearity, it is practically impossible to obtain rigorous analytical solutions for the cone penetration problem. As such, numerical modeling techniques, such as the finite element and finite difference methods, have become more popular in dealing with such problems.

While powerful and versatile, the numerical analysis of cone penetration is
also faced with many challenges involving the modelling of large strains, complex interface behavior and highly nonlinear soil response as the cone is pushed from the ground surface to depths of tens of metres. Various approaches have been reported in the literature: (a) strain path method with small strain finite element analysis (Teh & Houlsby, 1991), (b) cavity expansion with finite element analysis (Silva et al., 2006; Abu-Farsakh et al., 2003), (c) “steady state” finite element analysis (Yu et al., 2000), (d) large-strain finite element analysis with finite sliding contact (Sheng et al., 1997; Huang et al., 2004), (e) arbitrary Lagrangian-Eulerian approach with small strain analysis (Hu & Randolph, 1998; Lu et al., 2004), (f) large deformation analysis with Eulerian formulation (van den Berg, 1994; Hu, 2003) and (g) large-strain finite difference analysis with moving boundaries (Ahmadi et al., 2005).

The preceding body of numerical-related work has significantly contributed to the understanding of the cone penetration mechanism, and has led to the development of more rational approaches for estimating the cone tip factor. Nevertheless, due to the complexity of the problem, many of these analyses involved some idealization and simplification of one form or another. With few exceptions, one common idealization involved modelling the soil response as either fully drained or fully undrained, corresponding to penetration in cohesionless and cohesive soils respectively. Such a simplification precludes the need to carry out coupled consolidation analyses, which involves the additional complexity of modelling the time-dependent response. However, as will be discussed below, the response of real soils is seldom readily classified as either drained or undrained. This is especially
more so in the cone penetration problem, where rate effects play an important role in the interaction between the cone resistance and the drainage response of the surrounding soil.

The cone penetration test has been used on many types of soils, ranging from soft clays to granular sandy materials (Leroueil et al., 1995; Na et al., 2005). In practice, the test is normally carried out using a standard constant penetration rate of 20 ± 5 mm/s (ISSMFE IRTP, 1999; ASTM-D 5778). This penetration rate is selected so that the penetration process in clayey soils is largely undrained, while penetration in sand is drained. However, for intermediate soils such as clayey silts and silts, partially drained conditions may persist during penetration (Lunne, et al., 1997). This can complicate the interpretation of CPT results for soils of moderate to high permeability.

The influence of penetration rates on cone resistances has long been recognized (e.g. Bemben & Myers, 1974; Campanella et al., 1983; Powell & Quartermann, 1988). More recently, laboratory experiments were carried out using centrifuge models (e.g. Randolph & Hope, 2004; Chung et al., 2006) and calibration chamber (Kim, 2005; Kim et al., 2008) to examine the relationship between the factors affecting drainage and the measured cone resistance under controlled testing conditions. Using penetration rates that ranged from 0.005 mm/s to 3 mm/s, Randolph & Hope (2004) carried out a series of centrifuge tests on normally consolidated kaolin clay to explore the influence of penetration speed on the cone resistance and excess pore pressure. It was noticed that, as the penetration rate was
reduced within the undrained response regime, the measured tip resistance of the cone decreased slightly due to the reduced viscous effect. However, a substantial increase in resistance was observed when further reductions in the penetration speed rendered the penetration partially to almost fully drained. The significant increase in cone tip resistance is attributed to local strengthening of the soil around the cone tip as excess pore pressure dissipates (Chung et al., 2006). Similar experiments at 1-g level were carried out by Kim (2005) using the calibration chamber, who reported that the measured cone tip resistance in specially reconstituted clayey sands increased by a factor of 3-4 as penetration transitioned from undrained to fully drained conditions. These results are typically plotted in the form of the cone resistance ratio $q_{cnet}/q_{ref}$ versus the nondimensional velocity $V$, where $q_{cnet}$ is the net resistance obtained by subtracting the overburden stress $\sigma_{v0}$ from the measured cone resistance $q_c$ (i.e. $q_{cnet}=q_c - \sigma_{v0}$), $q_{ref}$ is the net cone resistance associated with undrained penetration, and

$$V = \frac{v.d}{c_v}$$

(5.1)

where $v$ is the penetration velocity, $d$ is the diameter of penetrometer (i.e. cone, in this case) and $c_v$ is the soil’s coefficient of consolidation. A typical S-shaped curve for this normalized response is shown on Figure 5.1. Recent work (Schneider et al., 2008) has also examined the influence of the overconsolidation ratio (OCR) and aging effects on the normalized resistance and excess pore pressure under different drainage conditions.

Despite the experimental evidences showing the effect of consolidation and
penetration rate on the tip resistance, this phenomenon has not been well-studied analytically or numerically. Many of the published numerical studies on cone penetration test were carried out using a total stress formulation that modeled either the drained or undrained response. The effect of consolidation, which might arise from partial drainage conditions caused by a combination of permeability and rate effects, was usually not considered. The work of Sheng et al. (1997) appeared to be one of the earliest reported analyses to incorporate both coupled consolidation effects and large deformation penetration from the free surface. Their analyses were carried out using ABAQUS, with frictional sliding at the soil-cone interface achieved via contact elements. The preliminary results presented in their paper showed that both positive and negative excess pore pressures were generated in the vicinity of the cone, and that the computed total resistance increased linearly with the friction angle. It is noted, however, that the calculation of contact frictional resistance in ABAQUS prior to version 6.6 was based on total stress, and not effective stress. Hence, frictional sliding contact computations conducted using ABAQUS version 6.5 or earlier were probably erroneous. In addition, the preliminary results reported by Sheng et al. (1997) did not include a detailed study of permeability or rate effects and their influence on the computed cone and soil responses.

Another study which incorporated consolidation effects was reported by Ahmadi (2000), who presented some preliminary results of the excess pore pressure response near the cone tip using the coupled stress flow feature in the finite difference code FLAC. The cone penetration was simulated by the method of
moving boundaries, which involved the systematic application of prescribed vertical and horizontal displacements to the soil nodes along the cone path. However, the cone was not explicitly modelled, nor was the cone-soil friction. The vertical displacement at the soil-cone interface was not computed, but prescribed based on observed results.

More recently, Silva et al. (2006) used finite element cavity expansion analysis with a coupled-consolidation formulation to investigate the effect of penetration rate on piezocone tests in clay. By simplifying the cone penetration process as a pure cavity expansion, the numerical challenges associated with modelling the complex soil-cone interaction were not considered. Furthermore, as the cone was not explicitly modelled, there is no direct output of cone tip resistance.

This chapter presents the results of a finite element study into the effects of cone penetration rate using coupled-consolidation and updated Lagrangian large-strain formulation. The analyses covered the full spectrum of responses, from drained to undrained conditions, during cone penetration in a clayey soil. Different drainage conditions were studied by varying the rate of cone penetration, and the computed results presented in the form of the normalized backbone curve. The influence of soil strength and stiffness on the shape of the normalized backbone curve was also studied. Finally, a method for the characterization of the backbone curves for different soil properties was proposed, based on the hyperbolic function proposed by Randolph and Hope (2004). This led to a method of deducing different soil properties from cone penetration test results.
5.3 Numerical modelling aspects

5.3.1 Model geometry

The software used for this study is ABAQUS/Standard V6.6. The geometry and boundary conditions of the axisymmetric finite element model are shown on Figure 5.2. The cone was modeled with an apex angle of 60° and a radius R of 0.5 m, which is much larger than the normal cone radius of 17.8 mm used in the field. However, it is consistent with the prototype radius of the centrifuge model cone reported in Randolph & Hope (2004), and therefore facilitates direct comparison of computed cone resistance and pore pressures with the reported centrifuge measurements. The effect of the penetrometer size can be accounted for by means of Randolph and Hope’s dimensionless velocity (Equation 5.1) and by expressing the cone resistance as the cone tip force normalized by the projected cross-sectional area. In addition, the use of a larger cone radius also moderates the requirements on the mesh density and grading, and allows the analysis to be conducted over a sufficiently large depth on a personal computer.

Four-noded bilinear elements with displacement and pore-pressure degrees-of-freedom were used for the soil domain. Preliminary trials using different element types indicated that this type of element is more stable than higher-order elements for coupled consolidation analysis involving large relative sliding movements at the penetrometer-soil interface.

The cone penetrometer was treated as a rigid body with infinite stiffness. At the start of the analysis, the cone tip was located just above the soil surface, with no
contact between the tip and soil, as shown on Figure 5.2(a). The cone tip was then pushed into the soil at a controlled rate to a depth of 60R. This method of modelling the penetration process differs from previous studies wherein the cone was pre-embedded under the free surface at the start of the analysis (e.g. Abu-Farsakh et al., 2003).

In a typical finite element analysis, the left boundary is the axis of symmetry and no lateral movement of the nodes on this boundary is allowed. Intuitively, however, one would expect soil along this boundary to flow around the penetrometer; this soil flow cannot be modeled if the left-hand boundary is set as the axis of symmetry. Secondly, points along the axis of symmetry have an initial radial co-ordinate of zero. Any lateral displacement from the axis of symmetry will therefore result in infinite radial strain, which will lead to numerical problems. To overcome this, the left boundary was offset slightly from the axis of symmetry to avoid these problems. In this study, the radial offset is 0.05R, in which R is the radius of the penetrometer. This approach is equivalent to prescribing a small initial cavity in the ground prior to penetration and has been used by Mahutka et al. (2006) to model pile penetration. Silva et al. (2005) also used a similar approach by starting the cavity expansion process from a finite radius. Preliminary trials indicated that reducing the initial cavity radius below 0.1R would not significantly alter the results of the analysis. At the start of the analysis, the soil at the left boundary was prescribed to be in contact with a smooth, rigid cylindrical surface, the function of which is to provide lateral support to the surrounding soil and maintain a K₀-geostatic
stress state prior to penetration. Drainage was permitted at all except the left boundary.

5.3.2 Large-sliding soil-cone interface

The interaction between soil and cone was modeled using surface-based contact algorithms with the finite-sliding tracking approach. The cone was modeled as a rigid body and the cone-soil interfaces were specified as a pair of contacting surfaces. ABAQUS allows overlapping but non-interacting surfaces to be specified; this allows the cone to overlap with the rigid cylindrical surface which supports the left boundary prior to penetration.

The contact constraint in the direction normal to the surface was enforced using the method of Lagrange multiplier. Soil-cone contact was assumed to be frictionless. As noted by Huang et al. (2004), it was difficult to isolate the shaft resistance from the total reaction force acting on the penetrometer when frictional effects were present. Also, it was found that the addition of friction at the soil-cone interface rendered the coupled analyses much more unstable in the present analyses. For these reasons, the present analyses were carried out assuming smooth soil-cone contact. If required, frictional effects may be incorporated by multiplying the computed cone resistance with a tip friction factor, following the approach of Huang et al. (2004).

While the non-overlapping condition was strictly enforced in the direction normal to the surfaces, arbitrary relative sliding, separation and re-closure between the
cone, the rigid cylindrical body and soil domain were allowed as the cone was pushed into the soil. This modeling approach differs from the conventional approach of modeling cone-soil contact, in which cone-soil interaction is approximated by imposing prescribed displacements boundary conditions or using joint elements which only allow infinitesimal relative displacements.

5.3.3 Large deformation formulation

As the cone advances, the soil in its vicinity experiences large deformation. To model such large deformation, the updated Lagrangian (UL) formulation was adopted in the present analyses. Furthermore, conventional infinitesimal strain measures might not be appropriate given the large deformations and distortions, and hence logarithmic strains were considered. The Arbitrary Lagrangian-Eulerian (ALE) method was not adopted in this study as this feature in ABAQUS could not be used for coupled consolidation analysis.

5.3.4 Elastic-plastic soil behavior

The soil was assumed to be isotropic and homogeneous and its behavior was characterized using the Drucker-Prager model. The elastic-perfectly-plastic Drucker-Prager model is defined as

\[ q - M p' - c' = 0 \]  

where \( q \) and \( p' \) are the deviator and mean effective normal stresses respectively. The parameter \( M \) is the friction coefficient. The parameter \( c' \) is the effective cohesion,
set to a nominally small value of 1 kPa here to avoid numerical instability. Preliminary analyses showed that the Drucker-Prager model is numerically more stable than the Mohr-Coulomb model when carrying out the present large deformation consolidation analyses in ABAQUS/Standard V6.6. Dijkstra et al. (2007) noted a similar stability issue with the Mohr-Coulomb model in analysing displacement pile installation, and hence adopted the Drucker-Prager model for their analyses. The Drucker-Prager model was reported give a cone resistance that is about 10% higher than that obtained using the Mohr-Coulomb model (Hu, 2003). Such a level of error is not considered significant in this study.

5.4 Analysis results

Figure 5.3 shows the calculated cone resistance profiles for different penetration rates, using the following soil parameters typical of clayey soil: (i) friction angle $\phi' = 23^\circ$, (ii) dilation angle $\psi = 0^\circ$, (iii) ratio of shear modulus to initial mean effective stress, hereafter termed “modulus ratio”, $G/p' = 35$, (iv) lateral stress coefficient $K_0 = 0.6$, (v) effective unit weight $\gamma' = 5.7 \text{kN/m}^3$, (vi) permeability $k = 5 \times 10^{-10} \text{m/s}$, (vii) effective Poisson’s ratio $\nu' = 0.3$. The modulus ratio used in this study is akin to Vesic’s (1972) rigidity index $I_r$. For a soil with a plasticity index of 50%, Skempton’s (1957) correlation between the undrained shear strength and the plasticity index yields a $s_u/\sigma'_{v0}$ ratio of 0.295, wherein $s_u$ is the undrained shear strength and $\sigma'_{v0}$ the in-situ effective overburden stress. Combining this with a modulus ratio of 35 leads to a rigidity index $G/s_u$ of 87. Using the modified Cam Clay model (Wroth &
Houlsby, 1985) would give a slightly lower rigidity index of about 74. These parameters are typical of those of a soft, normally consolidated clay.

### 5.4.1 Effect of different penetration rates

Figure 5.3 shows that for all penetration rates, the computed cone tip resistance ($q_c$) increases almost linearly with depth. Such a trend is consistent with a linearly increasing undrained shear strength profile and constant cone factor. The lowest cone resistances are obtained using the fastest penetration rates of 1, 0.1 and 0.01 mm/s, while the highest resistances are associated with the slowest penetration rates of 0.0001, $1.0 \times 10^{-5}$ and $1.0 \times 10^{-6}$ mm/s. The increase in penetration resistance as the penetration rate decreases is consistent with Randolph and Hope's (2004) observation.

Figures 5.4a-c show the excess pore pressure contours in the cone tip vicinity for penetration rates of 1, 0.001 and 0.0001 mm/s respectively, when the tip is at a depth of 40R (20 m). As shown on Figure 5.4a, the highest excess pore pressure around the cone tip is associated with the fastest penetration rate of 1 mm/s, and decreases with penetration rate (Figures 5.4b & c). In addition, although not shown here, the contours in Figure 5.4a are almost identical to those obtained using a penetration rate of 0.1 mm/s, suggesting that undrained condition prevailed at these two highest penetration rates.

The penetration rate commonly used in practice is about 20 mm/s, which is much faster than the highest rate of 1 mm/s considered in this study. However, the
The cone size adopted herein is much larger than that used for in-situ measurements. The effect of different cone sizes may be accounted for by using the non-dimensional velocity (Equation 5.1). A penetration rate of 20 mm/s for a 17.8 mm radius cone is therefore equivalent to a penetration rate of about 0.7 mm/s for the 500 mm radius cone modelled herein. Hence, the highest penetration rate used herein is consistent with that used to ensure undrained conditions for a typical field cone test.

5.4.2 Fully undrained penetration response

5.4.2.1 Comparison with strain-path predictions

Figures 5.5 & 5.6 compare the computed radial strain $\varepsilon_r$, the meridional shear strain $\varepsilon_{rz}$, and the maximum shear strain $\gamma_{\text{max}}$ in the cone vicinity at a depth of 40R, for a penetration rate of 1 mm/s, with the strain path results of Levadoux & Baligh (1980). As can be seen, both analyses produced similar radial strain ($\varepsilon_r$) and maximum shear strain ($\gamma_{\text{max}}$) contours. The meridional strain ($\varepsilon_{rz}$) shows larger discrepancies, with the finite element analysis yielding higher strain values ahead of and behind the cone tip. Levadoux & Baligh’s (1980) results also imply a much higher recovery of shear strain after the passage of the cone tip. For instance, along the vertical section AA’ in Figure 5.6b, there is a strain recovery of about 5% from the cone tip to the weak zone behind. This is much higher than the strain increment from the in-situ stress level to the point of yielding, which is typically less than 1% for most soils. It is unclear what causes this level of strain recovery in Levadoux & Baligh’s analysis. Teh and Houlsby (1991) noted that Levadoux & Baligh’s strain path analysis did not
completely satisfy all the equilibrium equations, even with iterative correction; this may be a possible reason.

5.4.2.2 Cone tip factor $N_c$

The cone tip resistance $q_c$ is commonly related to the undrained shear strength $s_u$ via a relation of the form

$$q_c = N_c s_u + \sigma_v$$

(5.3)

where $\sigma_v$ is the total overburden stress (e.g. Teh & Houlsby, 1991; Lu et al., 2004).

As the present study uses effective friction angle to characterize the soil strength in the Drucker-Prager model, the undrained strength $s_u$ at a given depth may be obtained using

$$s_u = \frac{1}{2} M p'$$

(5.4)

where $M$ is the friction coefficient $= 6\sin \phi'/(3-\sin \phi')$, and $p'$ is the mean effective stress. From Equations 5.3 and 5.4, the calculated cone tip factor may be expressed as

$$N_c = \frac{2q_{ref}}{Mp'}$$

(5.5)

in which $q_{ref} = q_c - \sigma_v$.

Figure 5.7 shows that, for a penetration rate of 1 mm/s, $N_c$ approaches a stable value of approximately 9.0, at a penetration depth of 24R (12 m) or greater. The stabilized cone factor shows good agreement with the analytical solutions of Teh & Houlsby (1991), Abu-Farsakh et al. (2003) and Lu et al. (2004), indicating that the present coupled analysis is able to capture the undrained response under rapid penetration.
5.4.2.3 Effect of soil stiffness and strength

Figure 5.8 shows the computed normalized undrained resistance \( \frac{q_{\text{ref}}}{p'} \) for different modulus ratios and friction angles. The dashed lines in this figure were obtained by combining Lu et al.’s (2004) relationship

\[
N_c = 3.4 + 1.6 \ln \left( \frac{G}{S_u} \right) - 1.9 \left( \frac{\sigma_{v0} - \sigma_{h0}}{2s_u} \right) \tag{5.6}
\]

with Equations 5.3 and 5.4 to yield

\[
\frac{q_{\text{ref}}}{p'} = M(2.25 + 0.8 \ln \frac{G}{p'} - 0.8 \ln M - \frac{2.85(1-K_0)}{(1+2K_0)M}) \tag{5.7}
\]

where \( \sigma_{v0} \) and \( \sigma_{h0} \) are the in-situ total vertical and horizontal stresses, respectively. Figure 5.8 shows reasonably good agreement between Equation 5.7 and the computed results.

Similarly, by plotting \( \frac{q_{\text{ref}}}{p'} \) against \( M \) as shown on Figure 5.9, a reasonably good match is obtained between the computed values and those from Equation 5.7 for modulus ratios ranging from 35 to 140. Furthermore, both the computed data and Equation 5.7 indicate an almost linear relationship between \( \frac{q_{\text{ref}}}{p'} \) and \( M \) for friction angles between 18° to 35°, with the extrapolated lines passing very close to the origin. Thus, if either \( \phi' \) or \( G/p' \) is known, Figures 5.8 and 5.9 or Equation 5.7 can be used to estimate the other unknown quantity from the undrained resistance.

5.4.3 Fully drained penetration response

In drained penetration, the effective cone resistance \( q'_c \) is often correlated to the in-situ effective vertical stress \( \sigma'_{v0} \) via

\[
q'_c = N_q \sigma'_{v0} \tag{5.8}
\]
wherein the cone factor $N_q$ has been derived using bearing capacity analysis (e.g. Durgunoglu & Mitchell, 1975), cavity expansion theory (e.g. Salgado et al., 1997; Yu, 2004) or numerical analyses (e.g. Hu, 2003; Huang et al., 2004; Ahmadi et al., 2005).

Figure 5.10 shows the computed cone factor in the present analysis for a penetration rate of 0.000001 mm/s, with the soil parameters used in obtaining the resistance profiles of Figure 5.3. This compares well with Yu’s (2004) results, which consider both cylindrical and spherical cavity expansion.

The calculated cone factors in Figure 5.10 are much smaller than those deduced from field measurements in cohesionless soils, which usually range from 20 to more than 100 (Lunne et al., 1997). This may be attributed to the low angle of friction and modulus ratio adopted in the analysis. As Figure 5.11 shows, higher friction angles and modulus ratios will lead to higher $N_q$ values. Moreover, these values also agree remarkably well with Yu’s (2004) analytical solution at low modulus ratios. For higher modulus ratios exceeding 100, there is greater divergence but the trend remains reasonably well-matched.

Huang et al. (2004) showed that the cone factor $N_q$ in cohesionless soils is also highly dependent on the dilation angle. Figure 5.12 shows the variation of computed cone factors $N_q$ with dilation angle, using parameters from Hu’s (2003) ALE analysis for the fully drained condition. Reasonably good agreement is obtained between the present results and those reported by Hu (2003).

Following the approach for undrained tests, the normalized drained cone
resistance $q_{\text{drained}}/p'$ is defined herein as

$$
\frac{q_{\text{drained}}}{p'} = \frac{q'_{c} - \sigma'_{s0}}{p'}
$$

(5.9)

As Figure 5.13 shows, for a given modulus ratio, the normalized drained resistance $q_{\text{drained}}/p'$ varies almost linearly with $M$, with the corresponding best fit lines passing very close to the origin. These lines are steeper than those shown on Figure 5.9 for the undrained condition.

The approximate linear trends passing through the origin shown on Figure 5.9 and Figure 5.13 indicate that, for a given modulus ratio, the resistance ratio $q_{\text{drained}}/q_{\text{ref}}$ is relatively independent of $M$, or the friction angle $\phi'$; this being also reflected in Figure 5.14. On the other hand, as Figure 5.15 shows, for a given friction angle, the resistance ratio increases significantly with the modulus ratio.

### 5.4.4 Undrained and drained plastic zones

Figure 5.16 shows the computed plastic radius $R_p$ and depth $Z_p$ associated with the plastic zone development in the tip vicinity due to cone penetration. For both the undrained and drained conditions, the computed plastic radii $R_p$ are quite close to those obtained from cylindrical cavity expansion.

### 5.4.5 Partially drained response

Figure 5.17 plots normalized cone resistances against non-dimensional velocities for the different penetration rates associated with the computed profiles of Figure 5.3. Following Randolph and Hope (2004), the normalized cone resistance $q_{\text{cnet}}/q_{\text{ref}}$ is
defined as the measured net cone resistance normalized by the reference (or undrained) resistance. Since the soil in this study is characterized using isotropic elastic parameters and coefficient of permeability, the coefficient of consolidation \( c_v \) can be deduced directly from the relationship:

\[
c_v = \frac{k}{m_v \gamma_w} = \frac{kD'}{\gamma_w}
\]  \hspace{1cm} (5.10)

in which \( k \) is the coefficient of permeability, \( m_v \) the coefficient of volume change of clay, \( \gamma_w \) the unit weight of water and \( D' \) the effective constrained modulus of the soil, which is given by

\[
D' = \frac{2G(1 - \nu')}{(1 - 2\nu')}
\]  \hspace{1cm} (5.11)

wherein \( \nu' \) is the effective Poisson’s ratio. Preliminary checks using finite element analyses of one-dimensional consolidation of a soil layer having properties identical to those used in the present penetration analyses showed that the \( c_v \)-values, evaluated using the root-time method, agreed closely with the values calculated using Equation 5.10.

Since the cone tip resistance is depth-dependent, the normalized cone resistances were calculated at three different depths, viz. 15m, 20m and 25m. As can be seen from Figure 5.17, the results obtained from different depths all fall along the same fitted curve. Following Randolph & Hope (2004) and Chung et al. (2006), the fitted curve can be represented by a hyperbolic function of the form

\[
\frac{q_{\text{net}}}{q_{\text{ref}}} = 1 + \frac{b}{1 + cV^m}
\]  \hspace{1cm} (5.12)

where \( b, c \) and \( m \) are constants with values of 1.1, 0.8 and 1.1 respectively. The
results indicate that the transition from undrained to drained response occurs roughly over a hundredfold decrease in non-dimensional velocity.

Equation 5.12 indicates that the normalized cone resistances approaches a value equal to \(1 + b\) as the non-dimensional velocity approaches zero corresponding to the drained response, which gives:

\[
b = \frac{q_{\text{drained}}}{q_{\text{ref}}} - 1
\]  

(5.13)

Using the linear best-fit correlation in Figure 5.14, it follows that

\[
b = \frac{q_{\text{drained}}}{q_{\text{ref}}} - 1 = 0.022 \frac{G}{p'} + 0.331
\]  

(5.14)

For the modulus ratio, \(G/p' = 35\), used in the present analyses, Equation 5.14 also results in \(b = 1.1\).

5.4.6 Effect of soil stiffness and strength on backbone curves

As Figure 5.18 shows, with a constant modulus ratio of 35, changes in friction angle do not result in much variation in the backbone curve. On the other hand, as Figure 5.19 shows, changes in modulus ratio result in much more significant variation in the backbone curve. This is consistent with the fact that the resistance ratio is more sensitive to the modulus ratio than friction angle, as indicated on Figures 5.14 and 5.15. As Figure 5.19 shows, undrained response is obtained for non-dimensional velocities greater than about 30. The results for each modulus ratio can be fitted using Equation 5.12, using values of the coefficients \(b\), \(c\) and \(m\) as shown on Figure 5.19. It is noted that the coefficients \(c\) and \(m\) remain relatively unchanged at
approximately 0.8 and 1.1, respectively.

Finnie and Randolph’s (1994) centrifuge model results for two calcareous soils indicate that the resistance ratio is approximately 10 for a predominantly silt sample, and 2 for a sand sample. As Figure 5.20 shows, Finnie and Randolph’s (1994) results for silt can be reasonably matched using a $G/p'$ ratio of 320 and a friction angle of 40°. Finnie and Randolph presented a fitted relation for the initial modulus $G_i$ which suggests that $G_i/p'$ can indeed be as high as 300 for effective confining pressure of less than about 50 kPa.

Finnie and Randolph attributed the lower resistance ratio of the sand sample to the high undrained resistance caused by dilation-induced negative pore pressures in the sand at low effective stress levels. Such dilatant behaviour cannot be effectively captured using the simple Drucker-Prager model adopted herein.

### 5.4.7 Comparison with Randolph and Hope’s (2004) centrifuge experimental results

As mentioned earlier, Randolph & Hope (2004) conducted centrifuge tests to examine the penetration rate effect in kaolin clay beds. A model cone penetrometer with 10 mm diameter and 60 degree apex angle was pushed into the clay ($\phi'=23°$) under a centrifugal acceleration of 100g. Under prototype conditions, the equivalent cone diameter was 1 m, which is identical to that used in the present study. As shown on Figure 5.19, Randolph and Hope’s (2004) centrifuge data show a similar trend as those obtained from the numerical analyses, and generally fall within the range
bounded by the computed backbone curves corresponding to modulus ratios between 35 and 105. Furthermore, Figure 5.21 shows good agreement between the computed and measured excess pore pressure ratio \( \Delta u/q_{c\text{net}} \) for modulus ratios of 35 and 70.

Figures 5.19 and 5.21 plot normalized quantities, and hence do not directly reflect \( q_{\text{ref}} \) and \( \Delta u \). As shown on Figures 5.22 and 5.23, under the fastest penetration rate of 1 mm/s, the undrained profiles of Randolph and Hope’s measured net cone resistance and excess pore pressure are narrowly bounded between the computed results for modulus ratios of 35 and 70.

**5.4.8 Effect of volumetric yielding**

To study the effect of plastic volumetric yielding, another series of analyses was conducted using the modified Drucker-Prager cap model in ABAQUS. This model (Figure 5.24) incorporates a compression cap, which expands with plastic volumetric strain \( \varepsilon_v^p \) according to the hardening rule

\[
\varepsilon_v^p = \frac{\lambda - \kappa}{1 + e_0} \ln \frac{p'}{p'_c}
\]

(5.15)

in which \( \lambda \) and \( \kappa \) are the isotropic compression and re-compression indices, respectively, and \( e_0 \) and \( p'_c \) are the reference void ratio and pre-compression pressure. The modulus ratio is related to \( \kappa \) via

\[
\frac{G}{p'} = \frac{2(1-2\nu)(1+e)}{2(1+\nu')\kappa}
\]

(5.16)

For \( e = 1.7 \) and \( \nu' = 0.3 \), the value of \( \kappa \) corresponding to \( G/p' = 35 \) is 0.035. The friction angle is 23°, and all other parameters are identical to those used to obtain the
results of Figure 5.3. As shown on Figure 5.25, the resistance ratio computed using the cap model (with $\lambda/\kappa = 5$) is about 15% smaller than the corresponding value without the cap.

Figure 5.26 shows the effect of $\lambda/\kappa$ and modulus ratio on the resistance ratio. For a given modulus ratio, larger $\lambda/\kappa$ ratios result in smaller resistance ratio. The results for different $\lambda/\kappa$ are approximately parallel to one another, including the “no-cap” case. Figure 5.27 plots, for different $\lambda/\kappa$ and modulus ratios, the reduction factor $F_r$, defined as the ratio of the resistance ratio obtained from a cap analysis (for a given $\lambda/\kappa$ and modulus ratio) to the “no-cap” case. The reduction factor does not appear to be significantly affected by the modulus ratios, and may be reasonably fitted by the line shown in Figure 5.27.

In many clayey soils, the $\lambda/\kappa$ ratio lies between 3 and 5 (Schofield & Wroth, 1968). For such soils, Figure 5.27 suggests that the reduction factor varies from 0.95 to 0.85. In other words, volumetric yielding will decrease the resistance ratio computed using the original Drucker-Prager model without cap by at most 15%. Although this factor can be incorporated into the backbone curves, the decrease may not be significant, given the uncertainty over the modulus ratio. For this reason, the results and discussions in this study are based on the original Drucker-Prager model without cap.
5.4.9 Effect of modulus profile

As shown in Figure 5.28, the effect of modulus profile is examined by comparing the results of three cases, viz. constant modulus ratio, $G$ increasing linearly with depth from a non-zero modulus at ground surface, and $G$ constant with depth. These three profiles are chosen so that they have a shear modulus of 5400 kPa at a depth of 18 m. For each profile, three penetration rates, namely 1 mm/s, 0.001 mm/s and 0.000001 mm/s, corresponding to fully undrained, partially drained, and fully drained conditions respectively, were studied.

As Figures 5.29a-c show, for all three penetration rates, the computed cone resistances from different modulus distributions intercept at a depth close to 18 m, where they share a common modulus. This suggests that it is the local soil modulus, rather than the overall modulus profile, that has primary influence on the cone tip resistance. This finding suggests an approach for deducing the backbone curve at any depth within an arbitrary soil profile, as follows:

i) For any arbitrary stiffness profile, calculate or estimate modulus ratio at the depth of interest.

ii) For this value of modulus ratio, the coefficient $b$ may be estimated from Equation 5.14, while $c = 0.8$, $m = 1.1$; this defines the backbone curve given by Equation 5.12.

The above procedure hinges on two important observations. The computed cone resistance is governed by the local stiffness and strength of soil. Also the backbone curve appears to be quite insensitive to variations in the friction angle. To
illustrate the applicability of the procedure, consider a uniform soil medium with a constant shear modulus of 5400 kPa, corresponding to a depth-decreasing modulus ratio. The friction angle is 23°, and other soil properties are identical to those previously considered. The data points on Figure 5.30 show normalized cone resistances for different non-dimensional velocities, corresponding to depths of 15, 20 and 25m, computed from finite element calculations. The dashed and solid lines show the fitted hyperbolic curves for the three depths, obtained via the procedure described above. As Figure 5.30 shows, the deduced backbone curves provide good fits to the computed results.

5.5 Application to soil properties evaluation

The results presented above also point the way towards using cone penetration tests to evaluate the coefficient of consolidation, the friction angle and the modulus ratio. This is illustrated herein using the data of Kim (2005) and Kim et al. (2008), which were obtained from cone penetration tests conducted at different velocities at two sites. Site 1 was located on the west side of a bridge over Bachelor’s run on State Road 18 (SR 18) in Carroll County, Indiana. Site 2 is located on the north side of Oliver Ditch on State Road 49 (SR 49) in Jasper County, Indiana. Details on soil stratigraphy and properties are given in Kim (2005). Key soil information is provided on Table 5.1. The friction angles for the soils at the two sites were not reported, but they are estimated to be between 30° to 35°, based on plasticity index correlations (Bjerrum & Simons, 1960; Kanji, 1974).
The cone resistance versus penetration data presented by Kim (2005) and Kim et al. (2008) are shown on Figure 5.31, for two depths at Site 1 and one depth at Site 2. For each depth, the corresponding $q_{\text{cnet}}$ values, together with the penetration velocities $v$, can be used to “solve” for the unknown quantities in the modified form of Equation 5.12, which is

$$\frac{q_{\text{cnet}}}{q_{\text{ref}}} = 1 + \frac{b}{1 + 0.8 \left( \frac{vd}{c_v} \right)^{1.1}}$$  \hspace{1cm} (5.17)

In Equation 5.17, $q_{\text{cnet}}$ and $v$ are known from field measurements, and $d$ is the cone diameter. To solve for the three unknowns, viz. $q_{\text{ref}}$, $c_v$, and $b$, a curve-fitting approach utilizing the field data from Kim (2005) is adopted herein. Figure 5.32 shows the results of the curve-fitting exercise for the data points associated with the three depths. Comparison of Tables 5.1 and 5.2 shows that the coefficient of consolidation obtained from this proposed method agree reasonably well with values from 1-D consolidation tests. It also correctly shows that the coefficient of consolidation at Site 2 is one order smaller than that at Site 1. The estimated $G/p'$ values of about 100 are reasonable for the clayey silts encountered at the two sites. The friction angle of $30^\circ$ obtained for Site 1, based on $K_0 = 0.5$ and a normalized undrained resistance $q_{\text{ref}}/p'$ of about 6, is consistent with available published correlations for a plasticity index of about 10. However, the back-calculated friction angle for Site 2 is greater than $40^\circ$, due to a high normalized undrained resistance exceeding 10. The fact that the normalized undrained resistance for Site 2 is approximately doubled that of Site 1 is unusual given that the index properties of the
soil at the two sites are broadly similar. One possible reason for this is the possibility of significant dilation in the soil at Site 2. Kim’s (2005) triaxial test data indicate that a soil sample extracted from a depth 12.8m at Site 2 has an $\frac{s_u}{\sigma' v_0}$ ratio of 0.76. This is much higher than the corresponding value of 0.46 for a sample extracted from a depth of 9.45m at Site 1, which had an over-consolidation ratio of between 1 and 1.2. Kim (2005) did not provide any data relating to the over-consolidation ratio for the Site 2 soil, but the higher $\frac{s_u}{\sigma' v_0}$ ratio for Site 2 may be indicative of a higher over-consolidation ratio and greater tendency to dilate. Finnie and Randolph (1994) also reported high $q_{ref}$ from a model foundation on sand, which was attributed to dilation. Since the above computations were conducted assuming zero angle of dilation, they are probably not applicable to highly dilative soils.

5.6 Concluding remarks

This chapter reports the results of coupled consolidation analyses to study the effects of penetration rates using ABAQUS/Standard V6.6. The modulus ratio $G/p'$, together with the friction angle, were used to characterize the cone resistance in an effective stress framework. The results for the limiting drained and undrained conditions were shown to agree well with published analytical solutions. Both the drained and undrained net cone resistances were found to increase with modulus ratio and friction angle. On the other hand, the ratio of the drained to undrained net cone resistances was found to increase only with modulus ratio, but was relatively
insensitive to friction angles up to about 35°.

The normalized backbone curves for different drainage conditions also agree well with the centrifuge results of Randolph and Hope (2004). The effect of the modulus ratio also provides a possible explanation for the high drained/undrained resistance ratio observed by Finnie and Randolph (1994) for calcareous silt. The computed backbone curve at any depth was found to depend primarily on the modulus ratio at that depth, and not the overall profile. In other words, the penetrometer test can provide a good reflection of the local soil properties around the cone tip. A simplified procedure was proposed to derive the backbone curve, for a given modulus ratio, using the hyperbolic fitting function proposed by Randolph and Hope (2004).

Finally, an application example is included in this chapter to show how the results may be used to derive the modulus ratio, angle of friction and coefficient of consolidation from field-measured cone resistances and known penetration rates. Comparison with field data at two sites provided by Kim (2005) and Kim et al. (2008) shows that the soil parameters estimated using the proposed method are consistent with the measured and correlated values from the reported laboratory data for one site, but appears to overestimate the angle of friction at the other. The laboratory information suggests that the soil at the latter site may exhibit significant dilation, the effects of which are not considered in this study.

The discussion above shows that the cone penetrometer test can not only be used to derive the undrained shear strength (for undrained tests) or cone factor (for drained tests). By conducting tests over an appropriate range of penetration rates,
parameters relating to the modulus, permeability and consolidation properties of the soil can also be inferred. Finally, it should be noted that the results of this study may not be applicable to highly dilative soils.

As stated earlier, this study on cone penetration serves as a precursor to the subsequent analysis of the sand compaction pile problem. The foregoing discussions and analyses validate the feasibility of the proposed numerical approach which uses ABAQUS/Standard V6.6 to solve deep penetration problems.
Table 5.1 Soil Properties for SR 18 and SR 49 (data from Kim (2005)).

<table>
<thead>
<tr>
<th>Site</th>
<th>Depth (m)</th>
<th>Sand (%)</th>
<th>Silt (%)</th>
<th>Clay (%)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>$c_v$ (m$^2$/s)*</th>
<th>$\phi'$ (°)**</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.5 – 10</td>
<td>12.1</td>
<td>76.8</td>
<td>11.1</td>
<td>23</td>
<td>26.3</td>
<td>19.5</td>
<td>6.8</td>
<td>4.72 x $10^{-6}$ (9.5m - 9.8m)</td>
<td>30-35</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.91 x $10^{-6}$ (9.8m - 10m)</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>13 - 14</td>
<td>15</td>
<td>64</td>
<td>21</td>
<td>23</td>
<td>21</td>
<td>12</td>
<td>9</td>
<td>3.64 x $10^{-7}$</td>
<td>30-35</td>
</tr>
</tbody>
</table>

* From 1-D consolidation tests (root-time method)
** Friction angles were not reported by Kim (2005). The reported range of 30 to 35° is estimated from correlations in the present study.

Table 5.2 Back-fitted parameters and estimated soil properties for SR 18 and SR 49

<table>
<thead>
<tr>
<th>Site (Depth)</th>
<th>$q_{ref}$ (kPa)</th>
<th>$c_v$ (m$^2$/s)</th>
<th>b</th>
<th>$G/p'$ from Equation 5.13</th>
<th>$\phi'$ (°) from Figure 5.8+</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site 1 (9.5m – 9.8m)</td>
<td>600</td>
<td>3 x $10^{-6}$</td>
<td>2.5</td>
<td>~100</td>
<td>30</td>
</tr>
<tr>
<td>Site 1 (9.8m – 10m)</td>
<td>600</td>
<td>8 x $10^{-6}$</td>
<td>2.5</td>
<td>~100</td>
<td>30</td>
</tr>
<tr>
<td>Site 2 (13m – 14m)</td>
<td>1200</td>
<td>3 x $10^{-7}$</td>
<td>2.3</td>
<td>~90</td>
<td>&gt;40</td>
</tr>
</tbody>
</table>

+ Based on $q_{ref}/p' = 6$ for Site 1 and $q_{ref}/p' >10$ for Site 2
Figure 5.1  Effect of penetration rate on cone resistance (after Randolph & Hope, 2004).

Figure 5.2  Axisymmetric finite element mesh for analysis: (a) initial mesh (b) deformed mesh after cone insertion.
Figure 5.3  Calculated cone resistance profiles for different penetration rates.

Figure 5.4  Computed excess pore pressure contours for different penetration rates.
Figure 5.5  Computed strain contours from the present FEM analysis at a penetration depth of 40R.
Figure 5.6 Strain contours during the penetration of a 60° cone obtained using the strain path method (after Levadoux & Baligh, 1980).
Figure 5.7  Comparison of analytical predictions and calculated $N_c$ for undrained condition.

Figure 5.8  Variation of normalized undrained cone resistance with $G/p'$. 
Figure 5.9  Variation of normalized undrained cone resistance with M.

Figure 5.10  Comparison of analytical predictions and calculated $N_q$ for drained condition. The factor F in Yu's (2004) results is a plastic zone shape factor.
Figure 5.11  Variation of $N_q$ with friction angle $\phi'$. 

Figure 5.12  Cone factors $N_q$ for various dilation angles $\psi$ using Hu’s (2003) parameters ($\phi' = 45^\circ$, $G/p' = 177$, $\nu' = 0.3$, $K_0 = 0.43$, $c' = 2$ kPa).
Figure 5.13  Variation of normalized drained cone resistance with M.

\[ q_{\text{drained}} = 0.022(G/p') + 1.331 \]

Figure 5.14  Variation of \( q_{\text{drained}} / q_{\text{ref}} \) with \( G/p' \).
Figure 5.15  Variation of $q_{\text{drained}}/q_{\text{ref}}$ with $\phi'$.  

Figure 5.16  Size of the elasto-plastic zone.
Figure 5.17  Normalized cone resistance versus non-dimensional velocity.

Figure 5.18  Normalized cone resistance versus non-dimensional velocity for various friction angles ($G/p' = 35$).
Figure 5.19  Normalized cone resistance versus non-dimensional velocity for various G/p' ratios.

Figure 5.20  Comparison with Finnie & Randolph (1994) experimental data.
Figure 5.21  Comparison between computed and experimental excess pore pressure ratios $\Delta u / q_{cnet}$.

Figure 5.22  Comparison between computed and experimental net cone resistance profiles during undrained cone penetration ($\phi' = 23^\circ$).
Figure 5.23  Comparison between computed and experimental excess pore pressure profiles during undrained cone penetration ($\phi' = 23^\circ$).

Figure 5.24  Modified Drucker-Prager/Cap model and its hardening curve. The hardening curve in the inset is obtained using $\kappa = 0.035$ and $\lambda = 5\kappa = 0.176$. 
Figure 5.25  Effect of cap on normalized cone resistance versus non-dimensional velocity plots.

Figure 5.26  Resistance ratio versus $G/p'$ for different $\lambda/\kappa$ ratios.
Figure 5.27  Effect of $\lambda/\kappa$ on the normalized cone resistance reduction ratio $F_r$.

Figure 5.28  Three idealized soil modulus distributions (G profiles).
Figure 5.29  Comparison of cone resistance profiles under different drainage conditions for three idealized soil modulus profiles.

Figure 5.30  Computed vs predicted backbone curves at three depths in a uniform soil with constant $G = 5400$ kPa and $\phi' = 23^\circ$. 
Figure 5.31 Measured normalized cone resistance at various penetration speeds (Kim, 2005; Kim et al., 2008).
Figure 5.32  Fitting of normalized field data from Kim (2005) and Kim et al. (2008) to solve for unknown parameters.

(a) Site 1 (SR18) : 9.5 m - 9.8 m
Fitting parameters:
$q_{ref} = 600 \text{ kPa}$
$c_v = 3 \times 10^{-6} \text{ m}^2/\text{s}$
$b = 2.5$

(b) Site 1 (SR18) : 9.8 m - 10 m
Fitting parameters:
$q_{ref} = 600 \text{ kPa}$
$c_v = 8 \times 10^{-6} \text{ m}^2/\text{s}$
$b = 2.5$

(c) Site 2 (SR49) : 13 m - 14 m
Fitting parameters:
$q_{ref} = 1200 \text{ kPa}$
$c_v = 3 \times 10^{-7} \text{ m}^2/\text{s}$
$b = 2.3$
Chapter 6: FINITE ELEMENT ANALYSIS OF SAND COMPACTION PILE INSTALLATION

6.1 Overview

This chapter presents the details and results of a series of numerical analyses conducted to study sand compaction pile (SCP) installation, and its impact on the surrounding clay. The installation of SCP was modeled numerically with ABAQUS/Standard V6.6, which has been shown to work well for the cone penetration problem studied in Chapter 5.

The numerical model and the procedures for simulating the SCP installation is first described, followed by discussions and interpretations of the computed results. The results are discussed in two parts, corresponding to the short-term soil response during SCP installation, and the long-term post-installation soil response. The first part examines the changes in soil stresses and strains at various stages during SCP installation, in which the computed total stresses and pore pressures histories are compared with centrifuge measurements. In the second part, the post-installation soil behavior is examined, focusing on soil strength improvements with time.

6.2 Finite element model

The SCP installation process in the centrifuge consists of casing insertion to the desired column depth, followed by withdrawal together with sand injection. The casing insertion is analogous to the cone penetration process, and was numerically simulated in a similar manner as described in Chapter 5. The additional challenge in modeling the SCP installation arises from the injection and compaction of sand during
the withdrawal phase, which will be described shortly.

6.2.1 Model geometry and boundary conditions

The axisymmetric finite element model is shown on Figure 6.1. The geometrical dimensions, boundary conditions, and material properties adopted in the numerical model are consistent with the dimensions of the prototype-equivalent of the centrifuge model discussed in Chapter 4. A domain of soil measuring 12 m in radial extent and 5 m in depth was modelled. Figure 6.1 also shows a 0.33 m radius ($R_c$) steel casing initially suspended above the soil surface, and a 0.5 m radius ($R_s$) sand compaction pile initially located beneath the soil body. These were both modelled as rigid cylindrical bodies.

Modeling of the sand compaction pile installation was performed in two stages, as depicted on Figure 6.2. In the first stage, the steel casing was jacked into the soil from above to simulate the casing installation process. During this phase, the left boundary of the soil domain was progressively displaced to the right due to the intrusion of the rigid casing (Figure 6.2a), up to a maximum of 0.33 m with the casing fully installed (Figure 6.2b). In the second stage, the steel casing was progressively withdrawn, accompanied by simultaneous injection of sand from the tip of the casing into the surrounding clay. Physically, the injected sand pushes and displaces the clay to form a sand column of a larger diameter than the steel casing. Numerically, this process was simulated by retracting the steel casing and simultaneously up-thrusting the rigid cylinder representing the sand compaction pile from the bottom up (Figure
6.2c), thereby causing further expansion of the cavity. The second stage displaces the left boundary of the soil domain further from 0.33 m to 0.5 m, the final radius corresponding to that of the newly formed sand compaction pile (Figure 6.2d).

The rightmost boundary of the soil domain was constrained against any horizontal movements, but allowed to slide freely in the vertical direction. Strictly speaking, the left boundary should coincide with the axis of symmetry prior to the insertion of the casing. However, such a condition would require constraining the nodes located on the axis of symmetry from any radial displacements and any outward movement of the nodes lying on the axis of symmetry would result in infinite radial strain. To overcome this difficulty, the method proposed in Chapter 5 was used, wherein the nodes along the leftmost boundary were given an initial offset of 0.02 m from the axis of symmetry and allowed to displace laterally and vertically during jack-in and sand feed. As discussed in Chapter 5, this is equivalent to creating a small cylindrical cavity with an equivalent radius of 0.06 \( R_c \) or 0.04 \( R_s \). To maintain geostatic conditions prior to casing jack-in, the small cylindrical cavity was supported by a smooth rigid cylindrical body, which did not interact with the casing and sand pile.

The bottom boundary of the model also requires special consideration. In the centrifuge experiments, the kaolin clay bed was underlain by a dense sand layer, which may be considered permeable but relatively incompressible. The conventional approach is to treat the base of the clay layer as the lower boundary of the finite element mesh and restrain it both vertically and horizontally. However,
such a condition cannot be applied in the present analysis, since the nodes along the base have to be free to displace in order to accommodate soil movements generated by the downward penetration of the casing, as well as the upward penetration of the sand compaction pile. To do so, the base of the clay layer was modeled to be resting on a rigid base, as shown on Figure 6.1.

In ABAQUS/Standard V6.6, the interaction between the free surfaces and the rigid bodies and base takes place through contact surfaces. Four pairs of contact surfaces were defined in this model to deal with the following:

i) the interaction between the steel casing and the adjacent soil during the casing penetration stage,

ii) the interaction between the sand compaction pile and the adjacent soil during the up-thrusting of the sand pile.

iii) the interaction between the rigid cylinder supporting the initial cavity and the left boundary of the soil domain, to prescribe the slight offset from the axis of symmetry, and

iv) the interaction between the unrestrained lower boundary of the soil domain and the underlying rigid support.

The first three pairs of contact surfaces were assumed to be smooth, while the last pair, at the interface between the lower boundary and rigid base support, was modeled as rough. The frictional behavior was approximated using the conventional Coulomb friction model with a coefficient of 0.42, derived from the friction angle of kaolin clay. In reality, there is likely to be some friction between the steel casing and soil, which
may cause some down-drag on the latter. However, as will be shown later, the introduction of friction onto the casing-soil interaction does not significantly affect the calculation results, whose influence tended to be limited and localized. In addition, the inclusion of friction was found to render the analysis more unstable and extremely difficult to converge. Hence, the casing-soil and sand-pile-soil interactions were both assumed to be frictionless in this study.

In the centrifuge experiments, the relatively permeable thin sand layer allows drainage along the lower boundary of the clay. Hence, in the numerical model, the lower boundary of the clay layer is assigned as a drainage boundary for consolidation analysis. The same condition applies to the top surface of the clay layer.

The drainage condition along the left boundary of the soil domain requires special consideration. This boundary was initially treated as undrained, prior to and during the installation of the steel casing. However, during the subsequent formation of the sand compaction pile, the nodes along this boundary progressively came into contact with the highly permeable pile as the latter was up-thrusted. As such, during this stage, it was necessary to continuously re-define the left boundary conditions so that drainage was permitted along the corresponding nodes of the left boundary in contact with the up-thrusting pile.

6.2.2 Model discretization

The soil domain was discretized using four-noded quadrilateral elements with four integration points. In addition to radial and vertical displacement components, each
node has an additional pore pressure degree of freedom for coupled consolidation analysis. As explained in Chapter 5 for the cone penetration test, the choice of element type is influenced by considerations of numerical stability and convergence. In Chapter 5, it has been shown that four-node bilinear elements are capable of accommodating large scale contact sliding, and are less susceptible to numerical instability associated with element distortion.

The discretized finite element mesh is shown on Figure 6.3. To reduce computational requirements, a coarser grading was adopted for the right half of the model, where the stress and deformation gradients are not as steep. The soil domain shown on Figure 6.3 contains 7102 nodes and 6930 elements. A typical element in the left half of the model is 0.1 m wide and 0.048 m thick, which is roughly one-third and one-seventh of the radius of the steel casing ($R_c$) respectively. The use of elongated elements (aspect ratios of about 2) works well for the present analyses. This may be attributed to the fact that, during penetration, the soil in the vicinity of the steel casing and SCP deformed primarily in radial compression, leading to a decrease in radial dimensions of the elements around the steel casing.

The steel casing and sand compaction piles were also modeled using 4-noded quadrilaterals. Pore pressure degrees of freedom, however, were not required for these elements. As will be explained afterwards, the steel casing and sand compaction pile were modeled as rigid bodies in this investigation. Hence, they could be modelled as a single rigid body, instead of finite elements. However, the latter approach was used herein because it allowed contact conditions along the casing
and SCP shafts to be varied (on an element by element basis); this would not have been possible with a single rigid body. This was useful in some computational cases, where the shape of casing might need to be adjusted in the midst of calculation, for instance, to replicate the densification of surrounding sand layer.

As shown on Figures 6.1 and 6.3, some alterations were made to the tip geometries of the steel casing and the sand compaction pile in the finite element model. Instead of cylindrical ends, the lower face of the steel casing was made conical while the upper face of the sand compaction pile was slightly tapered and rounded. Such modifications to the geometry were necessary to allow penetration of the casing and pile through the first several layers of elements without generating excessive distortions, which was found to lead to numerical instability and premature termination of the calculations. During penetration, the soil elements near the casing or pile tip will be simultaneously displaced downward and sideways to accommodate the displacement induced by the penetrator. This process results in large element distortions in the vicinity of the casing or pile tip, the severity of which was found to be mitigated to some extent by the use of tapered or rounded surfaces. Figure 6.4a shows the deformed mesh obtained using a casing with a conical tip angle of approximately 60°. It shows that element distortions near the casing tip are relatively well controlled and not too excessive. In contrast, the use of a flat end without any taper (Figure 6.4b), causes severe distortions in the elements beneath the casing, which usually leads to premature termination of the analysis. For the same reason, the rounded end of the sand compaction pile helped prevent excessive mesh
distortion as it is up-thrusted into the clay layer. Such minor alterations to the geometries are driven by considerations of numerical stability. They are not expected to significantly affect the overall response of the soil, as their influence is quite localized and largely limited to the immediate vicinity of the penetrator tips.

The Drucker-Prager soil model was adopted to simulate behavior of the kaolin clay used in the centrifuge tests. The same model was used to study the cone penetration problem in Chapter 5. In the present analyses, the soil parameters adopted are: (i) effective friction angle $\phi' = 23^\circ$, (ii) dilation angle $\psi = 0^\circ$, (iii) modulus ratio, $G/p' = 20$, (iv) lateral stress coefficient $K_0 = 0.6$, (v) effective unit weight $\gamma' \approx 6.1 \text{kN/m}^3$, (vi) permeability $k = 2 \times 10^{-8} \text{m/s}$, (vii) effective Poisson’s ratio $\nu' = 0.33$. The preceding parameters are chosen in accordance with kaolin clay properties reported by Goh (2003) and Purwana et al. (2005).

Compared to the clay, the deformation of the steel casing may, for all practical purposes, be considered negligible. Hence, the actual properties assigned to the casing are not critical, as long as it is modeled as being much stiffer than the soft soil. Trial runs indicated that there is essentially no influence on the soft soil behavior once the Young’s modulus $E$ of the casing exceeds $1 \times 10^7 \text{kPa}$. In the present study, the metal casing is assigned a high $E$ value of $2 \times 10^{10} \text{kPa}$.

The deformation of the sand within the sand compaction pile is likely to be extremely complex and is not within the scope of the present research. Hence, like the casing, it was also treated as an almost rigid body and assigned a similarly high $E$ value of $2 \times 10^{10} \text{kPa}$. Numerically, the effect of introducing such a rigid body is to
impose soil displacements equal to the expected radius of the sand compaction pile.

6.2.3 Modeling procedure

The numerical modeling of the SCP installation process involves several stages, as previously discussed and illustrated on Figure 6.2. These stages, which include the steel casing jack-in (Figure 6.2a & b) and subsequent withdrawal with simultaneous sand injection (Figure 6.2c & d), are designed to replicate the centrifuge experimental procedure. In order to study consolidation effects, the rate of casing installation and withdrawal prescribed in the numerical model, as well as the rate of formation of the sand compaction pile, are consistent with those used in the experiments.

In the numerical model, the completion of SCP installation was followed by an additional phase, in which the excess pore pressures generated by SCP installation were allowed to dissipate to negligible values. This was carried out to examine the post-installation consolidation of the soil.

6.3 Computed soil responses during SCP installation

In accordance with the modeling sequence, the numerical results will be presented in two parts as follows:

i) soft ground behaviour during SCP installation is covered in this section, and

ii) post-installation consolidation behavior of the soil will be addressed in the next section.
6.3.1 Soil deformation and strain

Figure 6.5a shows the computed deformed mesh at the instant when the casing penetrates to the 2.5m depth i.e. mid-depth. It is noted that only the first three to four columns of elements adjacent to the casing experienced significant shearing and radial compression. Further away, the elements were not as severely distorted, and generally retained their rectangular shapes after casing insertion. Near the surface, there was some heaving of the soil adjacent to the casing, which is expected.

The zone of severe element distortions may be linked to the notion of a residual shear plane. The casing jack-in is analogous to the installation of a displacement pile, in which a thin layer of soil adjacent to the pile is subjected to considerable remoulding, leading to the formation of a residual shear plane (Bond & Jardine, 1991; Randolph, 2003). The pattern of mesh distortions in the vicinity of the casing, as shown on Figure 6.5a, is consistent with the presence of a residual shear plane surrounding the driven pile shaft.

The notion of a residual shear plane is further examined in Figure 6.6a, which depicts the shear strain contours ($\varepsilon_{rz}$) during the casing jack-in at the same instant as before. The horizontal and vertical axes correspond to the respective radial and vertical distances along the soil domain normalized by the casing radius $R_c$ (0.33 m). As shown on Figure 6.6a, computed shear strains of about 10 to 20% were concentrated in a relatively narrow band surrounding the casing during jack-in, which is consistent with the formation of a shear plane in this zone.

Figure 6.7a shows the computed radial strain ($\varepsilon_r$) contours during casing
installation. As shown on the graph, some tensile strains developed beneath the casing tip during jack-in. The existence of such a tensile bulb was also alluded to by Levadoux & Baligh (1980), when using the strain path method to study the mechanism of cone penetration. Above the conical tip, the radial strains developed concentrically around the casing shaft, the influence of which extended over a larger zone than the shear component (cf. Figure 6.6a). This suggests that radial compression is the dominant mode of deformation for elements further away from the casing.

As described earlier, the injection of sand to form the SCP follows after the casing reaches the desired depth, which is the base of the clay layer in the present study. Numerically, this was simulated by pushing the cylindrical body representing the sand compaction pile to penetrate the soil from the bottom up, with the casing being withdrawn concurrently. Figure 6.5b shows the deformed mesh resulting from casing withdrawal with simultaneous pile intrusion, when the SCP is formed up to the mid-depth. Compared to the earlier deformed mesh resulting from casing insertion (Figure 6.5a), the elements have now been further compressed to accommodate the larger diameter sand compaction pile.

The computed shear strain ($\varepsilon_{rz}$) and radial strain ($\varepsilon_{r}$) contours at the same instant are presented in Figure 6.6b and Figure 6.7b. Both the vertical and radial axes in graphs are normalized by the SCP radius $R_s$ (0.5 m). As shown on Figure 6.7b the computed radial strains were further increased by the introduction of the sand compaction pile. In contrast, the computed shear strains during SCP formation were
noticeably smaller than those obtained during casing jack-in. This phenomenon may be understood by examining the shear mechanisms associated with casing jack-in and SCP formation. During casing insertion, the elements adjacent to the casing shaft were sheared downwards (Figure 6.5a). On the other hand, the subsequent penetration of the SCP from the bottom of the soil layer caused the elements to be sheared upwards (Figure 6.5b). Hence, the shear strains generated during casing jack-in were partially offset by the shear strain reversal caused by the subsequent SCP formation, resulting in a reduction of the shear-affected zone (Figure 6.6b).

### 6.3.2 Soil stresses and pore water pressure

While the present calculations can provide information on a variety of stress components, the present discussion will focus only on the computed effective radial stress ($\sigma'_r$) and total pore water pressure ($u$).

Figures 6.8a and b show the effective radial stress ($\sigma'_r$) contours as the casing penetrate and withdraw to the mid-depth. The initial geostatic radial stress field (shown as dashed lines) is superimposed for reference. Figure 6.8a shows that, during jack-in, there is some decrease in radial stresses within a small bulb-shaped zone beneath the casing tip. This is consistent with the tensile strain bulb highlighted earlier on Figure 6.7a. The subsequent passage of the tip resulted in the soil being pushed outward, thus generating compressive radial stresses as the soil was radially displaced to make way for the casing. Soil next to the casing shaft also appears to experience some stress relief after the passage of the casing shoulder.
Figure 6.8b shows the computed radial stress fields during the subsequent casing withdrawal and pile formation. The simulation of pile formation as an upward penetration process (involving a larger diameter penetrator) resulted in further increases of the radial stresses, compared to Figure 6.8a.

Figures 6.9a and b show the total pore pressure (u) contours during casing jack-in and subsequent pile formation. The initial hydrostatic pore pressures are shown as dashed lines on graphs. Figure 6.9a shows that, during casing jack-in, significant positive excess pore pressures developed within a bulb-shaped zone surrounding the conical section, which extended some distance up the sleeve. The total pore pressure distributions during pile formation and casing withdrawal are shown on Figure 6.9b. The up-thrusting pile progressively enlarged the cylindrical cavity, causing the further build-up of pore pressure. By the end of installation, a widely extended excess pore pressure field had accumulated in the soil domain. Figure 6.10a illustrates the computed excess pore pressure (Δu) radial distribution which was recorded at the end of SCP installation and extracted from four representative levels namely 1m, 2m, 3m and 4m below soil surface. Recall that the soil nodes which progressively came into contact with the upward penetrating sand column were allowed to drain during SCP formation. The edge of the sand column, therefore, served as a drainage boundary. Its influence on excess pore pressure dissipation is clearly shown on Figure 6.10a, where the computed excess pore pressures at the pile edge (r = Rs) were zero. Beyond the edge, the computed pore pressures increased quite rapidly to their maximum values at r ≈ 2Rs. For r > 2Rs,
the pore pressures decreased with distance, with little or no excess pore pressure beyond $7R_s$. The preceding set of data were further normalized with respect to their corresponding shear strengths ($\Delta u/s_u$), and plotted in Figure 6.10b. Also included in the graph are analytical solutions from the undrained cylindrical cavity expansion (Randolph & Wroth, 1979). Favorable agreement is observed for data $r > 2R_s$, where, as explained above, appeared to be unaffected by the presence of drainage boundary. On the other hand, apparent discrepancy is existent within the short distance (i.e. $r=1-2R_s$), resulting from the permeable sand column edge.

6.3.3 Comparison of ABAQUS results with centrifuge data using kaolin clay

As described in Chapter 4, centrifuge experiments were conducted to simulate SCP installation, in which the radial stress and pore pressure histories were monitored at selected locations within the kaolin clay bed. In this section, the corresponding results from the ABAQUS analyses are presented and compared with the centrifuge measurements.

In the experiments, total stresses and pore pressures were measured at depths of 30 mm (i.e. prototype 1.5 m), 55 mm (2.75 m) and 80 mm (4 m) using total stress transducers T1, T2, T3 and pore pressure transducers P1, P2, P3 (Figure 4.1). These transducers were placed at radial distances of approximately 30 mm, that is 1.5m or 3$R_s$, from the pile axis. Figure 6.11 shows the computed total stress histories (labelled “FEM (1.5 m”) plotted against the corresponding experimental measurements, at the three depths of interest. Both the experimental and numerical
results were plotted to prototype time scale. Despite some discrepancies, the trends indicated by the numerical predictions were generally consistent with the experimental data. Note that the curves labelled “FEM (1.5 m)” in the graphs are obtained from elements located at a prototype radial distance of 1.5 m (30 mm in model scale or 3Rs) at each transducer level. Also superposed on the graphs are the calculated stress histories at radial distances of 1 m (20 mm in model scale or Rs) and 2 m (40 mm in model scale or 4Rs), denoted as “FEM (1 m)” and “FEM (2 m)” respectively. The results suggest that the calculated stresses were quite sensitive to small changes in radial distance. Experimentally, this implies that slight variations in the transducer location relative to the SCP location are likely to affect the measured stress response. As shown on Figure 6.11, the measured centrifuge stress histories were generally consistent with the computed trends based on transducer locations of between 20 and 30 mm (model distances) from the pile axis. Similar conclusions may be drawn from the pore pressure comparisons shown on Figure 6.12.

As shown on Figure 6.11, the measured histories at the three depths consistently registered an abrupt, rapid decrease in total stress at the start of the casing withdrawal phase. This feature was previously highlighted in Chapter 4, and was attributed to several likely reasons, among which are the instantaneous cavity contraction and the reversal of the casing-soil friction force. The cavity contraction associated with SCP installation is a complex mechanism as explained in Chapter 4. Such a phenomenon is a difficult and challenging feature to model using the finite element approach. Only a simplified and illustrative analysis was performed here to
briefly examine its influence. The sole difference between this analysis and the preceding analyses is that the casing diameter was slightly reduced at the end of jack-in, prior to its subsequent withdrawal. Accordingly, prescribed displacements were applied to the surrounding clay to close the gap generated by the smaller casing diameter, thus resulting in cavity contraction. Figure 6.13a illustrates the effect of such a cavity contraction, where a notable stress drop is registered upon the reversal in casing penetration. The tendency is consistent with the experimental observations (Figure 6.11).

Besides cavity contraction, the effect of frictional soil-casing interaction was examined as well. As the incorporation of high frictional effects between the soil and casing contact surfaces introduced significant convergence difficulty in the numerical simulation, only one analysis was carried out to illustrate the influence of friction. Figure 6.13b shows the computed stresses from the calculations using a Coulomb friction coefficient of 0.3 at the soil-casing interface. It is noticed that the incorporation of frictional effects also led to a drop in the total stress, albeit smaller, as the casing reversed its penetration direction.

In addition, there may be other causes which contribute to the abrupt stress drops, such as the contrasting rigidity between the transducer and the soil, and the interaction between the reversal direction of the meridional shear stress and the total stress transducer.
6.3.4 Comparison of ABAQUS results with previous experimental data by Juneja (2002)

A separate analysis was performed by modifying the present finite element model to simulate an earlier centrifuge experiment reported by Juneja (2002). As described in Chapter 4, the experimental procedure and apparatus adopted by Juneja were similar to those used in the present test series. Hence, the finite element model described in Section 6.2 could be readily modified to simulate Juneja’s test conditions. Juneja’s tests were carried out using Singapore marine clay, the behaviour of which was approximated in the present analysis using the Drucker-Prager soil model with the following soil parameters: (i) effective friction angle $\phi' = 21.5^\circ$, (ii) dilation angle $\psi = 0^\circ$, (iii) the modulus ratio, $G/p' = 25$, (iv) lateral stress coefficient $K_0 = 0.63$, (v) effective unit weight $\gamma' \approx 5.7\text{kN/m}^3$, (vi) permeability $k = 5 \times 10^{-10} \text{m/s}$, (vii) effective Poisson’s ratio $\nu' = 0.3$. These parameters were chosen in accordance with the reported geotechnical properties of Singapore marine clay (Tan, 1983; Chua, 1990; Juneja, 2002; Arulrajah & Bo, 2008).

Comparisons of the numerical and experimental results for Juneja’s tests are shown on Figures 6.14 and 6.15, with additional relevant information tabulated in Table 6.1. Figure 6.14 plots the measured total stress histories (“Ta” to “Tc”) at three depths during the SCP installation, together with the corresponding numerical predictions (“FEM”). As can be seen, the numerical results agree reasonably well with the experimental measurements, considering the complexity of the experiment and the numerical models, as well as the simplicity of the constitutive model used to model soil behaviour. Besides the overall trend, there is also good agreement
between the computed and measured residual stress at the three depths, at the end of pile installation. Figure 6.15 shows the corresponding pore pressure comparisons during SCP installation. Again, there is good agreement between the numerical predictions and experimental measurements. Compared to the earlier analysis of Section 6.3.3, there appears to be better agreement between the computed and measured responses for Juneja’s tests. As Figures 6.14 and 6.15 show, Juneja’s experimental data do not manifest an abrupt drop at the start of the casing withdrawal; this may account for the difference. It should be noted that, in Juneja’s tests sand was not injected through the casing tip during jack-in. As a result, there was no sand layer formed around the casing which could undergo densification resulting in cavity contraction. Furthermore, Juneja used a longer and wider backing plate underneath total stress transducer, which might have ameliorated some of the effect of shear stress reversal on the total stress transducer.

6.4 Post-installation stress and strength conditions in the soil

As mentioned earlier, the numerical analyses did not terminate with the completion of SCP installation. Instead, an additional phase was added to examine the long-term behaviour of the soil, during which the excess pore pressures generated during pile installation were allowed to dissipate. The resulting post-installation strain and stress fields are discussed in the following subsections, together with the associated strength improvements of the soil after consolidation.
6.4.1 Pore pressure and stress field following pile installation

For the soil domain considered in this study, the numerical analysis indicates that the excess pore pressures generated by SCP installation took about $4 \times 10^6$ seconds, or approximately 46 days, after pile installation to fully dissipate. Figures 6.16a - d show the pore pressure contours at different times after the pile installation, for the prototype equivalent of the kaolin clay model shown in Figure 6.1. That is to say, Figure 6.16a shows the pore stress distribution immediately after pile installation, Figure 6.16b - c shows the interim contours at two instances during post-installation consolidation and Figure 6.16d shows the long-term, steady-state pore pressure contours (i.e. hydrostatic pore pressure field).

By the end of excess pore pressure dissipation, significant build-up is observed in effective radial stress ($\sigma_r'$) field as shown in Figure 6.17, which depicts the long-term, steady-state stress distribution. The dashed lines in the graphs denote the initial in-situ stress state of the soil. As can be seen, the radial stresses in the pile vicinity were substantially increased. Similar trend is identified in the mean effective stress ($p'$) contour plots as shown on Figure 6.18. After the excess pore pressures had fully dissipated, the mean effective stresses were considerably higher than their initial values within a zone of $6 R_s$ from the pile. At a distance of $3 R_s$, the ultimate mean effective stresses were approximately 1.5 times their corresponding initial values.

The stress contours described above indicate that, upon dissipation of excess pore pressures, the long-term mean effective stresses in the pile vicinity were
significantly increased as a result of SCP installation. As to be discussed in the
following sections, the increases in mean effective stresses may be translated into soil
strength improvements resulting from SCP installation.

6.4.2 Strength improvement effect

As discussed in Chapter 5, the use of the Drucker-Prager soil model in the present
study allows the undrained strength $s_u$ to be inferred from the mean effective stress $p'$
via the following correlation

$$s_u = \frac{1}{2} M p'$$

(6.1)

where $M$ is the friction coefficient $= 6\sin \phi'(3-\sin \phi')$. Using this correlation, the
mean effective stress contours shown on Figure 6.18 may be transformed into the
long-term, post-installation undrained strength contours shown on Figure 6.19. Also
included on the figure are the initial in-situ undrained strength contours of the soil,
inferred using Equation 6.1, denoted with dashed lines. Substantial set-up of shear
strength was discernible within 6 pile radii from the center of the pile, which is
characterized with the same features as the preceding mean normal stress contours
(Figure 6.18).

As mentioned in Section 4.3.2, centrifuge T-bar data were used to evaluate the
undrained strength improvements with depth at a radial distance of $3R_s$ (30 mm).
The results were presented in Figure 4.13, for both the short term condition shortly
after pile installation, and the long term condition after significant pore pressure
dissipation. Figure 6.20 shows how these measured trends compare against the
computed strength improvements. Following the notation of Figure 4.13, P-O is the measured in-situ soil strength, while P-A and P-B represent the measured short-term (2-3 mins model time after SCP installation) and long-term strength profiles at the radial distance of 3R_s, respectively. Their counterparts from the present finite element analysis are plotted on the corresponding graphs, and labelled “P-O (FEM)”, “P-A (FEM)”, and “P-B (FEM)”, respectively. It can be seen that, except for the top 1.5 m of soil, the numerical predictions compare favourably with the experimental measurements. The discrepancies at shallow depths may be attributed to near-surface over-consolidation of the experimental soil sample, in which the kaolin clay deposit was pre-loaded under 1-g condition before undergoing 50-g consolidation in the centrifuge. The pre-loading pressure was about 6.5 kPa, which resulted in the top 1 m or so layer of soil becoming over-consolidated. Such over-consolidated soil behaviour was not captured by the Drucker-Prager model, which may result in the observed discrepancy between the numerical and experimental results at shallow depths. For the normally consolidated soil at greater depths, Figure 6.20 shows good agreement between the computed and measured data for the in-situ, the short-term and the long-term undrained strengths.

A comparison study is also performed for the strength improvement ratio, I_{su}, as shown on Figure 6.21. The strength improvement ratio I_{su} is herein defined as the ratio of the post-installation, post-consolidation undrained shear strength over the pre-installation undrained shear strength of the soil:

\[ I_{su} = \frac{s_u \text{ post}}{s_u \text{ pre}} \]  

(6.2)
in which $s_{u-i}$ is the post-installation, post-consolidation undrained shear strength and $s_{u-i}$ the pre-installation undrained shear strength of the soil. It therefore quantifies the magnitude of the long-term strength improvement due to the SCP installation. As Figure 6.21 shows, the numerical data predict strength improvement ratio of about 1.5 at a radial distance of $3R_s$, which is quite close to the experimental value of approximately 1.75. The experimental values were obtained by dividing the data of the preceding post-installation, post-consolidation strength profile (P-B) by the corresponding values of the pre-installation soil strength profile (P-O).

In conclusion, favorable agreement has been achieved between the experimental and numerical results not only in terms of absolute (short and long term) strength values, but also the strength improvement ratio. This affirms the validity of the present finite element analysis for assessing SCP-induced strength improvements, i.e. the set-up effect. More significantly, it paves the way towards a comprehensive evaluation and quantification of the strength set-up caused by SCP installation, which will be described in the following sections.

### 6.5 Strength improvement profile – Parametric studies

The previous discussion highlighted that soil’s post-installation, post-consolidation undrained shear strength field is substantially improved over its in-situ strengths due to installation of SCP. The strength enhancement is heavily dependent on the radial distance. The strength improvement profile in the radial direction is therefore of particular interest. The subsequent sections present a parameter study on the $I_{su-r/R_s}$
6.5.1 Changes in numerical modeling aspects

For the parametric studies, several changes were made to the finite element model since there is no longer a need to replicate the centrifuge model. Firstly, the thickness of soil domain was increased from 5m (Figure 6.1) to 14m (Figure 6.22), to allow changes with depth to be better reflected. To keep computational time within manageable levels, slightly coarser elements were used. The mesh shown in Figure 6.22 has 10752 nodes and 10481 elements. Besides, change was also made to the SCP installation rates such that it more represented the general SCP construction situations. According to one field construction record (Kitazume, 2005), the casing penetration and casing withdrawal/SCP formation rates were set at 50 mm/s and 10 mm/s respectively. Other than the above, the other aspects of numerical modelling were identical to those described in section 6.2.

6.5.2 Strength improvement profiles

With the modified finite element model, the numerical simulation was first carried out to model the SCP installation in kaolin clay with properties as described in Section 6.2.2. Figure 6.23 plots the computed post-installation, post-consolidation strength improvement ratio with radial distance at three representative depths namely 4m (8\(R_s\)), 7m (14\(R_s\)) and 10m (20\(R_s\)) below surface. As can be seen, data from all three depths roughly fall along the same line. Also superimposed in the graph is the normalized correlations.
radius of plastic zone or plastic radius \( (R_p/R_s) \) estimated from the cylindrical cavity expansion calculation (Vesic, 1972; Randolph & Wroth, 1979). As Figure 6.23 shows, the improved zone, where \( I_{su} > 1 \), is concentrated within the plastic radius. Furthermore, as Figure 6.24 shows, within the improved zone, the strength improvement ratio \( I_{su} \) changes with normalized radius \( r/R_s \) in logarithmic manner and can be fitted by a straight line of the form:

\[
I_{su} = A - B \ln(r/R_s) \quad (6.3)
\]

or,

\[
\Delta I_{su} = I_{su} - 1 = (A - 1) - B \ln(r/R_s) \quad (6.4)
\]

where \( A \) and \( B \) are fitted coefficients; \( \Delta I_{su} \) is strength gain. \( A \) represents the maximum strength improvement ratio (i.e. the one on the pile face) and \( B \) corresponds to the gradient of \( I_{su} \sim \ln(r/R_s) \) curve. In this case, \( A \) and \( B \) is around 2.0 and 0.51.

### 6.5.3 Effects of friction angle and modulus ratio

Cone penetration analyses discussed in Chapter 5 showed that the important parameters for the penetrometer problem are the effective friction angle \( \phi' \) and modulus ratio \( G/p' \). Since the sand pile installation process also involves similar penetration and cavity expansion processes, one would surmise that these two parameters would also significantly affect SCP. Figure 6.25 presents the calculated strength improvement profiles for soils for a constant modulus ratio \( (G/p'=20) \) but different friction angles \( (\phi'=18^\circ, 30^\circ \text{ and } 35^\circ) \). The \( I_{su} \sim \ln(r/R_s) \) curves on the graph were all plotted for three depths namely 4m, 7m and 10m. Together with Figure 6.24, these graphs show that, for a given friction angle, the strength improvement
profiles at different depths fall roughly along the same trend line, which can be approximated as a logarithmic function of the form given by Equation 6.3. As shown on Figure 6.26 for the middle depth of 7m, $I_{su}$ remains linearly correlated with $\ln(r/R_s)$ within the improved zone as $\phi'$ ranges from 18°-35°. Increasing the friction angle also increases the strength gain, with consequent increases in A and B. In all cases, the extent of the improved zone is well-predicted by the plastic radius.

Figure 6.27 shows the computed strength improvement radial profiles for soils with a constant friction angle ($\phi'$=23°) but different modulus ratios ($G/p'$= 40 and 80) for three different depths of 4m, 7m and 10m. Together with Figure 6.24, the graphs show that, for a given modulus ratio, the $I_{su} \sim \ln(r/R_s)$ correlation at different depths fall roughly along the same trend line. This trend line may be approximated as a logarithmic function (Equation 6.3) for modulus ratios of between 20 and 40. For $G/p'=80$, the deviation from linearity becomes more evident. Thus, for higher modulus ratios, the linear relationship for $I_{su}$ may not hold. As shown on Figure 6.28 for the middle depth of 7m, the increase of modulus ratio accentuates the strength improvement and apparently extends the improved zone; this is also reflected in the plastic radius. However, the gradient of $I_{su} \sim \ln(r/R_s)$ line within the improved zone, that is the parameter B, seems to be insensitive to the $G/p'$ change. Graphically, the $I_{su} \sim \ln(r/R_s)$ line is simply translated side-wise as $G/p'$ increases. Once again, the extent of the improved zone remains well-represented by the plastic radius, which also expands with increasing modulus ratio.

To fit the strength improvement radial profiles from different sets of friction
angle and modulus ratio, values of A and B in Equation 6.3 need to be adjusted accordingly. Figure 6.29 summarizes A values fitted for various friction angles from $\phi' = 18^\circ-35^\circ$ and modulus ratios $G/p' = 20-120$. The horizontal axis in the graph corresponds to the friction coefficient $M$ which is a dimensionless parameter of friction angle as described earlier. As Figure 6.29 suggests, A is influenced by both by both friction angle and modulus ratio. Increasing friction angle and modulus ratio both result in the greater A value. In view of the physical interpretation of A, it also implies that the maximum strength gain (i.e. the one on the pile face) can be accentuated by the increase of friction angle and/or modulus ratio. On the other hand, as Figure 6.30 shows, although B changes with the friction coefficient, it is nearly unaffected by the modulus ratio. It is consistent with previous observation that the gradient of $I_{su}\sim\ln(r/R_s)$ line remains almost unchanged when modulus ratio varies.

Figure 6.29 and 6.30 in combination provide the means to deduce the appropriate values of A and B for soft soils with friction angle and modulus ratio falling within the aforementioned typical ranges. The values of A and B can then be used with Equation 6.3 to assess the strength improvement radial profile ($I_{su}\sim r/R_s$ curve). The computed strength improvement profile may have important implications in the design practice of SCP. As reported by Kitazume (2005), sand compaction piles in the field are usually constructed with a high replacement ratio ranging from 0.3 to 0.8. This translates into a tributary zone for each pile that covers a plan area extending from about 1 $R_s$ (in which the piles are touching) to 2 $R_s$ from
the pile axis. For typical soft clay with friction angle of 25° and modulus ratio of 30, corresponding value of A and B is around 2.1 and 0.52, which is interpolated from Figure 6.29 and 6.30. Therefore, Equation 6.3 estimates that the soil within this tributary zone (1 - 2 Rs) is likely to experience strength enhancements between 74% to 110%, (i.e. $I_{su} = 2.1$ at 1Rs and 1.74 at 2Rs). For multiple SCPs, there is also an interaction effect from adjacent piles, which Lee et al. (2004) postulated may be estimated by superimposing the improvement effects from individual piles.

The possibility that the strength gain may be significant also implies that lower replacement ratios may be viable if the strength gain is accounted for in design. For instance, for a low replacement ratio of 10%, most soil falls within the tributary area stretching from 1Rs to 3Rs. For the same typical soil as discussed above, the strength gain is still above 50%, which is undoubtedly substantial. If it is accounted for in the design, the strength enhancement of soil incurred by SCP installation will be conducive to promoting SCP with lower replacement ratio.

6.6 Concluding remarks

In this chapter, the axisymmetric finite element model for simulating the installation of a sand compaction pile was developed, and the numerical results were presented and discussed for both the short-term and long-term soil responses. The numerical results were compared against those measured in the centrifuge experiments discussed in Chapter 4. Despite some discrepancies, there was generally reasonable agreement
between the numerical predictions and experimental observations. Possible reasons for the discrepancies were highlighted and discussed.

The updated post-installation, post-consolidation strength conditions in the soil were inferred from the computed effective stresses, assuming a Drucker-Prager constitutive model. The magnitude and extent of SCP-induced strength improvement were depicted by a computed strength improvement radial profile (i.e. \( I_{su} - r/R_s \) curve). It was found that the \( I_{su} - r/R_s \) curve for soil with various stiffness and strength properties can be well approximated by the logarithmic relationship (Equation 6.3). The fitted coefficients A and B in Equation 6.3 are governed by the soil’s friction angle and modulus ratio. Based on the numerical findings, a procedure was developed to estimate the SCP-induced strength enhancement at various radial distances for soils of different strengths and stiffness.

The present computational results, in combination with the experimental data of Chapter 4, provide useful insight into the fundamental mechanisms involved in SCP installation, and more importantly provide a simple and practical means for estimating the strength set-up in clay incurred by SCP installation. The latter, in particular, may promote the use of SCP with lower replacement ratios, by considering the strength set-up in clay.
Table 6.1 Summary of Juneja’s (2002) centrifuge experimental information from selected tests.

<table>
<thead>
<tr>
<th>Data &amp; Transducer identifier referred to in this chapter</th>
<th>Transducer type</th>
<th>Transducer position</th>
<th>Transducer &amp; Test identifier, as referred to in Juneja (2002)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Depth (m)</td>
<td>Radial distance (to pile axis) (m)</td>
</tr>
<tr>
<td>Ta</td>
<td>TST</td>
<td>2.8</td>
<td>1.4</td>
</tr>
<tr>
<td>Pa</td>
<td>PPT</td>
<td>5.6</td>
<td>1.98</td>
</tr>
<tr>
<td>Tb</td>
<td>TST</td>
<td>2.8</td>
<td>1.4</td>
</tr>
<tr>
<td>Pb</td>
<td>PPT</td>
<td>4.7</td>
<td>1.98</td>
</tr>
<tr>
<td>Tc</td>
<td>TST</td>
<td>4.2</td>
<td>1.05</td>
</tr>
<tr>
<td>Pc</td>
<td>PPT</td>
<td>5.6</td>
<td>1.48</td>
</tr>
</tbody>
</table>

*Test T2, T5 and T6 were all conducted at 70g. The casing jack-in and withdrawal rates (in model scale) are 2.14 mm/s and 0.5 mm/s, 1.85 mm/s and 0.71 mm/s, 1.9 mm/s and 0.54 mm/s at T2, T5, T6, respectively.
Figure 6.1  Schematic illustration of finite element model.
Figure 6.2 Modelling procedure: (a) casing starts to penetrate; (b) casing insertion ends; (c) casing withdraws with simultaneous sand injection; (d) the installation completes and post-installation consolidation of soil commences.
Figure 6.3  Mesh of the finite element model.

Figure 6.4  Deformed mesh resulting from the use of (a) conical and (b) flat ends.
Figure 6.5  Deformed mesh recorded when (a) casing penetrates to the 2.5m depth and (b) SCP is formed up to the 2.5m depth.
Figure 6.6  Computed shear strain ($\varepsilon_{rz}$) contours registered at the instant when (a) casing penetrates to the 2.5m depth and (b) SCP is formed up to the 2.5m depth.

Figure 6.7  Computed radial strain ($\varepsilon_r$) contours registered at the instant when (a) casing penetrates to the 2.5m depth and (b) SCP is formed up to the 2.5m depth.
Figure 6.8  Computed radial stress (\(\sigma'_r\)) contours captured at the instant when (a) casing penetrates to the 2.5m depth and (b) SCP is formed up to the 2.5m depth.

Figure 6.9  Computed total pore water pressure (u) contours captured at the instant when (a) casing penetrates to the 2.5m depth and (b) SCP is formed up to the 2.5m depth.
Figure 6.10  Radial distribution of (a) excess pore water pressure ($\Delta u$) and (b) normalized excess pore water pressure ($\Delta u/s_u$) at different depths, upon the completion of the SCP installation.
Figure 6.11  Comparison of measured and computed total stresses at depths of (a) 1.5 m; (b) 2.75 m; (c) 4 m.
Figure 6.12  Comparison of measured and computed pore pressure at depths of (a) 1.5 m; (b) 2.75 m; (c) 4 m.
Cavity contraction results in a drop in the total stress, upon the reversal of casing penetration.

Friction results in a drop in the total stress, upon the reversal of casing penetration.

Figure 6.13  Influence of (a) cavity contraction and (b) frictional soil-casing interaction on the calculated total stress.
(a) Comparison at the radial distance of 1.4m, depth of 2.8m

(b) Comparison at the radial distance of 1.4m, depth of 2.8m

(c) Comparison at the radial distance of 1.05m, depth of 4.2m

Figure 6.14  Comparison of measured (Juneja, 2002) and computed total stress.
Figure 6.15  Comparison of measured (Juneja, 2002) and computed pore pressure.
Figure 6.16  Computed total pore pressure (u) contours at instants of (a) 0s, (b) 5.2E4s, (c) 2.6E5s, and (d) 4.0E6s after the SCP installation.
Figure 6.17  Computed long-term, steady-state contours of radial stress ($\sigma_r'$).

Figure 6.18  Computed long-term, steady-state contours of mean normal stress ($p'$).
Figure 6.19 Computed post-installation, ultimate undrained shear strength ($s_u$) contours.

Figure 6.20 Comparison between experimental measurements and numerical predictions of strength profiles.
Figure 6.21 Comparison between experimental measurements and numerical predictions of strength improvement ratio $I_{su}$.
Figure 6.22  New and deeper finite element mesh for parametric study.
Figure 6.23  Strength improvement radial profile ($I_{su}$~$r/R_s$ curve).

Figure 6.24  Strength improvement radial profile ($I_{su}$~$\ln(r/R_s)$ curve).
Figure 6.25  Strength improvement radial profiles for soil with the same modulus ratio ($G/p' = 20$), but different friction angle: (a) $\phi' = 18^\circ$; (b) $\phi' = 30^\circ$; (c) $\phi' = 35^\circ$. 

\[ I_{\text{un}} = 1.83 - 0.41 \ln(r/R_s) \]

\[ I_{\text{un}} = 2.21 - 0.69 \ln(r/R_s) \]

\[ I_{\text{un}} = 2.3 - 0.75 \ln(r/R_s) \]
**Figure 6.26**  Strength improvement radial profiles for different friction angles.
Figure 6.27 Strength improvement radial profiles for soils with the same friction angle ($\phi' = 23^\circ$), but different modulus ratio: (a) $G/p' = 40$ and (b) $G/p' = 80$.

Figure 6.28 Strength improvement radial profiles for different modulus ratios.
Figure 6.29  Fitted coefficient A for soils with G/p' ranging 20-120 and M ranging 0.7-1.2.

Figure 6.30  Fitted coefficient B for soils with G/p' ranging 20-120 and M ranging 0.7-1.2.
Chapter 7: CONCLUSIONS

7.1 Summary of findings

The strength set-up or the set-up effect (Asaoka et al., 1994b) in soil, due to sand compaction pile installation, has been alluded to or highlighted in various studies including field investigations (Enokido et al., 1973; Aboshi et al., 1979; Yagyu et al., 1991; Asaoka et al., 1994b; Matsuda et al., 1997), experimental tests (Lee et al., 2001 & 2004; Juneja, 2002; Ng, 2003), numerical and theoretical analyses (Asaoka et al., 1994b; Lee et al., 2004; Guetif et al., 2007). However, in practice, the set-up of soil’s strength is usually not considered in the SCP design. This is likely due to the many unknowns regarding the strength set-up phenomenon, such as its magnitude, spatial extent, and influence factors, etc. As such, it is difficult to establish a procedure to account for strength set-up in the design. The present research seeks to obtain a comprehensive and in-depth understanding of the set-up effect, based on which a simplified procedure was proposed to consider strength set-up in design.

This research comprised both centrifuge experiments and numerical analysis. A total of five experiments were conducted in the experimental study. The experimental results illustrated the changes in soil response while the SCP was being installed nearby. More importantly, they also provided reliable strength measurements which helped to quantify the SCP-induced strength improvement. The influence of consolidation during SCP installation was also examined by comparing the results from two different sequence of pile installation.

The current numerical study dealt with two aspects of the modelling of deep penetration problems, namely (i) simulation of the cone penetrometer under various
rates of penetration, and (ii) simulation of sand compaction pile installation and the accompanying strength changes in the soil. The former was included herein to examine penetration rate effects and, more importantly, to validate the various numerical techniques proposed for modelling deep penetration problems with consolidation effects. By incorporating the coupled-consolidation and updated Lagrangian large-strain formulations, finite element analyses were carried out to study the cone response for a wide range of penetration rates. The numerical results compared favorably with analytical and other numerical solutions, as well as centrifuge experimental results. The use of the normalized back-bone curve allowed the results to be presented in a compact and meaningful way for studying the influence of penetration rate. The observed correlation between the back-bone curves and the soil properties led to the development of a practical method for deducing some key soil properties from penetration test results.

The same numerical techniques used for modeling the deep cone penetration were adopted for simulating the sand compaction pile installation. The numerical results provided useful information on the response of various variables such as strains, stresses and pore pressures. The post-installation, post-consolidation strength field could also be deduced based on the computed effective stresses. Reasonable agreement was obtained between the computed and measured stress and pore pressure histories, as well as the strength improvement effect. The numerical results were post-processed to illustrate the magnitude and extent of SCP-induced strength improvement. It was found that the logarithmic function may be used to
characterize the variation of strength improvement ratio with radial distance from the pile axis. These findings were used to propose a practical approach for considering the strength set-up effect in sand compaction pile design.

In summary, the findings from this research study may be expressed as follows:

i) The installation of sand compaction pile was noticed to exert significant influence on the surrounding clay, where considerable changes in the radial stress and pore pressure were registered. Generally, the peak values of stress and pore pressure increases recorded during casing jack-in and withdrawal could be reasonably predicted by cavity expansion theories. The results from the present study using kaolin clay are generally consistent with those reported by Juneja (2002) using Singapore marine clay.

ii) Substantial strength enhancements were observed in the soil after pile installation. The magnitude of the strength improvement was noted to be affected by consolidation effects, as well as the number of piles. By carrying out the tests for two different installation sequences, two distinct consolidation scenarios were produced which in turn led to different strength improvement behaviour. It was found that the timely dissipation of excess pore pressures between successive pile installations can enhance the set-up effect. Besides, increasing the number of piles can also result in additional strength improvement in the soil.

iii) The finite element analyses was able to reasonably replicate the response of the cone penetrometer for a wide range of penetration rates, associated with
the full spectrum of consolidation conditions. The calculated results form the
limiting drained and undrained results were noted to agree well with published
analytical solutions. Both the drained and undrained net cone resistance
increased with the increasing modulus ratio and friction angle. On the other
hand, the ratio of the drained to undrained net cone resistance appeared to be
quite insensitive to the friction angle, but increased with the modulus ratio.

iv) By plotting the computed cone resistances from different penetration rates in
the form of the normalized back-bone curve, the influence of cone penetration
rate was clearly illustrated. Favorable agreement was observed between the
computed back-bone curve and the centrifuge experimental results of
Randolph and Hope (2004). In addition, the characteristic back-bone curves
were found to be influenced by the local strength and stiffness properties of
the soil in the immediate vicinity of the cone tip. Based on the numerical
results, a procedure was developed from which important soil properties such
as the modulus ratio, the angle of friction and the coefficient of consolidation
can be deduced from the cone penetration test results. Its practical
application was illustrated through a field example.

v) The numerical simulation of sand compaction pile installation provided useful
information on how the stresses, strains and pore pressures change during and
after the pile installation. The strength fields could also be inferred from the
computed effective stresses. The calculated radial stress and pore pressure
histories compared well with the present experimental results and Juneja
(2002)’s centrifuge measurements. Furthermore, the deduced short-term and long-term strength profiles were also in favorable agreement with the experimental measurements.

vi) The strength improvement radial profile ($I_{su}$/$r/R_s$ curve) quantified the magnitude and extent of the SCP-induced strength improvement in the soil. It was found that significant strength improvement was approximately concentrated within the plastic zone defined by the cavity expansion theory. The $I_{su}$/$r/R_s$ curve may be approximated by the logarithmic function, whose fitting parameters were correlated to the soil’s stiffness and strength properties. Based on results from the numerical parametric studies, a simplified procedure was proposed to estimate the magnitude of SCP-induced strength enhancement. This provides a means for incorporating the set-up effect in the design of sand compaction piles,

7.2 Recommendations for future research

The following issues may be examined in future research work in this area:

i) There is a need to investigate the effect of increasing confinement on the strength gain in sand. As discussed in the earlier chapters, the installation process of sand compaction pile involves displacing the soft clay with sand, which would lead to a substantial stress and subsequent post-consolidation strength set-up in clay. The 'improved' clay in turn increases the lateral confinement on the sand pile itself. Such enhancement will result in
additional strength improvement in the sand column, thus contributing to the overall strength of the improved composite ground. This effect, which was not considered in the present study, should be examined in a future study so as to obtain a more complete picture of the overall strength improvement characteristics.

ii) Another pertinent issue is the stress concentration ratio, which helps to evaluate the relative strength contribution of sand column and clay in the composite ground. Thus far, the estimation of stress concentration ratio is largely empirical or based on field measurement data. There remains a lack of reliable approach to predict the stress concentration ratio for various replacement ratios. Research attempts in future may be oriented towards proposing a practical and proper approach for estimation of stress concentration ratio, which help to take full advantage of strength set-up in clay and confinement enhancement effects in sand.

iii) There is a need to extend the numerical modeling to three-dimensional analysis of pile group installation. While the current analysis successfully replicated the single pile installation using an axisymmetric idealization, the simulation of pile group installation was not addressed. The latter simulation is more challenging numerically, due to the cumulative soil deformation arising from multiple pile installation which causes convergence difficulty. Moreover, the required computational time and resources also spiral sharply due to the consideration of the third dimension.
REFERENCES


