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EFFECTS OF LATERAL SOIL MOVEMENTS ON PILES

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**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
NANYANG TECHNOLOGICAL UNIVERSITY**

2005

Effects of Lateral Soil Movements on Piles

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School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University
in fulfilment of the requirements for the degree
of Doctor of Philosophy of engineering

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SUMMARY

Piles are commonly subjected to induced loads from nearby construction activities such as approach embankments, excavation, tunnelling and moving slopes. In order to better understand the mechanism of this complicated soil-pile interaction problem, experimental and numerical studies have been carried out to investigate the behaviour of pile foundations embedded in soft clay when subjected to lateral soil movements.

In the experimental work, a series of model tests on single piles and pile groups subjected to a uniform soil movement profile were carried out to investigate the ultimate soil pressure acting on the piles. For single pile tests, the results showed that the pile shape had some effects on the ultimate soil pressure. The ultimate soil pressure for a square pile was 12% higher than that for a circular pile. For pile group tests, the ultimate soil pressure acting on an individual pile within a group was different from that of a single pile. The ultimate soil pressures for individual piles in a group were generally smaller than those of single piles. The ultimate soil pressures on the “front” row piles were always much larger than those on the “middle” row or “back” row piles. The differences in the ultimate soil pressures between the “middle” row piles and the “back” row piles were less significant, and so were the differences in the ultimate soil pressures between the “side” piles and the “centre” pile in the same row. The ultimate soil pressures acting on an individual pile within a group differed with different pile spacings, numbers of piles and arrangements of piles. There was an obvious trend that the pile group factor F_p , in terms of the ultimate soil pressures for the whole group, reduced with an increased number of piles, and increased with increasing pile spacing. For a pile group with the same pile number but different arrangements, a 10% difference still existed in the group factors.

Numerical studies were also carried out using three-dimensional finite element analyses. For the back analyses of the single pile model tests, the computed ultimate soil pressures were in good agreement with those measured from the model tests. Parametric studies of full-scale piles indicated that the behaviour of the piles was

significantly influenced by the pile flexibility, the magnitude of soil movement, the pile head boundary conditions, the shape of soil movement profile and the thickness of the moving soil mass. For a pile with restrained pinned-head and pinned-tip conditions, the maximum bending moment in the pile increased with increased pile flexibility, magnitude of soil movement and thickness of the moving soil mass. The provision of the head restraint increased the pile bending moments. The maximum pile bending moment for a rectangular soil movement profile was larger than that for a trapezoidal profile.

Based on the experimental and numerical results, a simplified method has been proposed to estimate the maximum pile bending moment. The method generally gave an upper-bound estimation of the maximum pile bending moments in the cases considered.

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LIST OF SYMBOLS

ϕ	friction angle of soil
ϕ'	effective friction angle of soil
γ, γ_s	unit weight of soil
γ_p	unit weight of pile
η_p	plastic viscosity of soil
η_F, η_Q	shape factor
μ	friction coefficient of the soil-pile interface
μ_f	pile flexibility reduction factor
μ_m	magnitude of the soil movement factor
μ_r	pile head boundary condition factor
μ_s	soil movement profile factor
μ_z	thickness of moving soil mass factor
ν_p	Poisson's ratio of pile
ν_s	Poisson's ratio of soil
θ	angle
ρ	pile deflection
ρ_{max}	maximum pile deflection
σ_p'	preconsolidation pressure
σ_v'	effective overburden pressure
τ	soil shear stress obtained from the pressuremeter curves
τ_l	horizontal shear stress at the centroid of the pile element
τ_y	yield stress of soil
ξ_i	elastic slip of the interface element
ψ	dilation angle of soil
ψ'	effective dilation angle of soil
c_u	undrained shear strength of soil
c_v	coefficient of vertical consolidation
c'	effective cohesion of soil
d	diameter (circular pile) or width (square pile) of a pile

e_0	initial void ratio
h	height
k	coefficient of permeability
l	length
l_i	characteristics contact surface length of the interface element
n	number of piles in a row or in a group
p	lateral soil pressure
p_L	limiting soil pressure
p_p	net pressuremeter pressure
p_u	ultimate soil pressure
p_{ui}	ultimate soil pressure for an individual pile in a group
p_{us}	ultimate soil pressure from the “standard” test
q_s	unit skin friction of the interface elements
t	wall thickness of a pipe pile
v_l	average flowing velocity of soil
w	water content
y	soil movement or displacement
y_u	ultimate soil displacement
y_{ui}	ultimate soil displacement for an individual pile in a group
y_{us}	ultimate soil displacement from the “standard” test
z	depth from the ground surface
z_s	thickness of the moving soil mass
A_i	area of a small zone of the pile element
C_c	compression index
C_r	recompression index
E_i	elastic modulus of the interface elements
E_p	Young’s modulus of pile
$E_p I_p$	bending stiffness of pile
E_s	elastic modulus of soil
F	side shear drag
F_p	group factor in terms of ultimate soil pressure for a whole group
F_{pi}	group factor in terms of ultimate soil pressure for an individual pile
F_{pri}	group factor in terms of ultimate soil pressure for a row piles

F_{yi}	group factor in terms of ultimate soil displacement for an individual pile
F_s	factor of safety
G_s	specific gravity
I_p	moment of inertia of pile
K	lateral resistance factor, $K = p_u/c_u$
K_0, K_{0x}, K_{0y}	coefficient of earth pressure at rest
K_R	pile flexibility factor
L	embedded length of pile
LL	liquid limit
M	pile bending moment
M_{max}	maximum pile bending moment
P	load or force
P_u	ultimate soil force per unit length, $P_u = p_u \times d$
PL	plastic limit
Q	frontal soil resistance
S_1, S_2, S_3	distance from the centre of pile to the boundary
S_f	shear force
S_h	centre-to-centre pile spacing for piles in a row
S_v	centre-to-centre pile spacing for piles in a line
T_R	tension resistance of the interface element

Chapter 1

Introduction

1.1 Background

The majority of piles are designed to support loads that are applied directly to the pile head by a structure. However in many cases piles are subjected to induced loads from nearby construction activities. Examples are: piles supporting bridge abutments adjacent to approach embankments, existing piles adjacent to pile driving, piles adjacent to excavation and tunnelling, and piles in moving slopes. Piles may also be specially designed to restrain soil movements when they are used to stabilise unstable slopes and potential landslides.

Figure 1.1 shows some typical cases for piles subjected to lateral forces resulting from soil movements. Before the soil starts to move, the soil surrounding the piles at any depth is in equilibrium under the initial stress state. Once the soil starts moving, the stress state in the soil around the piles will change from the initial state to a new equilibrium state. This results in the development of lateral forces on the pile shafts. These lateral forces will induce additional bending moments and deflections in piles, which in the worst instance may lead to the structural distress or failure of the piles.

A critical factor in the design of piles subjected to lateral loadings resulting from soil movement is the ultimate soil pressure p_u , which is defined herein as the maximum value of the limiting soil pressure along the length of the pile shaft when piles are subjected to large soil movements. Knowledge of the ultimate soil pressure could be used, for example, to carry out a “ p - y ” type analysis to study the pile behaviour (Poulos 1973; Byrne et al. 1984; Goh et al. 1997). Although considerable research work has been carried out using theoretical analysis (Broms 1964; Poulos and Davis 1980; Randolph and Houlsby 1984) as well as numerical analysis (Chen 1994; Bransby 1995; Pan 1998), there are still some uncertainties about the ultimate soil pressure for single piles.

(a) Embankment

(b) Excavation

(c) Unstable slope

Figure 1.1 Piles subjected to lateral forces resulting from soil movement

Moreover, piles are generally used in groups with close spacing (2.5~3.5 times pile diameter from centre to centre), and the number of piles and the pile arrangement may differ. In most situations, the ultimate soil pressure acting on an individual pile within a group is different from a single pile. To date, only limited research has been carried out to investigate the group effect on the ultimate lateral soil pressure.

At the same time, the behaviour of piles subjected to lateral soil movements is influenced by many factors, such as the pile flexibility, the pile fixity conditions, the magnitude of soil movement, the soil movement profile etc. Therefore, further numerical analysis is required to provide insights into the influence of some key factors on the pile response, not only on the soil pressure, but also on the lateral deflection, the shear force and the bending moment distributions along the pile shaft.

1.2 Objectives and Scope of the Study

Lateral soil movements due to excavation, tunnelling, unstable slopes and embankment construction can induce bending moments and deflections in the pile foundations as illustrated in Figure 1.1. Excessive soil movements may cause distress or failure of the piles. Therefore, the main aim of this study is to investigate the behaviour of single piles and pile groups in soft clay when subjected to lateral movements.

This research project involves extensive experimental model tests and three-dimensional finite element analyses. The work includes the following main aspects:

- 1) Conducting laboratory experiments on model single piles to investigate the ultimate soil pressure acting on the pile, and the effects of pile shape on the ultimate soil pressure.
- 2) Conducting laboratory experiments on model pile groups to investigate the effects of a number of parameters, including the pile spacing, the number of piles and the arrangement of piles in a group, on the ultimate soil pressures acting on individual piles within a group.

- 3) Using a three-dimensional finite element program to back-analyse the model test results and examine the influence of some input parameters.
- 4) Conducting three-dimensional finite element analysis to further study the behaviour of full-scale single piles and pile groups with various pile flexibility, pile head boundary conditions, magnitudes of soil movement, soil movement profiles and thickness of moving soil mass.
- 5) Conducting three-dimensional finite element analysis of case studies from the literature to further validate the 3-D program as well as to examine the soil-pile interaction behaviour from the field tests.
- 6) Proposing a simplified method to estimate the maximum bending moment in piles when subjected to lateral soil movements.

1.3 Outline of Thesis

The outline of this thesis is described below:

- 1) *Chapter 1* gives a brief introduction of the existing problems and outlines the main objectives of this research work.
- 2) *Chapter 2* presents a review of the literature relevant to this research topic. The review covers empirical and theoretical studies as well as numerical studies on laterally loaded single piles and pile groups. Laboratory tests on laterally loaded piles are also reviewed.
- 3) *Chapter 3* describes the set-up of the experimental apparatus, and the testing procedures and programs for single piles and pile groups.
- 4) *Chapter 4* presents the experimental results of preparatory tests and single pile model tests.
- 5) *Chapter 5* presents the experimental results of pile group tests and interpretation of the results.
- 6) *Chapter 6* presents the 3-D back analyses of the single pile model tests.
- 7) *Chapter 7* presents the 3-D analyses of full-scale single piles and pile groups.
- 8) *Chapter 8* describes the 3-D analyses of two cases from the literature.
- 9) *Chapter 9* proposes a simplified method to estimate the maximum bending moment in piles.

- 10) *Chapter 10* concludes the major findings from this study and makes some recommendations for future work.

Chapter 2

Literature Review

2.1 Introduction

Considerable research work has been carried out on piles subjected to lateral loads. In this chapter, a brief review of empirical and theoretical studies as well as numerical studies is presented. This is followed by a summary of previous laboratory studies on laterally loaded piles.

2.2 Empirical and Theoretical Studies

Laterally loaded piles are commonly divided into two categories. One is the so-called “active” pile in which the pile is subjected to external loads at the pile head by building loads, earthquakes, waves, etc. The other is the “passive” pile in which the pile is subjected to lateral soil pressure as a result of lateral soil movements from landslides, slope failure, excavation, tunnelling, etc. The behaviour of both active piles and passive piles, i.e. bending moments, shear forces and deflections, is governed by the soil pressures acting on the pile shaft. Knowledge of the ultimate soil pressure could be used, for example, to carry out a “ p - y ” type analysis to study the pile behaviour (Poulos 1973; Byrne et al. 1984; Goh et al. 1997). Although some empirical or theoretical methods described below (Broms 1964; Randolph and Houlsby 1984; Reese 1984) were first developed for active piles, the value of the ultimate soil pressure p_u derived by these authors is commonly adopted for passive piles as well (e.g. Poulos 1973; Goh et al. 1997). Some available methods to estimate the lateral forces/pressures acting on piles are summarised below.

Broms Method

Broms (1964a, b) presented methods for the calculation of the ultimate lateral resistance and lateral deflections at working loads of single piles driven into cohesive soil as well as cohesionless soil. As mentioned in the previous paragraph, the ultimate soil pressure from Broms’ method that was originally developed for

active piles is commonly used for passive pile analysis. For short piles in cohesive soils, the soil pressure was assumed to be zero at the ground surface to a depth of $1.5d$ (d is the diameter or width of pile). Below this depth, a constant value of between $8.28c_u$ and $12.56c_u$ for various pile sections with smooth or rough surface was obtained based on the theory of plasticity, in which c_u is the undrained shear strength of the soil. To simplify the calculations, the ultimate soil pressure p_u below $1.5d$ was recommended as follows:

$$p_u = 9c_u \quad (2-1)$$

Similarly, for short piles in a cohesionless soil, the ultimate soil pressure p_u along the pile at any depth was assumed as follows:

$$p_u = 3K_p \sigma'_v \quad (2-2)$$

$$K_p = \frac{1 + \sin \phi'}{1 - \sin \phi'} = \tan^2 \left(\phi' + \frac{\pi}{4} \right) \quad (2-3)$$

in which ϕ' is the effective friction angle of the soil and σ'_v is the effective overburden pressure.

De Beer and Wallays Method

De Beer and Wallays (1972) presented an empirical design method for embankment piles as shown in Figure 2.1. When the factor of safety F_s of the whole soil mass was larger than 1.6, the soil around the piles was assumed to be in a state far from the rupture state and a uniform pressure was assumed to act on the piles over the full depth of the soft stratum. When the F_s was less than 1.6, the full ultimate soil pressure $10.5c_u$ was assumed to act on the pile in opposing directions above and below the point where the slip circle intersected the pile. This method can only predict the maximum bending moment and does not allow prediction of the distribution of the bending moments along the pile shaft. Furthermore, a conventional stability analysis to determine F_s is required in order to use this method.

(a) $F_s > 1.6$ (b) $F_s < 1.6$ **Figure 2.1 De Beer and Wallays method (after Stewart 1992)*****Poulos Method***

Poulos (1973) developed a boundary element method for analysing a single pile in a soil subjected to lateral soil movements. The pile was assumed to be a thin vertical strip with width, length and constant flexibility, and was divided into finite elements. The soil was assumed to be elastic, but lateral yield of the soil at the soil-pile interface was also taken into account. The method required an input of the magnitude of the free-field soil movement at each depth and the ultimate soil

pressure acting on the pile. By imposing displacement compatibility between the pile and the adjacent soil, the behaviour of a pile (bending moments, shear forces and deflections) was determined. Parametric studies indicated that the more critical factors that affected the pile behaviour were the pile flexibility, the pile head boundary conditions, the soil movement profile and the magnitude of soil movement. A number of comparisons between instrumented field data and those predicted by the method were also presented to reveal fair agreement. However, a good prediction from this method depends on an accurate estimation of the magnitude of soil movement and the ultimate soil pressure. Furthermore, although this method can be modified to accommodate pile groups, a major problem with such an analysis is to determine how the group effects influence the value of the ultimate soil pressure.

Ito and Matsui Method

Ito and Matsui (1975) developed two theoretical methods to estimate the lateral forces on stabilising piles in a row due to the surrounding soil undergoing plastic deformation, as shown in Figure 2.2. In their analysis, it was assumed that two types of plastic states occurred in the ground around the rigid piles, one of which was a plastic state satisfying the Mohr-Coulomb's yield criterion and the other was a plastic state in which the soil was considered as a visco-plastic solid. The former was called the theory of plastic deformation and the latter was called the theory of plastic flow.

In the theory of plastic deformation in a comparatively hard soil layer, as for the cohesive soil, the ultimate soil force P_u acting on a pile per unit thickness of layer in the direction of the x-axis was obtained as follows:

$$P_u = c_u \left\{ D_1 \left(3 \log \frac{D_1}{D_2} + \frac{D_1 - D_2}{D_2} \tan \frac{\pi}{8} \right) - 2(D_1 - D_2) \right\} + \gamma z (D_1 - D_2) \quad (2-4)$$

in which c_u is the shear strength of the soil, γ is the unit weight of the soil and z is the depth from the ground surface.

Figure 2.2 Ito and Matsui method (Ito and Matsui 1975)

Using the theory of plastic flow in a soft soil layer, the ultimate soil force P_u was obtained as follows:

$$P_u = \sqrt{2m\tau_y} \left\{ \sqrt{1 + \frac{m}{2\tau_y D_2^2}} - \sqrt{1 + \frac{m}{2\tau_y D_1^2}} + \log \frac{D_1 \left(1 + \sqrt{1 + \frac{m}{2\tau_y D_1^2}} \right)}{D_2 \left(1 + \sqrt{1 + \frac{m}{2\tau_y D_2^2}} \right)} \right\} +$$

$$(D_1 - D_2) \left\{ \frac{(\sqrt{2} - 1)\pi^2 m}{8D_2^2} + (\sqrt{2} - 1) \sqrt{\left(\frac{\pi^2 m}{8D_2^2} \right)^2 + \frac{\pi^2 m \tau_y}{4D_2^2}} + \frac{m}{D_1 D_2} + \sqrt{2}\tau_y - 2c + \gamma z \right\} \quad (2-5)$$

in which,

$$m = 16\eta_p v_1 D_1 / \pi^2,$$

τ_y = the yield stress of the soil,

η_p = the plastic viscosity of the soil, and

v_1 = the average velocity of the soil flowing uniformly through arc surface BB' .

However, the difficulty in obtaining the input parameters, such as η_p and v_1 , limits the applicability of this method.

Viggiani Method

Viggiani (1981) presented a limit equilibrium method to analyse the interaction mechanism between the sliding cohesive soil and the piles. As shown in Figure 2.3, three failure models (A, B and C) were proposed for non-yielding piles, also known as rigid piles, and another three possible failure modes (B1, BY and B2) were proposed for piles whose yield moments were smaller than the bending moments acting on them.

Figure 2.3 Failure modes (Viggiani 1981)

The occurrence of a particular failure mode was controlled by the geometry of the problem (the length and diameter of the pile, the thickness of the sliding soil mass), the yield moment of the pile section and the undrained shear strength of the stable and sliding soil. The ultimate soil pressure was found to range from $3.1c_u$ to $4.3c_u$.

within the sliding soil and had a single value of $7.2c_u$ in the stable soil. Hence he recommended that the most likely values of the ultimate pressure should be $4c_u$ within the sliding soil and $8c_u$ in the stable soil. These low values could be due to the fact that the piles considered by Viggiani (1981) failed through the formation of a plastic hinge, and hence the ultimate lateral resistance was not reached. Apparently, a proper choice of the failure mode for a particular problem is required for a reasonable prediction, and the influence of group effects is not considered.

Briaud et al Method

Briaud et al. (1984) and Smith (1987) proposed that the actual ultimate soil force P_u acting on a pile consisted of the frontal soil resistance Q and side shear drag F resulting from friction, as shown in Figure 2.4.

Figure 2.4 Actual theoretical stress distribution (Smith 1987)

Based on pressuremeter test results, Briaud et al. (1984) put forward the following empirical equations to calculate the ultimate soil force considering the shape factor of piles.

$$P_u = Q + F \quad (2-6)$$

$$Q = p_p d \eta_Q \quad (2-7)$$

$$F = \tau d \eta_F \quad (2-8)$$

in which,

p_p = net pressuremeter pressure,

d = pile diameter (circular pile) or wide (square pile),

η_Q (shape factor) = 1.0 for square piles and 0.75 for circular piles,

η_F (shape factor) = 2 for square piles and 1 for circular piles, and

τ = soil shear stress obtained from the pressuremeter curves by the subtangent method.

Randolph and Houlsby Method

Randolph and Houlsby (1984) proposed exact solutions for the ultimate lateral pressure acting on a circular pile in cohesive soil using classical plastic theory. Two approaches were used to estimate the collapse loads. In the first approach, i.e. the lower bound approach, a stress distribution was assumed in the soil and the stress was in equilibrium with a given applied load. In the other approach, a failure mechanism was postulated and the collapse load was estimated by equating the rate of dissipation of energy within the deforming soil mass to the work done by the external load. It was found that the ultimate lateral pressure p_u ranged from $9.14c_u$ for a completely smooth circular pile up to $11.94c_u$ for a completely rough circular pile. The following expression was recommended by Randolph and Houlsby (1984):

$$p_u = 10.5c_u \quad (2-9)$$

Springman Method

Based on the results of some centrifuge model tests, Springman (1989) developed a relatively simple design method for embankment piles, as shown in Figure 2.5. A parabolic profile of lateral soil pressure distribution acting on the piles was assumed. The magnitude of the soil pressure was obtained from a complex expression taking into account the average differential soil-pile movement.

Figure 2.5 Springman method (after Stewart 1992)

Reese Method

Another active pile relationship that is commonly used was proposed by Reese (1984). The ultimate soil pressure for clays (p - y type analysis) was suggested as follows:

$$p_u = \left(3 + \frac{\gamma z}{c_u} + \frac{0.5z}{d} \right) c_u \text{ or } 9c_u, \text{ whichever is smaller} \quad (2-10)$$

in which z is the depth from the ground surface.

2.3 Numerical Studies

Compared with empirical and theoretical studies, numerical methods seem to be quite promising because they can handle more complicated problems, such as piles embedded in a multi-layer soil subjected to various profiles of lateral soil movements. Both two-dimensional (2-D) and three-dimensional (3-D) numerical methods are summarised below. It should be pointed out that with all numerical approaches, the selection of the appropriate parameters is the key issue.

2.3.1 Two-Dimensional Analysis

Lane and Griffiths (1988) presented numerical solutions for the ultimate pressure to cause failure of a laterally loaded circular pile in frictional soil. The analysis was performed using a ‘non-axisymmetric’ approach involving Fourier expansions in the tangential directions and this allowed a rather simple transformation from rough to smooth conditions at the soil-pile interface. The pile was displaced incrementally into the elasto-plastic soil. The tension at the soil-pile interface was allowed and no gap between the soil and the pile was developed. Good agreement was obtained between the numerical results and the analytical solution presented by Randolph and Houlsby (1984).

Maugeri and Motta (1992) carried out a series of numerical analyses to investigate the effects of lateral loading on piles induced by sliding slopes. The analyses were based on the concept of the modulus of subgrade reaction in which a non-linear hyperbolic relationship between the pile load and the relative soil-pile displacement was adopted. It was found that the ultimate soil pressure varied between $3c_u$ and $3.5c_u$ for the flowing soil, and $5.6c_u$ and $8c_u$ for the firm clay below the sliding surface.

Stewart (1992) assumed the ultimate soil pressure of $10.5c_u$ in plane-strain finite element analysis to investigate the response of pile groups influenced by lateral soil movement from a nearby surface loading. The piles were represented by equivalent sheet pile walls and the soft clay was modelled as elastic-plastic obeying the Tresca failure criterion as well as the Modified Cam Clay failure criterion. The analysis, which allowed for lateral yield at the soil-pile interface, compared reasonably well with the centrifuge data and generally gave acceptable predictions of the pile bending moment distribution. The results of the analysis suggested that an elastic-plastic model for the soft clay stratum could be used.

Chen (1994) presented a numerical model using a developed finite element program (AVPULL) with a plane-strain approach to analyse the behaviour of piles undergoing lateral soil movements. The piles were modelled as rigid and square in

shape, while the soil was assumed to be purely cohesive undrained clay obeying the Tresca failure criterion. A Goodman-type interface element (Goodman et al. 1968) was used to model slip between the soil and the pile. Figure 2.6 shows the coarse mesh used to model an infinitely long row of piles. Half of a pile and rollers were used to take advantage of the symmetrical boundary conditions. S_1 is the distance from the centre of the pile to the lower outer boundary (AA) from which a uniform soil displacement was applied, S_2 is the distance from the centre of the pile to the upper outer stress-free boundary (BB), and S_h is the centre-to-centre spacing between two piles in a line perpendicular to the direction of soil movement.

Figure 2.6 Finite element mesh for an infinitely long row of piles (Chen 1994)

At the lower outer boundary AA, a uniform soil displacement y was applied incrementally to the first row of nodal points, while the upper outer boundary BB was set to be stress-free. It was found that when S_1 was larger than $7.5d$ and S_2 was larger than $4.5d$, results of the analysis were not affected by the boundary. It was also found that when S_h was larger than $4.5d$, the piles behaved like single isolated piles and did not affect each other. The ultimate soil pressure for a single isolated pile was found to be $11.4c_u$, as shown in Figure 2.7.

Figure 2.7 Normalised “ p - y ” relationship for a single isolated pile subjected to lateral soil movement (Chen 1994)

For piles in two infinitely long rows, as shown in Figure 2.8, the values of p_u for different spacings are listed in Table 2.1. S_v is the centre-to-centre spacing between piles in the direction of soil movement.

Figure 2.8 Piles in two infinitely long rows (Chen 1994)

Table 2.1 K values for piles in two infinitely long rows (Chen 1994)

Case	S_h/d	S_v/d	$K = p_u / c_u$	
			“Leading” pile	“Trailing” pile
I-1	3	3	6.7	10.1
I-2	3	6	10.9	12.0
I-3	6	3	6.2	6.8

Bransby (1995) also conducted a series of similar finite element analyses through the program CRISP94 (Britto and Gunn 1987; 1990) on a single pile as well as on pile rows. The piles translating through elastic-plastic soil were modelled as rigid circular adherent discs. A very fine mesh of 760 cubic strain triangles and 400 increments were used. Interface elements were not used to model slip between the soil and the pile. It was found that the ultimate soil pressure for a single isolated pile was $11.75c_u$. For two infinitely long rows of piles with different spacings, it was found that the leading and trailing pile p - y curves were almost identical with a maximum difference between rows of 0.8%, which was different from the results given in Table 2.1 by Chen (1994).

Goh et al. (1997) presented a simplified numerical procedure (BCPILE) based on the finite element method for analysing the response of single piles subjected to lateral soil movements. The complex phenomenon of the soil-pile interaction was modelled by hyperbolic soil springs. The value of p_u for clays was taken to be $9c_u$. Comparisons were made between the observed behaviour of full-scale tests and centrifuge model tests and those computed by the proposed analytical method. Based on parametric studies, empirical design solutions for pile foundation systems at the base of a sloped embankment were also proposed. Using the same numerical procedure, Goh et al. (2003) carried out an actual full-scale instrumented study to examine the behaviour of an existing pile from nearby deep excavation activities. The theoretical predictions were found to be in reasonable agreement with the measured results.

2.3.2 Three-Dimensional Analysis

Soil-pile interaction is essentially a three-dimensional (3-D) problem, and a thorough study requires a 3-D analysis that is computationally expensive to perform. As described below, some researchers have used 3-D analysis to obtain excellent results and simulate laboratory tests or field tests.

Kooijman and Vermeer (1988) carried out a quasi three-dimensional elastic-plastic analysis of laterally loaded piles by neglecting vertical displacements, and simulating the pile translation into the soil. Because a very fine grid and the soil-pile interface elements were used in the analysis, good numerical predictions for the ultimate loads were obtained. The numerical values deviated less than 2.5% from the theoretical solutions by Randolph and Houlsby (1984).

Trochanis (1988) examined the influence of the non-linear behaviour of soil on the axial and lateral response of piles due to monotonic and cyclic loading by three-dimensional finite element analysis through the commercial program ABAQUS (Hibbitt, Karlsson and Sorensen, Inc.). Both single pile and coupled piles models were studied. Several factors including relative slippage or separation between a pile and the soil, and soil plasticity away from the pile were investigated.

Springman (1989) conducted a full three-dimensional finite element analysis on a row of piles using a linear elastic model. Although the mesh was very coarse (81 linear strain blocks), the results were in close agreement with those obtained from centrifuge model tests (see description in Section 2.4.2).

Brown and Shie (1990a, b) developed several 3-D models for the analysis of a single row of piles and two rows of piles subjected to lateral loads using the commercial program ABAQUS (Hibbitt, Karlsson and Sorensen, Inc.). Constitutive models for soil included a simple elastic-plastic model with Von Mises yield surface and associated flow and an extended Drucker-Prager model with nonassociated flow. Frictional interface elements were used to model slippage at the soil-pile interface. It was found that the effects of pile spacing within a single row of piles

(or the front row of a group) were relatively small for piles spaced at 3 diameters on centre or more in undrained clay. For two rows of piles, it was also found that the trailing row of piles was subjected to a significant reduction in maximum soil resistance.

Chaoui et al. (1994) presented a 3-D study on the pile group effects in unstable slopes using the finite program CESAR-LCPC. The 3-D geometry and boundary conditions for a single row of piles with centre-to-centre spacing, e , are shown in Figure 2.9.

Figure 2.9 3-D geometry and boundary conditions (Chaoui et al. 1994)

Because of symmetry, only half of the problem was considered. Interface elements were used in the analysis. The Drucker-Prager model was used for the soil, with unit weight $\gamma_s = 19 \text{ kN/m}^3$, Young's modulus $E_s = 18 \text{ MPa}$, Poisson ratio $\nu_s = 0.33$, cohesion strength $c' = 5 \text{ kPa}$, effective friction and dilation angle $\phi' = \psi' = 18^\circ$, and coefficient of earth pressure at rest $K_{ox} = K_{oy} = 0.5$. The concrete pile was assumed to be linear elastic with $\gamma_p = 25 \text{ kN/m}^3$, $E_p = 2.23 \times 10^4 \text{ MPa}$ and $\nu_p = 0.25$. The interface elements were modelled with $E_i = 18 \text{ MPa}$, tension resistance $T_R = 1 \text{ MPa}$ and the unit skin friction $q_s = 20 \text{ kPa}$. By observing the accumulated plastic zone

with imposed displacement, Chaoui et al. (1994) found no group effects occurred for a pile spacing higher than $3.5d$.

Bransby and Springman (1996) investigated the three-dimensional behaviour of two infinitely long rows of piles adjacent to a surcharge. No interface elements were used in this analysis. The mesh comprised 810 elements and had 13922 degrees of freedom. The bending moment and pile pressure profiles from the analysis compared well with those measured in the centrifuge model tests.

Chen (2001) conducted a series of non-linear finite difference analyses on the effects of an embankment slope on the response of lateral loaded piles using the commercial computer programs FLAC and FLAC^{3D} (Itasca 1997; 1998). Both single piles and pile groups subjected to passive loading were studied to understand the mechanics of soil-pile interaction. For single piles with different shapes, square piles were found to exhibit stiffer behaviour and provide higher soil resistance for p - y curves than circular piles. For a single row of piles on a 30° inclined slope, it was found that the group effects for one row of piles on the ultimate soil resistance is small when the pile spacing was greater than about $4d$.

Xu and Poulos (2001) employed a general 3-D coupled boundary element approach to analyse the response of piles subjected to passive loadings. A number of theoretical expressions for soil movements were developed and presented. Pile responses when subjected to some typical passive loadings, such as soil shrink/swelling, soil surface surcharge, tunnelling, soil movements arising from driving piles and cavity formation in soil, were examined.

2.4 Laboratory Model Tests

The complex soil-pile interaction behaviour has also been studied experimentally by conducting laboratory model tests on laterally loaded piles. Although laboratory tests have the advantages of low cost and easy operation compared with field tests, an effective laboratory test requires exact and logical simulation of the prototype. Conventional 1-g tests and centrifuge tests (refer to Springman 1989; Stewart 1992)

have been conducted by many researchers in the laboratory to investigate the behaviour of laterally loaded piles in sand or clay. It is generally recognised that the results of the conventional 1-g model tests in saturated clay are more reliable because the behaviour of the clay is not influenced by the normal (e.g. confining or surcharge) stresses; however, the results of the conventional 1-g tests in sand may not be as reliable because of scale and normal stress effects.

Laterally loaded piles are commonly divided into two categories. One is the “active” pile in which the pile is subjected to external loads at the pile head. The other is the “passive” pile in which the pile is subjected to lateral soil pressure as a result of lateral soil movements. This section summarises the experimental studies carried out on “active” and “passive” piles, with particular focus on pile group behaviour.

2.4.1 Laboratory Tests on Active Piles

Cox et al. (1984) presented the studies of a number of laboratory model tests to investigate the efficiency of pile groups in very soft clay under lateral loading. The tests were run in very soft clay with moisture content of 59% and shear strength of 2 kPa. The steel piles were 25.4 mm diameter open-ended tubes, with penetrations of two, four, six, or eight diameters. The transducers were used to measure the load in individual piles. The lateral loading was applied to the piles in one direction at a slow rate (0.97 mm/min) displacing through the soil. Tests were made on single piles and on three-pile and five-pile groups with clear spacings of 0.5, 1, 2, 3, and 5 diameters in-row and in-line arrangements. Efficiencies derived from lateral load were given for individual piles in a pile group as well as for the entire group.

Meyerhof and Sastry (1985), and Sastry and Meyerhof (1990) carried out a series of laboratory model tests on single rigid piles and flexible piles in clay under inclined load, respectively. The rigid circular piles were 1100 mm long with an outer diameter of 74 mm and a wall thickness of 7 mm, and the flexible piles were 1250 mm long with an outer diameter of 73 mm and a wall thickness of 7.4 mm. The clay with an average c_u of 15 kPa was packed into a steel drum of about 0.6 m diameter

and 1.4 m height or 1.65 m diameter and 1.05 m height in the tests of rigid piles or flexible piles, respectively. The piles were jacked into the clay and the load test was then immediately carried out under undrained conditions. Reasonable agreement was found between the results of these tests and predicted behaviour of the piles.

Shibata et al. (1989) carried out a number of laboratory model tests on free-headed pile groups in sand undergoing lateral loading. The schematic of the apparatus is shown in Figure 2.10. To make the ground as homogeneous as possible, sand was compacted previously in the cylindrical chamber, and then prepared by boiling technique. A pulley was attached to the outer wall to allow for lateral loading. Two types of model circular pile instrumented with strain gauges were used in the tests. One was made of aluminium with an outer diameter of 20 mm, a wall thickness of 1.6 mm and a length of 800 mm. The other was made of chloridized-vinyl with an outer diameter of 22 mm, a wall thickness of 2.2 mm and a length of 800 mm. The effect of pile spacing, number of piles, pile arrangement and rigidity of piles, were investigated. Data on group efficiency were presented for comparison with theoretical predictions based on the method developed by Randolph (1981). It was found that there was a maximum discrepancy of about $\pm 30\%$ between the measured and predicted values of group efficiency.

Figure 2.10 Lateral loading apparatus (Shibata et al. 1989)

Abendroth and Greimann (1990) conducted a number of model tests on piles with different boundary conditions. The steel square tubular pile was 1500 mm long with a width of 25 mm and a wall thickness of 0.813 mm. The behaviour of pinned-head friction piles in loose sand, fixed-head friction piles in loose or compacted sand and fixed-head bearing piles in loose sand were examined respectively. The characteristic pile and soil parameters, i.e. skin friction, lateral displacement and resistance, settlement and end-bearing capacity were derived from the model tests and then used in finite element analysis to idealise the nonlinear soil behaviour by Winkler springs.

Adachi et al. (1994) conducted two kinds of centrifuge model tests to investigate the ultimate lateral resistance of single piles as well as the interaction between two piles in a group. The aluminium piles embedded in dense sand were 400 mm long with an outer diameter of 15 mm and a wall thickness of 1 mm. The lateral displacement was applied at the pile head and the foot of the pile was fixed. It was found that the load distribution ratio of the front pile was larger than that of the rear pile. For the back piles, the closer the spacing between the piles, the smaller was the reduced ratio of reactive force. The study indicated that the evaluation of the subgrade reaction of the front piles could be treated as a single pile.

Mayne et al. (1995) conducted a number of laboratory model tests on free-head rigid drilled piles in anisotropically overconsolidated kaolin clay under static and cyclic lateral and moment loading. Circular drilled concrete piles with diameters d of 51 mm, 89 mm and 175 mm and different lengths L ($L/d = 3\sim 8$), were used to evaluate the effects of pile diameter and geometry. The prestress level for clay was 18~60 kPa. It was found that the load-displacement behaviour under monotonic static lateral loading was highly nonlinear, but it could be represented adequately by a hyperbola that included only two parameters: initial stiffness and ultimate capacity. The ultimate lateral capacity increased with increased shaft depth, diameter and pile surface roughness.

McVay et al. (1995) conducted a series of centrifuge tests on single piles and 3×3 -pile groups in loose to dense sands at three-diameter ($3d$) and five-diameter ($5d$)

spacings. The tests were modelled in the centrifuge at 45g. The model circular pipe piles were 9.6 mm in outer diameter and 296 mm long. In all tests, free-headed piles were driven and laterally loaded in flight without stopping the centrifuge. Results of the tests showed that the ratio of lateral resistance of a group to a single pile, i.e. efficiency, was independent of soil density. The group efficiency at $3d$ spacing was 0.74, whereas at $5d$ spacing the group efficiency was 0.94.

Rao et al. (1998) conducted out a number of experimental model tests on pile groups in marine clay undergoing lateral loading. The tests were conducted in a rectangular test tank of 1200 mm length by 800 mm width and 1100 mm height. Marine clay with a water content of 36% and an average shear strength of 5.3 kPa was placed in the test tank in 50 mm thick layers by hand packing. Model piles made of aluminium pipes were 25.4 mm or 12.7 mm in outer diameter, and those made of mild steel were 21.5 mm in outer diameter. Different pile embedment to diameter ratios (L/d) of 20, 30 and 40 were used in the tests. A pulley attached to the loading frame applied the lateral load on the pile head. It was found that the behaviour of pile-supported structures under lateral loads depended mainly upon the critical spacing of the piles, which was a function of both the pile embedment depth and the arrangement of piles with respect to the direction of loading.

Dyson and Randolph (2001) conducted a series of centrifuge model tests in calcareous sand to determine the behaviour of single piles subjected to lateral loading. The model aluminium piles used for the majority of the tests were 13 mm in diameter and 340 mm long. The piles instrumented with 13 levels of strain gauge were preinstalled or jacked in a strongbox with internal dimensions of 650 mm long by 390 mm wide by 325 mm deep. A number of factors were explored, including method of installation, rate of loading and pile head restraint. Based on the results, a load-transfer (p - y) model was proposed with modification factors taking into account of the effects of installation and loading rate.

Llyas et al. (2004) conducted a series of centrifuge model tests on capped-head pile groups in normally consolidated (NC) and overconsolidated (OC) kaolin clay. The centrifuge tests were conducted at 70g. The behaviour of 2, 2×2, 2×3, 3×3 and 4×4

piles in a group with a spacing of $3d$ or $5d$ was examined. The model piles were made of hollow square aluminium tubes with an outer width of 12 mm and an embedded length of 210mm. The piles were connected by a solid aluminium pile cap placed just about the ground level, and the pile cap was displaced at a rate of 0.05 mm/sec. The undrained shear strength of the NC clay varied linearly with depth from 0 to 20 kPa, and the undrained shear strength of the OC clay varied 10 to 25 kPa. It was found that the average lateral load per pile decreased with increasing number of piles in the group. The reduction in pile group efficiency was less severe for piles in OC clay than that of piles in NC clay. The shadowing effect of leading piles over trailing piles was also observed, and the shadowing effect was most significant for the leading row piles and less significant on subsequent rows of trailing piles.

Table 2.2 summarises the main details of laboratory tests on “active” piles described in this section.

Table 2.2 Laboratory tests on “active” piles

Reference	Pile			Tank Dimensions	Soil type
	Material	d (mm)	L (mm)		
Cox et al. (1984)	Steel	25.4	Various ^a	640 × 640 × 580 mm	Remoulded kaolin
Meyerhof and Sastry (1985)	Steel	74	1100	Sand: Φ 1 m, 1.6 m high; Clay: Φ 0.6 m, 1.4 m high	Packed clay, sand
Shibata et al. (1989)	Aluminium Chloridized -vinyl	20	800	Φ 1.65 m, 1.05 m high	Sand
		22	800		
Abendroth and Greimann (1990)	Steel	25	1500	Not available	Sand
Sastry and Meyerhof (1990)	PVC	73	1250	Φ 1.65 m, 1.05 m high	Packed clay, sand
Adachi et al.* (1994)	Aluminium	15	400	520 × 240 × 430 mm	Sand
Mayne et al. (1995)	Concrete	51, 89, 175	Various ^b	Not available	Overconsoli- dated kaolin
McVay et al.* (1995)	Steel	9.6	296	Not available	Sand
Rao et al. (1998)	Aluminium	12.7, 25.4	Various ^c	1.2 × 0.8 × 1.1 m	Packed Marine clay
	Steel	21.5			
Dyson and Randolph (2001)*	Aluminium	13	340	650 × 390 × 325 mm	Sand
Llyas et al. (2004)*	Aluminium	12	17.5	Φ 550 mm, 375 mm high	Kaolin clay

^a $L/d = 2, 4, 6, 8$; ^b $L/d = 3, 4, 6, 8$; ^c $L/d = 20, 30, 40$.

* For centrifuge tests.

2.4.2 Laboratory Tests on Passive Piles

Matsui et al. (1982) carried out a series of model tests on piles in a row due to lateral soil movement to check the validity of the theoretical equations (Equation 2-4 and 2-5) presented by Ito and Matsui (1975). The commercial clay (c_u from 9.8 to 22.4 kPa) was used in the tests as well as Toyoura sand. The schematic view of the test equipment is shown in Figure 2.11. The internal dimensions of the container box are 60 cm long by 30 cm wide by 30 cm deep. Circular piles with diameters of 20 mm, 30 mm and 40 mm and a length of 300 mm were used in the tests. Piles in a row were set at the centre of the container box and the pile diameter as well as the pile spacing could be changed. Based on the experimental results, it was found that the relation between the lateral force acting on a pile and the soil displacement could be represented by a bi-linear curve with an inflection point. It was also found that the ultimate pressure could be approximately estimated as 1.6 times the theoretical lateral pressure based on the theory of Ito and Matsui (1975).

Figure 2.11 Schematic view of the test equipment (Matsui et al. 1982)

Adachi et al. (1989) conducted a series of two-dimensional model tests in sand to investigate the preventive mechanism of the piles used for stabilising landslides. The model piles were either circular or square in shape and had diameters or widths of 20, 30 and 50 mm. It was found that the loads acting on the circular piles were generally smaller than those acting on the rectangular piles by about 15%. The results also indicated that parallel piles could stabilise sliding by an arching effect in front of the piles.

Springman (1989) investigated the performance of a row of free-headed piles and a pile group experimentally in the centrifuge for different pile and foundation geometries. The general arrangement of centrifuge models for the pile group is shown in Figure 2.12. The aluminium circular piles instrumented with strain gauges were 12.7 mm in outer diameter and 300 mm long. Pile response, in terms of bending moment, deflection and lateral pressure, was determined for surcharge loads applied to the centrifuge model.

A series of similar centrifuge tests were also conducted by Stewart (1992) on two rows of piles with pile cap. Square piles made of brass were 3.18 mm in width and 244 mm long. Pile response was monitored in terms of bending moment distribution and pile cap deflection. Both the undrained response and long term consolidation behaviour were observed. Various pile group configurations, embankment geometries and soil stratigraphies were also examined.

(a) Plan View

(b) Elevation view

**Figure 2.12 Arrangement of centrifuge models for pile group
(Springman 1989)**

Chen (1994) conducted a series of laboratory model tests on single piles as well as pile groups in calcareous sand undergoing lateral movement. The experimental steel vessel is shown in Figure 2.13. The aluminium circular piles with outer diameters of 25, 35 and 50 mm, wall thickness of 1.2, 1.5 and 2 mm, and a length of 1000 mm were used in the tests. The piles instrumented with 10 strain gauges were jacked into sand through an installation guide. A triangular profile of lateral soil movement was applied by rotating the upper parts of the steel plates, which were hinged at mid-height. The maximum bending moment induced in a single pile was found to be dependent on pile embedded length, pile diameter and the pile head fixity conditions. For a pile in a group, the group effect derived from the maximum

bending moment was dependent on some factors, including pile arrangement, pile spacing, pile head fixity condition and position of the pile.

(a) Plan View

(b) Elevation view

Figure 2.13 Experimental steel vessel (Chen 1994)

Bransby (1995) conducted four centrifuge tests at a nominal radial acceleration of 100g to investigate the behaviour of a group of 2 rows of 3 piles when an adjacent vertical surcharge was applied to the soil. Circular piles made of duralumin were 12.7 mm in outer diameter and 300 mm long. The pile group was pushed into the soil using a guide apparatus until the pile cap base was touching the top clay surface. Pile pressures were deduced from bending moment data for the piles in both front and back rows. The results revealed that lateral pile group displacements caused by an adjacent surcharge were greater than the recommended serviceability limits.

Pan (1998) carried out a series of laboratory model tests on single piles and coupled piles in soft clay subjected to a uniform soil movement profile. The rectangular steel piles were 20 mm wide, 295 mm long and 6 mm and 2 mm thick for the rigid and flexible piles, respectively. Five pressure transducers were mounted on the pile surface. The clay sample (300 wide by 300 mm long by 200 mm high) had an average undrained shear strength of 18 kPa. Figure 2.14 shows the apparatus set-up for the single ‘passive’ pile test. A rectangular profile of soil movement was imposed on the clay sample by controlling a hydraulic jack manually at a constant rate of 1.5 mm/min. The results showed that the ultimate soil pressure for a single ‘passive’ pile was independent of pile fixity conditions and pile flexibility. It was found that the ultimate soil pressure p_u ranged from $10c_u$ to $10.6c_u$ for a single passive pile. For two-pile groups, the values of p_u for different spacings are listed in Table 2.3. The values of the ultimate soil pressure acting on a pile in a group were smaller than that of a single pile.

Table 2.3 K values for two-pile groups in laboratory tests (Pan 1998)

Case		S_H/d	S_V/d	$K = p_u/c_u$			
Two piles in a line	I-1	—	3	“Leading”	7.8~8.2	“Trailing”	4.3~7.6
	I-2	—	5	pile	7.1~8.7	pile	8.1~8.9
Two piles in a row	I-3	3	—	6.7~7.1			
	I-4	5	—	8.7~8.9

(a) Plan view

(b) Elevation view

Figure 2.14 Apparatus for single passive pile test (Pan 1998)

Ong (2000) conducted a number of laboratory model tests on single piles and coupled piles in dry Ottawa sand under different sand densities, soil movement profiles and surcharges. The model test rig was able to induce either rectangular or triangular soil movement profile. The model circular steel piles were 30 mm in outer diameter and 500 mm long. The piles, instrumented with five miniature pressure transducers, were fixed rigidly at the bottom and cantilevered into the sand mass. The lateral soil pressure was found to be dependent on the surcharge load as well as the pile spacing in the couple-pile tests.

Leung et al. (2003) conducted a series of centrifuge model tests on free-head and capped-head pile groups consisting of 2, 4, 6 piles located adjacent to an unstrutted deep excavation in dry Toyoura sand. The model piles were fabricated from hollow square aluminium tubes with an outer width of 9.53 mm and an inner width of 6.35 mm. All centrifuge tests were conducted at 50g. The pile length embedded in sand was 250 mm, and the final model excavation depth was 90 mm with a total wall embedment of 160 mm. It was found that when two free-head or capped-head piles with the centre-to-centre spacing of $3.2d$ were arranged in a row parallel to the retaining wall at a distance of $5d$, the interaction effect between piles was insignificant. When two piles were arranged in a line perpendicular to the wall, the existence of a front pile reduced the detrimental effect of excavation-induced soil movement on the rear pile. For free-head 4-pile or 6-pile groups, the induced bending moment decreased as the number of piles increased.

Table 2.4 summarises the main details of laboratory tests on “passive” piles described in this section.

Table 2.4 Laboratory tests on “passive” piles

Reference	Pile			Tank Dimensions	Soil type
	Material	d (mm)	L (mm)		
Matsui et al. (1982)	Not available	20, 30, 40	300	$600 \times 300 \times 300$ mm	Overconsolidated kaolin, sand
Adachi et al. (1989)	Not available	20, 30, 50	250	Not available	Sand
Springman (1989)*	Aluminium	12.7	300	$675 \times 200 \times 170$ mm	Sand and clay, clay
Stewart (1992)*	Brass	3.18	244	$650 \times 390 \times 325$ mm	Clay and sand
Chen (1994)	Aluminium	25,35,50	1000	$565 \times 450 \times 700$ mm	Sand
Bransby (1995)*	Duralumin	12.7	300	$675 \times 200 \times 170$ mm	Sand and clay
Pan (1998)	Steel	20×2 ; 20×6	295	$300 \times 300 \times 200$ mm	Overconsolidated kaolin
Ong (2000)	Steel	30	500	$750 \times 500 \times 1100$ mm	Sand
Leung et al. (2003)*	Aluminium	9.53	250	$540 \times 200 \times 470$ mm	Sand

* For centrifuge tests.

2.5 Summary

2.5.1 p_u for Single Piles in Cohesive Soil

The behaviour of both active piles and passive piles, i.e. bending moments, shear forces and deflections, is governed by the soil pressures acting along the pile shaft. Tables 2.5, 2.6 and 2.7 summarise the values of the ultimate soil pressure p_u for single piles in cohesive soil obtained by empirical and theoretical methods, numerical methods and laboratory tests, respectively. As can be seen from the tables, considerable uncertainty of the magnitude of p_u in relation to the soil displacement, soil properties and soil-pile interface characteristics is evident. Another source of uncertainty is the measured c_u value, since the c_u is not a fundamental material property. The c_u is affected by the mode of testing, boundary conditions, rate of loading, confining stress level and other variables. The value of the ultimate soil displacement y_u (listed in Table 2.8), which is defined as the soil movement to mobilise the ultimate soil pressure p_u , is as uncertain as p_u . But the results also suggest that a y_u value of $1.0d$ is sufficient to fully develop the ultimate soil pressure in soft clay.

It should be pointed out that the majority of the laboratory tests mentioned above focused on the pile deflections and the pile bending moments. The ultimate soil pressure was derived from the pile bending moments measured by strain gauges based on assumptions of the fixity conditions at the pile head and pile tip. However, the actual fixed conditions at the pile head and pile tip for the laboratory tests may differ from the assumptions (Shibata et al. 1989). For example, a pile that is assumed to be fixed at the pile head may behave as a pinned head pile in the actual test, and vice versa. Therefore, there is some doubt as to the accuracy of the ultimate soil pressure derived from the pile bending moments. Pan (1998) and Ong (2000) measured the ultimate soil pressure directly using the pressure transducers. However, the values they obtained were still questionable because the soil pressure distribution along the pile width was not uniform (Briaud et al. 1984) and the side shear force between the soil and the pile could not be measured by the pressure transducers and thus was neglected.

Table 2.5 K values for single piles in cohesive soil in empirical and theoretical studies

Reference	$K = p_u/c_u$		Remarks
	Calculated	Suggested	
Broms (1964a)	8.28-12.56	9	—
Matlock (1970)	—	9	—
Bea (1971)	7-12	—	—
De Beer and Wallays (1972)	—	10.5	—
Ito and Matsui (1975)	3.33	—	—
Poulos and Davis (1980)	8.28~11.14	—	Square shaped piles
Viggiani (1981)	3.1-4.3	4	Within sliding surface
	7.2	8	Below sliding surface
Randolph and Houlsby (1984)	9.14-11.94	10.5	—
Reese (1984)	—	Smaller of 9 or $3 + \gamma z/c_u + 0.5z/d$	—

Table 2.6 K values for single piles in cohesive soil in numerical studies

Reference	$K = p_u/c_u$		Remarks
	Calculated	Assumed	
Lane and Griffiths (1988)	9.14~11.94	—	—
Springman (1989)	—	10.5	—
Maugeri and Motta (1992)	3-3.5	—	Within sliding surface
	5.6-8	—	Below sliding surface
Stewart (1992)	—	10.5	—
Chen (1994)	11.4	—	Square shaped piles
Bransby (1995)	11.75	—	—
Goh et al. (1997)	—	9	—

Table 2.7 K values for single piles in cohesive soil in laboratory tests

Reference	$K = p_u/c_u$	Remarks
Schapery and Dunlop (1978)	10-20	Passive pile
Meyerhof and Sastry (1985)	6.3	Rigid active pile
Sastry and Meyerhof (1990)	2.8	Flexible active pile
Pan (1998)	2.7-8.4	Active pile
	10-10.6	Passive pile

Table 2.8 y_u values obtained in the laboratory tests for passive piles

Reference	y_u	Remark
Matsui et al. (1982)	0.5~1.0d	in clay
	0.2~0.25d	in sand
Cox et al. (1984)	0.3d	in clay
Adachi et al. (1989)	0.1~0.16d	in sand
Pan (1998)	0.5~0.7d	in clay

In addition, either circular or square piles are used in most pile foundations. Table 2.9 summarises some of the K values corresponding to single piles with different shapes for comparison. It can be seen that there are some uncertainties about whether the shape factor should be considered in the design of piles subjected to lateral loading.

Therefore, further studies on single piles subjected to lateral soil movement using experimental and numerical methods are desirable.

Table 2.9 K values for single piles with different shapes in cohesive soil

Reference	$K = p_u/c_u$		Remark
	Circular pile	Square pile	
Broms (1964a)	9	9	Suggested value
Poulos and Davis (1980)	—	8.28~11.14	—
Briaud et al. (1984)	7.25	10	Herein assumed: $^*p_p = 9c_u, \tau = 0.5c_u$
Randolph and Houlsby (1984)	9.14~11.94	—	—
Chen (1994)	—	11.4	—
Bransby (1995)	11.75	—	—

* p_p and τ are referred to the definition in *Briaud et al Method* in Section 2.2.

2.5.2 p_u for Pile Groups in Cohesive Soil

Generally, piles are often used in groups in practice, and the number of piles in a group usually can differ. In most situations, the ultimate soil pressure acting on an individual pile within a group is different from a single pile, resulting in a reduction of pile group capacity. The reduction depends mainly on the pile spacing, the number of piles and the arrangement of piles in a group. Although some model tests on active piles (Cox et al. 1984; Adachi et al. 1994; Llyas et al. 2004) and passive piles (Chen and Poulos 1997; Pan et al. 2002) and some field tests on active piles (Meimon et al. 1986; Brown et al. 1987, 1988; Rollins et al. 1998; Ng et al. 2001) have been carried out, it appears that the issue of group effects remains unresolved. The group reduction factors of these researchers are presented in Chapter 5, alongside the model test results from this study. Therefore, more laboratory model tests and numerical studies are needed to examine the behaviour of individual piles within a group subjected to lateral soil movements.

Chapter 3

Experimental Apparatus and Test Set-up

3.1 Introduction

As mentioned in Chapter 2, laboratory tests have been conducted by a number of researchers on laterally loaded piles. The majority of these tests focused on the pile deflections and the pile bending moments. Although the ultimate soil pressure could be derived from the measured bending moments with assumed pile fixity conditions, the accuracy was questionable. Furthermore, some uncertainties remain, such as the shape effects and the group effects on the ultimate soil pressure acting on the piles. Hence, a specially designed apparatus has been fabricated and a series of laboratory tests have been carried out on instrumented model piles in cohesive soil undergoing lateral movement to measure the ultimate soil pressure. The testing program involved two main parts, namely single pile tests and pile group tests.

In practice, piles may be rigid or flexible and subjected to different profiles of lateral soil movements, such as triangular, rectangular or trapezoidal, as shown in Figure 3.1(a). For simplification, the present apparatus has been designed to apply a uniform rectangular profile of lateral soil movement on the rigid piles with pinned head and tip conditions to obtain the ultimate soil pressure. The aim is to simulate a unit cell at zone A as shown in Figure 3.1. The unit cell is assumed to be below the ground surface at a depth of more than $1.5d$ (d is the pile diameter) or just above any potential slip circle with a much smaller height h than the length L of the pile. According to the theoretical analysis proposed by Broms (1964a) and De Beer and Wallay (1972), the soil flows uniformly around the pile and exerts a constant value of ultimate soil pressure on the pile so long as the relative displacement between soil and pile is large. The application of such a soil movement profile was also used and justified by some previous researchers (Matsui et al. 1982; Cox et al. 1984; Pan et al. 2000), and so the results from the present experimental work are expected to be of some practical use.

Figure 3.1 Principles for experimental tests set-up

The main objectives of this experimental study were: (a) to use laboratory experiments to investigate the effects of pile shape on the ultimate soil pressure; and (b) to use laboratory experiments to investigate the pile group effects on the ultimate soil pressure, with attention on a number of parameters, including the pile spacing, the number of piles and the arrangement of piles in a group.

As stated in Section 2.4, conventional 1-g tests and centrifuge tests have been conducted by many researchers in the laboratory to investigate the behaviour of laterally loaded piles in sand or clay. It is generally recognised that the results of the conventional 1-g model tests in saturated clay are reliable because the behaviour of the clay is not influenced by the normal (e.g. confining or surcharge) stresses. Hence, only conventional 1-g model tests are considered in this study.

In this chapter, details of the experimental apparatus, sample preparation, testing procedure and testing program are described. The experimental results of preparatory tests and single pile tests are presented in Chapter 4, and the results of pile group tests are presented in Chapter 5.

3.2 Experimental Apparatus

A simple schematic diagram illustrating the overall testing arrangement is shown in Figure 3.2. It consisted of two parts, i.e. sample preparation and pile installation and loading, and was illustrated in Section 3.3. The main apparatus used in the present laboratory model tests are summarised in Table 3.1 and described in detail below.

Figure 3.2 Schematic diagram of model test arrangement

Chapter 3. Experimental Apparatus and Test Set-up

Table 3.1 Main apparatus

Apparatus	Function	Dimensions	Capacity	Remarks
Mixer	mix kaolin clay	—	0.05 m ³	AGI-50
Consolidometer	consolidate kaolin slurry	400 mm diameter, 800 mm in height	—	loading by air pressure
Sampling box	trim and contain the soil sample	300×300×150 mm with additional cutting edge	—	steel
Vessel	as main testing frame	570×322×165 mm (internal)	—	operated by motor gear system
Button load cell	measure lateral forces on piles	20 mm diameter, 9 mm in height	1 kN	TML
Load cell	measure load applied by motor gear loading system	—	50 kN	TML
LVDT	measure displacement	—	100 mm, 50 mm	TML
Test gauge	control air pressure during consolidation	—	400 kPa	accuracy: ± 0.25 %
Model piles	—	16, 25 mm dia. circular piles and 25 mm width square pile, 308 mm long	—	steel
Tubes	remove the soil and create a vertical hole before pile installation	13, 22 mm outer dia. circular tubes and a 22 mm outer width square tube, 1 mm thick wall,	—	steel
LDPE sheets	reduce friction	various	—	plastic
Data logger	record data from load cells and LVDTs	—	—	TDS-302

Mixer

A mechanical mixer (AGI-50) of 0.05 m³ capacity was used to mix the kaolin with tap water into a slurry. It took two and a half hours to obtain the slurry for each test.

Consolidometer

A consolidometer was used to make the kaolin sample, as shown in Photo 3.1. It comprised a cylinder, a base and a top-cover with a piston. The cylinder of 400 mm internal diameter and 800 mm height was made of stainless steel.

Photo 3.1 Consolidometer

Before pouring the kaolin slurry into the consolidometer, the smooth inner surface of the cylinder was greased so as to minimise the friction between the soil and the cylinder wall during consolidation. There were three large holes near the centre of the base for draining water from the consolidometer. At the centre of the top-cover there was a hole through which a piston could be moved up and down during consolidation. The piston rod was a hollow tube through which water draining from

the slurry could be removed. On the top cover there were two valves to exert and release the compressed air, respectively. These three parts of the consolidometer, i.e., the cylinder, the base and the top-cover with a piston, were bolted together and sealed by 'O' rings so as to prevent leakage of water and compressed air during consolidation.

Sampling box

The sampling box made of stainless steel was used to trim a block soil sample and was then used to retain the soil during all model tests. It comprises two parts, i.e., the main box and the cutting edge, as shown in Photo 3.2.

Photo 3.2 Sampling box

The main box was 300 mm long, 300 mm wide and 150 mm high (internal dimensions) with a wall thickness of 10 mm. There were two handles welded on the walls of the main box. The sharp cutting edge of 20 mm height was bolted to the main box before the block soil sample was trimmed. The internal walls of the

sampling box were greased so as to minimise wall friction during trimming (Davie and Sutherland 1978). After the soil sample was trimmed, the cutting edge was removed.

Vessel

The vessel was designed as the main testing frame. It comprised five parts, i.e. a tank, two sets of cover and base plates, a fixing system, a guiding system, and a motor gear loading system.

The tank was made of stainless steel plates of 16 mm thickness, and was 570 mm long, 322 mm wide and 165 mm high (internal dimensions) supported on four angle plates, as shown in Photo 3.3. There were a rectangular hole in the base for accommodating a base plate and a small circular hole in the wall for accommodating the pushing piston of the motor gear loading system.

Photo 3.3 Tank

Chapter 3. Experimental Apparatus and Test Set-up

Two sets of cover and base plates made of stainless steel were used for the tests, as shown in Figure 3.3. Set A was used either for a circular pile with a diameter of 25 mm or a square pile with a width of 25 mm. There was a square hole with a width of 29 mm in the cover plate and base plate, respectively. The width of the square hole was 4 mm larger than the diameter or width of the pile so that there was no contact between the plates and the pile during loading.

Figure 3.3 Two sets of cover and base plates

Chapter 3. Experimental Apparatus and Test Set-up

Set B was used for a circular pile as well as pile groups with diameter(s) of 16 mm. There were 25 circular holes with diameters of 20 mm in the cover plate and base plate, respectively. The distance between the centrelines of the circular holes was 1.5 times the pile diameter, i.e., $1.5d = 24$ mm. The diameter of the circular holes was 4 mm larger than the diameter of the circular pile to ensure that there was no contact between the plates and the pile during loading.

The base plate was placed in the hole in the base of the tank, and then bolted to the tank. After the sample box was placed exactly in the tank, the cover plate was placed on the top of the sample box and bolted to the tank.

The fixing system comprised four angle bars and six rectangular strips, as shown in Photo 3.4. The four angle bars with slots were mounted on the sidewalls of the tank for fixing the rectangular strips, which were used as a reaction frame in the tests. The rectangular strips were 19 mm wide, 30 mm high and 400 mm long. In each rectangular strip there were five circular recesses with a diameter of 20.5 mm and a depth of 1 mm. This would later enable a button load cell to be attached to the recess by two-sided adhesive tape.

The guiding system comprised a set of guiding frame, the tube guiding strips and the pile guiding strips, as shown in Photo 3.5. The guiding frame was mounted on the sidewalls of the tank for fixing the guiding strips. There were some screw holes in the horizontal bar of the guiding frame to keep the guiding strips in the exact position so that the piles could be installed precisely in the soil. The pile guiding strips comprised two symmetric pieces so that they could be bolted together during pile installation and removed after installation. The guiding holes in the guiding strips were 50 mm high with the same diameter or width as piles or tubes so that the piles or tubes could be installed vertically.

Photo 3.4 Fixing system of vessel

Photo 3.5 Guiding system of vessel

Chapter 3. Experimental Apparatus and Test Set-up

The motor gear loading system comprised a control panel, a motor, a gearbox and a pushing piston, as shown in Photo 3.6. A TML CDP-50 LVDT (Linear variable differential transformer) was mounted to measure the displacement of the pushing piston and monitor the displacement rate applied by the motor gear loading system. The plot of a preparatory test in Figure 3.4 shows that the motor gear loading system can apply a constant displacement rate of 1.05 mm/min on the sampling box. The rate was close to that of the model tests by Cox et al. (1984) and Pan et al. (2000), in which the rates of 0.97 mm/min and 1.5 mm/min were used, respectively, to ensure undrained conditions for cases of lateral loaded piles embedded in a soft clay.

Photo 3.6 Motor gear loading system

Figure 3.4 Displacement rate applied by the motor gear loading system

Button load cell

Button load cells (TML CLS-1kNA) of 1 kN capacity, 20 mm diameter and 9 mm height were used to measure the lateral force(s) acting on the pile(s), as shown in Figure 3.5 and Photo 3.7. The measured load was applied only on the compact head of the button load cell, which was a tiny cylinder of 2.5 mm diameter and 1 mm height. Two button load cells were needed to measure the lateral force acting on a pile and a total of eighteen cells were needed in a 9-pile group test.

Figure 3.5 Button load cell

Photo 3.7 Button load cell

Load cell

A TML KCM-50kNA load cell of 50 kN capacity was used to measure the load applied by the motor gear loading system.

LVDT (Linear variable differential transformer)

A TML CDP-100 LVDT of 100 mm capacity was used during consolidation to monitor the settlement of the kaolin clay by measuring the vertical displacement of the piston of the consolidometer.

A TML CDP-50 LVDT of 50 mm capacity was mounted on the gearbox of the motor gear loading system to measure the lateral soil displacement and monitor the displacement rate.

Test gauge

A test gauge with an accuracy of $\pm 0.25\%$ was used to control the air pressure during consolidation of the kaolin slurry, as shown in Photo 3.8.

Photo 3.8 Test gauge

Model Piles

Three types of model piles of the same length but different cross-sectional shapes were used in tests, as shown in Figure 3.6 and Photo 3.9. The steel piles were 308 mm long with a small cap of 3 mm thickness and two circular recesses in the shaft. The pile cap was designed to keep each pile at the same level during installation. The circular recesses with a diameter of 2.8 mm and a depth of 0.5 mm were designed to accommodate the compact head of a button load cell. The embedded pile length was 150 mm and the Young's modulus of the piles was 2.1×10^8 kPa. Pile-1 was a circular tubular pile with an outer diameter of 25 mm and an inner diameter of 18 mm. Pile-2 was a square tubular pile with a width of 25 mm and an inner diameter of 18 mm. Pile-3 was a circular pile with a diameter of 16 mm.

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Figure 3.6 Model piles

Photo 3.9 Model piles

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There are two common methods to evaluate the flexibility of a pile. One was presented by Poulos (1971) using the flexibility factor K_R , which was defined as:

$$K_R = \frac{E_p I_p}{E_s L^4} \quad (3-1)$$

in which E_p is the Young's modulus of the pile, I_p is the moment of inertia of the pile, E_s is the elastic modulus of the soil and L is the embedded pile length. A pile with a K_R value larger than 0.01 generally denotes a relatively rigid pile.

The other common method to assess pile flexibility is the ratio of the maximum deflection to the length of the pile, i.e. ρ_{max}/L . The value of maximum deflection ρ_{max} could be obtained by using the differential equation of deflection curve of a beam (Gere and Timoshenko 1991).

For Pile-3, the value of K_R was 0.278 according to Equation (3-1) in which $L = 150$ mm, $E_p = 2.1 \times 10^8$ kPa and $E_s = 200c_u$ (c_u was assumed as 24 kPa); the value of ρ_{max} was 1.106×10^{-1} mm and ρ_{max}/L was 1/1898 while the ultimate soil pressure p_u exerting on the pile was assumed as $10.5 c_u$ (after Randolph and Houlsby 1984). These results indicated that Pile-3 was rigid enough to meet the assumption of this experimental work mentioned in Section 3.1. Pile-1 and Pile-2 were also rigid because their outer diameters or widths were larger than that of Pile-3.

Other than the pile flexibility requirement, the choice of the pile diameter was influenced by the limited dimensions of the testing vessel. Figure 3.7(a) and 3.7(b) show the plan views of a single pile with a diameter of 25 mm and a 9-pile group with diameters of 16 mm installed in the sampling box, respectively. In the figure, S_1 is the distance from the centre of the pile (or the front pile in a group) to the right rigid boundary of the sampling box from which a uniform soil displacement was applied. S_2 is the distance from the centre of the pile (or the back pile in a group) to the left rigid boundary of the sampling box. S_3 is the distance from the centre of the pile (or the side pile in a group) to the upper and lower rigid boundary of the sampling box.

As mention in Section 2.3.1, the 2-D analysis of Chen (1994) showed that when S_1 was larger than $7.5d$, and when S_2 and S_3 were larger than $4.5d$, the results of the analysis were not affected by the boundary. A summary of available boundary conditions of piles in the tank from the literature is shown in Table 3.2. The present boundary conditions met the requirement of the 2-D results by Chen (1994) and the values of S_1 , S_2 and S_3 were also larger than or close to the minimum values in Table 3.2.

Table 3.2 Summary of boundary conditions of piles in the tank

Reference	Pile d (mm)	Single pile or pile group configuration	S_1	S_2	S_3
Matsui et al. (1982)	40	Piles in a row	$3.75d$	$3.75d$	—
Cox et al. (1984)	25.4	Five piles in a row ($S_h = 4d$) Five piles in a line ($S_v = 3d$)	$12.6d$ $6.6d$	$12.6d$ $6.6d$	$4.6d$ $12.6d$
Pan (1998)	20	Single pile	$7.5d$	$7.5d$	$7.5d$
		Two piles in a row ($S_h = 3d$)	$7.5d$	$7.5d$	$6d$
		Two piles in a row ($S_h = 5d$)	$7.5d$	$7.5d$	$5d$
		Two piles in a line ($S_v = 3d$)	$6d$	$6d$	$7.5d$
		Two piles in a line ($S_v = 5d$)	$5d$	$5d$	$7.5d$

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(a) A single pile with a diameter of 25 mm

(b) A 9-pile group with diameters of 16 mm

Figure 3.7 Boundary conditions of piles in sampling box during tests

Tubes

Three tubes of the same length of 380 mm and wall thickness of 1 mm (Figure 3.8) were used to remove the soil and create a vertical hole before pile installation so that the disturbance to the soil during pile installation would be minimal. Tube-1, Tube-2 and Tube-3 were applicable to Pile-1, Pile-2 and Pile-3 in Figure 3.6, respectively. The outer diameter or width of the tube was 3 mm smaller than that of the corresponding pile so that the pile would still be in contact with the soil after pile installation, i.e. no gap (void) would exist.

Figure 3.8 Tubes

LDPE sheets

Unlubricated Low Density Polyethylene plastic sheets (LDPE) with a thickness of 0.25 mm were used in the tests. The specification of the LDPE sheets is as follows:

- Description: film grade F210-6
- Density: 0.922 g/cm³
- Flexural Modulus: 2000 kg/cm³
- Characteristics: excellent process ability, transparency & strength, medium slip

The unlubricated LDPE sheets have been found to be effective in reducing friction by 80% (Chow 2003). The preparatory test results of Chow (2003) also showed that lubrication between the LDPE sheets did not increase the reduction of friction.

Data logger

A data logger TDS-302 was used to automatically record the data from the button load cells, the LVDTs and the load cell during tests, and then transfer data to a computer. A 9-pile group required 21 channels to record all readings, which included 18 channels for the button load cells attached to the piles, 2 channels for the LVDTs to measure the settlement of the kaolin slurry and the lateral soil displacement respectively, and 1 channel for the load cell to measure the load applied by the motor gear loading system.

3.3 Sample Preparation and Testing Procedure**3.3.1 Soil Sample Preparation**

The model tests were carried out on consolidated kaolin clay samples. Commercial kaolin was used for preparing a block clay sample by consolidating kaolin slurry in a consolidometer (see Photo 3.1). After the consolidation of the slurry, the soil sample was set up. Detailed procedures for the consolidation of the kaolin slurry as well as the setting up of the soil samples are described below.

3.3.1.1 Consolidation of Kaolin Slurry

Commercial kaolin was mixed with tap water into a slurry with a water content of 170% (about 2.4 times the liquid limit) by the mixer. From preparatory tests, it was found that 25.8 kg of kaolin was required for each test to get a desired block height of 200 mm after consolidation under 130 kPa. It took two and a half hours to obtain a uniform slurry.

Before pouring the kaolin slurry, two sheets of filter cloth and one sheet of filter paper were placed on the base of the consolidometer to prevent the loss of fines during consolidation. An 'O' ring was placed in the groove of the base to prevent any leakage between the base and the cylinder of the consolidometer. Then the cylinder was placed on the base by a crane and bolted to the base. The smooth inner

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surface of the cylinder was greased subsequently so as to minimise the friction between the soil and the cylinder wall.

The kaolin slurry was scooped manually from the mixer and poured into the consolidometer. Care was taken to reduce the inclusion of air bubbles during this process by allowing the slurry to flow laterally under its own weight. During this process, two slurry samples from the upper and lower portions were taken for water content determination. After pouring, one sheet of filter paper and two sheets of filter cloth were placed on the surface of the slurry. Then the top cover with a piston of the consolidometer was installed and bolted to the cylinder. Two drainage outlets, one from the top of the piston and the other from the base of the consolidometer, were then connected to a plastic container so that the top and bottom of the slurry were at the same piezometric level.

A low air pressure of 40 kPa was applied initially to squeeze out the air and water trapped between the piston and the surface of the slurry as well as in the hole of the piston rod. Then a constant consolidation air pressure of 130 kPa was applied to the consolidometer. The air pressure was controlled and monitored by the test gauge with an accuracy of $\pm 0.25\%$.

In order to check the completion of primary consolidation of the slurry, a TML CDP-100 LVDT was mounted on the top of the piston to measure the settlement of the slurry. The data from the LVDT were recorded by the data logger TDS-302. The primary consolidation was assumed to be complete when the measured settlement of the slurry was less than 0.02 mm for a two-hour period. Generally, it took 96 hours (4 days) for primary consolidation to be complete. Then the air pressure was slowly released to zero.

The procedure for consolidation of the kaolin slurry is summarised as follows:

1. Mix kaolin with tap water into a slurry.
2. Place filter cloth, filter paper and 'O' ring on the base of the consolidometer.
3. Bolt the cylinder to the base of the consolidometer.

4. Pour slurry into the consolidometer.
5. Take two slurry samples for water content determination.
6. Place filter cloth and filter paper on the surface of the slurry.
7. Install the top cover of the consolidometer.
8. Connect drainage outlets.
9. Apply a low air pressure of 40 kPa to squeeze out the air.
10. Apply a constant consolidation air pressure of 130 kPa.
11. Install a LVDT and monitor the settlement of the slurry.
12. Release the air pressure to zero after primary consolidation is complete.

3.3.1.2 Soil Sample Set-up

After the consolidation of the kaolin slurry, the cylinder of the consolidometer was unbolted from the base and lifted up by a crane. Then the circular soil sample of 400 mm diameter and 200 mm height was pushed slowly out of the consolidometer onto a 320 mm wide, 640 mm long and 15 mm thick wooden plate (with a smooth and lubricated surface) by applying an air pressure of 10 kPa to the piston of the consolidometer.

Subsequently, the sampling box of $300 \times 300 \times 170$ mm (with the cutting edge of 20 mm height) was placed on the top surface of the sample and the consolidometer was used as a weight to push the sampling box vertically down to trim off the sides of the circular sample. Since the cutting edge was sharp and the internal walls of the sampling box were well greased, the disturbance to the sample due to trimming was minimal. The side trimmings from the sample were used for water content determination. The excess top portion of soil (about 30 mm) was trimmed off by a steel wire. Then the soil sample together with the sampling box was rotated 90 degrees and the cutting edge of the sampling box was removed. The excess bottom portion of soil (20 mm) was also trimmed off by the steel wire. Finally, the sample together with the main box of the sampling box was rotated back to its horizontal position on the wooden plate.

Pan (1998) previously reported that the water content of a consolidated soil sample of 200 mm height was almost constant with depth except near the top and the bottom of the sample. Therefore, the approaches mentioned above to trim off the top and bottom portions of the soil were used to ensure that the water content was almost constant in the remaining 150 mm of the soil block.

The procedure for soil sample set-up is summarised as follows:

1. Unbolt the cylinder from the base of the consolidometer and lift up the cylinder.
2. Apply an air pressure of 10 kPa to the piston.
3. Push the soil sample out of the consolidometer onto a wooden plate.
4. Place the sampling box on the sample and trim it using the sampling box.
5. Use the side trimmings for water content determination.
6. Trim off the excess top portion of the soil.
7. Rotate the soil sample together with the sampling box 90 degrees.
8. Remove the cutting edge of the sampling box.
9. Trim off the excess bottom portion of the soil.
10. Rotate the soil sample together with the sampling box back.

3.3.2 Single Pile Installation and Loading

The procedure for single pile installation and loading is illustrated in Figure 3.9, and the view of the final assembled apparatus prior to testing is shown in Photo 3.10.

First, the base plate was bolted to the tank and then three layers of plastic sheets were placed on the base plate, as shown in Figure 3.9(a). As stated in Section 3.2, unlubricated Low Density Polyethylene plastic sheets (LDPE) were used to reduce friction in the tests. To prevent the LDPE sheets from moving or distorting during placing of the sampling box in the tank, two-sided adhesive tape were used between each LDPE sheet and the base of the tank. The top sheet was longer than the middle sheet and the bottom sheet was the shortest so that each sheet could be glued to the base of the tank. After placing the sampling box in the tank, the glued edges of the top and middle sheets were cut off so that they could slide freely. To ensure that the

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pile would not be in contact with the LDPE sheets during loading, a hole that was the same size as the hole in the base plate (refer to Figure 3.3) was cut in the bottom sheet; for the top and middle sheets, the holes were of the same width but twice the length.

Second, the wooden plate together with the sampling box were lifted up and placed on the top of the tank. The sample box together with the sample block was pushed slowly into the tank in the testing position by sliding along the smooth and lubricated surface of the wooden plate. In order to observe the soil surface failure pattern when the test was complete, square grid lines were drawn on the sample surface, as shown in Photo 3.11. The spacing of the grid lines was 20 mm by 20 mm. Then three layers LDPE sheets of the same dimensions were placed on the sample surface, as shown in Figure 3.9(b). To ensure that the pile would not be in contact with the LDPE sheets during loading, a hole that was the same size as the hole in the top plate was cut in the top sheet; for the middle and bottom sheets, the holes were of the same width but twice the length. The purpose of placing the plastic sheets under the bottom and on the top surface of the soil sample was to minimise the friction between the soil and the steel plates, and to envelop the excess holes in the cover and base plates Set B (Figure 3.3). Moreover, since the soil sample moved with the plastic sheets during loading, there would be minimal disturbance to the top and bottom portions of the soil.

Third, the cover plate was placed on the plastic sheets and bolted to the tank to prevent soil surface heave during testing. Next the guiding frame was installed and the tube guiding strip was bolted to the guiding frame on the horizontal bar, as shown in Figure 3.9(c).

To remove the soil and create a vertical hole before pile installation, a tube was pushed by hand through the tube guiding strip into the soil sample, as shown in Figure 3.9(d).

After the tube passed through the entire sample block, the end of the tube was cleaned. Removal of the tube through the guiding strip created a hole that was 3 mm

smaller than the cross-section of the pile. Subsequently the pile guiding strip was bolted to the guiding frame at the same position instead of the tube guiding strip, as shown in Figure 3.9(e).

Then the model pile was pushed vertically by hand through the pile guiding strip into the sample block until the pile cap touched the guiding strip, as shown in Figure 3.9(f).

Two rectangular strips (refer to Photo 3.4), one on the top and the other under the bottom of the tank, together with the button load cells glued on them, were placed on the angle bars of the fixing system. The rectangular strips were moved horizontally and carefully along the slots of the angle bars until the compact head of the button load cell entered into the tiny recess of the pile shaft and was in contact with the pile. This contact between the pile and the reaction frame (rectangular strip) was to ensure pinned-head and pinned-tip conditions of the pile during testing. Then the rectangular strips were fixed, as shown in Figure 3.9(g).

The pushing piston of the motor gear loading system was then moved manually until it was just in contact with the sampling box. Next the load cell, LVDT and button load cells were connected to the data logger TDS-302 and initialised. Before the testing was performed, the guiding frame and the pile guiding strip were removed. Finally, the motor gear loading system was triggered half an hour after pile installation to minimise the effects of pile installation on the test results, as shown in Figure 3.9(h). The displacement of the sampling box was controlled at a constant rate of 1.05 mm/min to ensure undrained conditions. The maximum soil displacement was equal to or more than one diameter or width of the pile so that the ultimate soil pressure could be fully mobilised. The lateral forces acting on the pile were recorded by the button load cells, and the displacement of the sampling box was recorded by a LVDT.

The procedure of single pile installation and loading is summarised as follows:

1. Bolt the base plate to the tank.
2. Place LDPE plastic sheets on the base.
3. Place the wooden plate together with the sampling box on the top of the tank.
4. Move the sampling box together with the sample block to the testing position.
5. Draw grid lines and place LDPE plastic sheets on the sample surface.
6. Place the cover plate and bolt it to the tank.
7. Cut off the glued edges of the LDPE sheets on the base of the tank.
8. Install the guiding frame and the tube guiding strip.
9. Push the tube into the soil to create a vertical hole.
10. Remove the tube and the tube guiding strip.
11. Install the pile guiding strip.
12. Push the pile into the soil.
13. Install the rectangular strips on the angle bars.
14. Move the pushing piston of the motor gear system to the testing position.
15. Connect the load cell, LVDT and button load cells to the data logger.
16. Initialise the load cell, LVDT and button load cells.
17. Remove the guiding frame and the pile guiding strip.
18. Trigger the motor gear loading system and apply a constant displacement rate.
19. Record the data from the load cell, LVDT and button load cells.

Figure 3.9 Procedure of single pile installation and loading

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Photo 3.10 View of assembled apparatus in single pile tests

Photo 3.11 Grid lines on sample block surface

3.3.3 Pile Group Installation and Loading

For the pile group installation, the procedure essentially followed that for single pile installation and loading (illustrated in Figure 3.9). The differences were that the phases from Figure 3.9(d) to 3.9(g) were repeated as the piles were installed one by one in a group in the soil sample. It should be noted that only piles with a diameter of 16 mm were used in the group tests and the number of holes in the plastic sheets were equal to the number of the piles in a group. Pile groups with different numbers of piles (2~9), various pile spacings ($3d$, $4.5d$ and $6d$) and arrangements could be installed in the soil sample using the cover and base plates Set B (Figure 3.3). Photo 3.12 shows the view of the final assembled apparatus in a 9-pile group test prior to testing.

(a) General view

(b) Top view

Photo 3.12 View of assembled apparatus in a 9-pile group test

3.4 Testing Program

A series of laboratory model tests were conducted on single piles and pile groups undergoing lateral soil movements. The testing program was divided into three phases, i.e. preparatory tests, single pile tests and pile groups tests.

The temperature during all tests varied between about 28°C and 33°C. Because the instruments used in the tests were relatively insensitive to the temperature changes mentioned above, the effects of the temperature variations on the results should not be significant.

3.4.1 Preparatory Tests

A series of calibration and soil characterisation tests listed below were carried out before performing the actual model tests. The main results of these preparatory tests are presented in Chapter 4.

- 1) A series of calibration tests were carried out on each instrument, i.e. button load cell, load cell and LVDT. Trial and calibration tests of the in-situ devices were also conducted. For instance, the displacement rate (shown in Figure 3.4) applied by the motor gear loading system was calibrated.
- 2) A series of standard laboratory tests were carried out to determine the basic properties of kaolin, such as water content, density, grain size, liquid limit (LL), plastic limit (PL), specific gravity (G_s). The compressibility of kaolin slurry during consolidation was also studied by measuring the settlement of slurry.
- 3) A series of unconsolidated-undrained (UU) triaxial tests were conducted to determine the undrained shear strength of consolidated sample blocks. Mini-lab vane shear tests were also carried out for comparison.
- 4) A series of consolidated-undrained (CU) triaxial tests and an oedometer test were carried out to determine other engineering properties of the prepared soil samples.

3.4.2 Single Pile Tests

Three model tests for single piles subjected to lateral soil movements were carried out to investigate the effects of pile shape and diameter on the ultimate soil pressure, as listed in Table 3.3. An additional single pile test S-4 was conducted to check the repeatability of the results.

Table 3.3 Testing program for single piles

Test number	Pile shape	d (mm)
S-1	circular	25
S-2	square	25
S-3	circular	16
S-4	circular	16

3.4.3 Pile Group Tests

Generally, piles are often used in groups in practice, and the number of piles in a group usually can differ. In most situations, the ultimate soil pressure acting on an individual pile within a group is different from a single pile, resulting in a reduction of pile group capacity. The reduction depends mainly on the pile spacing, the number of piles and the arrangement of piles in a group. To better understand the group effects on the response of an individual pile within a group, it is desirable to carry out pile group tests and compare the results with those of single pile tests. Therefore, fifteen model tests for pile groups subjected to lateral soil movements were carried out, as listed in Table 3.4. Different numbers of piles (2~9) in a group are arranged in various configurations with different pile spacings, as shown in Figures 3.10 to 3.15. The terms “front”, “middle” or “back” are used, for convenience, to name a pile in terms of its position relative to the direction of soil movement; the terms “side” and “centre” are used to name a pile in terms of its relative position in a group. According to the definition, a “front” pile encounters the soil movement before a “back” pile. Because of the limited dimensions of the testing vessel, only circular piles with a diameter of 16 mm were used for group tests.

Table 3.4 Testing program for pile groups

Test number	Num.of piles	Reference of arrangement	S_H/d	S_V/d
G-1	2	Fig. 3.10(a)	3	—
G-2	2	Fig. 3.10(b)	6	—
G-3	2	Fig. 3.10(c)	—	3
G-4	2	Fig. 3.10(d)	—	4.5
G-5	2	Fig. 3.10(e)	—	6
G-6	3	Fig. 3.11(a)	3	—
G-7	3	Fig. 3.11(b)	—	3
G-8	3	Fig. 3.11(c)	3	3
G-9	3	Fig. 3.11(d)	3	3
G-10	4	Fig. 3.12(a)	3	3
G-11	4	Fig. 3.12(b)	6	6
G-12	5	Fig. 3.13	3	3
G-13	6	Fig. 3.14(a)	3	3
G-14	6	Fig. 3.14(b)	3	3
G-15	9	Fig. 3.15	3	3

Figure 3.10 Two-pile groups

Figure 3.11 Three-pile groups

Figure 3.12 Four-pile groups

Figure 3.13 A five-pile group

Figure 3.14 Six-pile groups

Figure 3.15 A nine-pile group

3.5 Summary

This chapter presents the development of the laboratory apparatus for measuring the ultimate soil pressure acting on single piles and pile groups when subjected to lateral soil movements. The testing procedures for single piles and pile groups have been illustrated. The results from the present experimental work should provide insights into the soil-pile interaction behaviour and enhance the state-of-the-art on passive pile behaviour.

By means of this test set-up, four model tests for single piles with different shapes and diameters were carried out to investigate the effects of pile shape and diameter on the ultimate soil pressure. Another fifteen model tests for pile groups with different numbers of piles, pile spacings and pile arrangements were carried out to investigate the group effects on the response of individual piles within a group. The results of these tests are presented in Chapter 4 and 5, respectively.

Chapter 4

Experimental Results

Part I: Preparatory and Single Pile Tests

4.1 Introduction

This chapter firstly presents the results of preparatory tests to determine the engineering properties of the clay samples, followed by the results of model tests on single piles subjected to lateral soil movements. The results of pile group tests are presented in Chapter 5.

4.2 Preparatory Test Results

A series of preparatory tests were carried out to determine the engineering properties of the clay samples. The results of basic properties of kaolin, kaolin slurry compressibility, undrained shear strength and other engineering properties of the consolidated clay samples are presented below.

4.2.1 Basic Properties of Kaolin

As mentioned in Section 3.3.1, commercial kaolin was used for preparing the soil samples for the tests. The main advantages of kaolin are its relatively high permeability, commercial availability and consistently uniform properties of the prepared soil samples.

A series of standard laboratory tests, i.e. specific gravity, Atterberg limit and particle-size analysis, were carried out to determine the basic properties of kaolin. Figure 4.1 shows the particle size distribution of kaolin. The other basic properties of kaolin are listed in Table 4.1.

Figure 4.1 Particle size distribution of kaolin

Table 4.1 Basic properties of kaolin

Property	Value
Specific gravity, G_s	2.62
Liquid limit, LL	75%
Plastic limit, PL	42%

4.2.2 Compressibility of Kaolin Slurry

As mentioned in Section 3.3.1.1, in order to check the completion of primary consolidation of a slurry, a TML CDP-100 LVDT was mounted on the top of the consolidometer piston to measure the settlement of the slurry. A typical settlement plot of kaolin slurry under a consolidation pressure of 130 kPa is shown in Figure 4.2. Consistent results were observed in all consolidation tests of the soil samples. Coefficient of vertical consolidation, c_v , was estimated using the t_{90} method by plotting the settlement against the square root of the time. The estimated c_v was about 40 m²/yr. It was assumed that primary consolidation was complete when the

settlement of the slurry was less than 0.02 mm for a two-hour period. This took 96 hours (4 days).

Figure 4.2 Settlement curve of kaolin slurry

4.2.3 Undrained Shear Strength of Soil Samples

The undrained shear strength c_u of clay depends on its water content, stress history and current effective stress. Shear strength may be measured by various methods both in the field and in the laboratory, and the results of different tests on identical samples are likely to be different. In an attempt to obtain reliable values of c_u of the soil samples, three kaolin block samples were prepared from slurry using the procedure described in Section 3.3.1. Unconsolidated-undrained (UU) triaxial tests were carried out on two of the blocks (C-1 and C-2) to determine c_u and mini-lab vane shear tests were carried out on the third block (C-3) for comparison.

Standard unconsolidated-undrained triaxial tests (UU) were conducted on specimens of 38 mm diameter and 76 mm height with a confining pressure of 300 kPa to determine the undrained shear strength. As sample storage may result in disturbance, UU tests were carried out on the specimens that were freshly taken from the soil block after consolidation. A typical stress-strain curve is shown in

Chapter 4. Experimental Results Part I: Preparatory and Single Pile Tests

Figure 4.3. Almost all the specimens failed by plastic shearing at about 5% axial strain. The average undrained shear strength was approximately 24 kPa and the following equation was obtained:

$$c_u / \sigma_v' = 0.185 \quad (4-1)$$

in which c_u is the undrained shear strength and σ_v' is the overburden stress. This agreed well with the c_u/σ_v' ratio of 0.18 reported by Pan (1998) and 0.185 reported by Stewart (1992) for laboratory tests of consolidated kaolin.

Figure 4.3 Typical stress-strain curve

For comparison, mini-lab vane shear tests were conducted to determine the undrained shear strength. The testing followed the procedure described by Head (1982). The apparatus was self-contained with a vane measuring 12.7 by 12.7 mm. The tests were carried out directly on the consolidated circular block sample of 400 mm diameter. The locations of five tests are shown in Figure 4.4. After completing five tests, the top portion of the soil (about 75 mm high) was trimmed off by a steel wire and another five tests were carried out at the same locations.

Figure 4.4 Plan view of vane test locations in soil sample

The average value of all ten undisturbed shear strengths was approximately 28 kPa and was slightly larger than the c_u of 24 kPa from UU tests. Bjerrum (1973) suggested that correction factors should be considered in field vane tests (VST). However, few correction factors have been suggested for laboratory vane tests. If a correction factor of 0.86 from the field VST correlation by Bjerrum (1973) is used (with $PI = LL - PL$ shown in Table 4.1), the corrected undrained shear strength was also 24 kPa and in agreement with the UU test results.

4.2.4 Consistently Uniform Properties of Soil Samples

In order to obtain uniform soil samples with almost uniform undrained shear strength for all the model tests, the following operating conditions were adopted:

- 1) 27 bags of commercial kaolin clay were purchased in bulk to ensure consistency of the kaolin.
- 2) 25.8 kg of kaolin together with 44.2 kg of tap water were used in each consolidation test so as to obtain a kaolin slurry with the same water content.
- 3) During consolidation, the same air pressure of 130 kPa (monitored by the test gauge with an accuracy of $\pm 0.25\%$) was applied to the consolidometer.

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- 4) The undrained shear strength of the soil sample depends on its water content. Pan (1998) previously reported that the water content of a consolidated soil sample of 200 mm height was almost constant with depth except near the top and the bottom of the sample, as shown in Figure 4.5. As explained in Section 3.3.1.2, in this project the top and bottom portions of the soil were trimmed off and the water content was found to be almost constant in the remaining 150 mm of the soil block (average water content = 59.6%).
- 5) As mentioned in Section 3.3.1.2, the trimmings from the sides of the circular sample were used to determine the water content and subsequently verify the undrained shear strength.

Figure 4.5 Water content with depth

Table 4.2 shows the results for all the soil samples. It can be seen that the water content and the undrained shear strength of different consolidated samples were almost identical, verifying the consistently uniform properties of the prepared soil samples. The average undrained shear strength was 24.2 kPa.

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Table 4.2 Properties of the soil samples

Test Number	Slurry water content (%)	Sample water content (%)	c_u (kPa)
C-1	171	59.8	23.8
C-2	174	59.6	24.3
S-1	174	60.0	24.1 [*]
S-2	175	59.9	24.4 [*]
S-3	170	59.4	24.0 [*]
S-4	172	59.6	24.8 [*]
G-1	169	58.9	24.8 [*]
G-2	170	59.7	24.5 [*]
G-3	174	59.7	24.2 [*]
G-4	174	60.0	24.2 [*]
G-5	175	59.7	24.6 [*]
G-6	169	59.2	24.9 [*]
G-7	175	60.1	23.5 [*]
G-8	174	59.4	23.5 [*]
G-9	174	59.6	23.8 [*]
G-10	172	59.8	23.7 [*]
G-11	172	59.4	24.8 [*]
G-12	174	59.6	23.7 [*]
G-13	174	59.9	24.2 [*]
G-14	175	60.2	23.4 [*]
G-15	170	59.3	24.5 [*]
Average	173	59.6	24.2

^{*}The value obtained from the side trimmings

4.2.5 Other Engineering Properties of Soil Samples

An oedometer test and three consolidated-undrained (CU) triaxial tests were carried out to study the other engineering properties of the prepared soil samples. Undisturbed specimens were taken from the mid-height of the prepared soil sample blocks (C-1 and C-2).

The oedometer test procedure strictly followed the standard BS1377 (Head 1982). The inside wall of the cutting ring was greased with silicon to minimise the side friction. Saturated filter papers were placed on each end of the specimen before it was positioned into the oedometer. In all, eight load increments were employed with a maximum total vertical stress of 1100 kPa. The typical load history used was: loading from 35 kPa, 70 kPa, 140 kPa, 280 kPa, 550 kPa, eventually to 1100 kPa, and then unloading to 280 kPa, and 70 kPa. The tests were conducted in a temperature-controlled room with the range of 23 ± 2 °C. The oedometer test took 7 days to complete.

Figure 4.6 shows a typical void ratio/log pressure curve. The preconsolidation pressure estimated from the void ratio/log pressure curve using the Casagrande construction method was approximately 120~130 kPa, which was in agreement with the actual applied consolidation pressure of 130 kPa. The initial void ratio e_0 was 1.494, the compression index C_c was 0.55, the recompression index C_r was 0.15, the coefficient of permeability k obtained under the load increment from 70 kPa to 140 kPa was 1.1×10^{-8} m/s and the saturated unit weight γ was 16.4 kN/m³.

Three standard consolidated-undrained (CU) triaxial tests were performed on undisturbed specimens, following the procedure described by Head (1982). The specimens, 38 mm in diameter and 76 mm in height, were vertically trimmed from the block sample. The specimens were nearly fully saturated with the degree of saturation of more than 98%. Prior to shearing, they were consolidated with the cell pressure of 200, 300, and 500 kPa, respectively, while the back pressure was set to 100 kPa. The effective friction angle ϕ' from the obtained Mohr-Coulomb failure line, as shown in Figure 4.7, was 22.6°.

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Figure 4.6 Void ratio/log pressure curve

Figure 4.7 Consolidated-undrained (CU) triaxial test results

4.3 Single Pile Tests

As listed in Table 3.3, three model tests for single piles subjected to lateral soil movements were carried out to investigate the effects of pile shape and diameter on the ultimate soil pressure, and another test was conducted to check the repeatability of the results. This section presents the results of single pile tests and highlights the major findings.

The temperature during all model tests varied between about 28°C and 33°C. Because the instruments used in the tests were relatively insensitive to the temperature changes mentioned above, the effects of the temperature variations on the results should not be significant. In all model tests, the same constant displacement rate of 1.05 mm/min was used to ensure undrained conditions, as the coefficient of consolidation c_v (40 m²/yr) and the coefficient of permeability k (1.1×10^{-8} m/s) determined by one-dimensional consolidation test were low.

4.3.1 Determination of p_u from Pressure-Displacement (p - y) Curves

In general, load-displacement (P - y) or pressure-displacement (p - y) curves on pile foundations can exhibit any one of the three shapes (Hirany and Kulhawy 1989), as shown in Figure 4.8. The peak value of curve I and the asymptote value of curve II give the ultimate resistance of the pile foundations. However, most pile load test data (e.g., Matsui et al. 1982; Hirany and Kulhawy 1989; Pan et al. 2000) resembled curve III. Experience and judgement are needed to define the failure point as Hirany and Kulhawy (1989) have suggested. The criterion used in this study is based on the approach suggested by Hirany and Kulhawy (1989) for interpreting the results of the model piles in which load-displacement curves resembled curve III. The same approach was also used by Pan et al. (2000) and is introduced briefly below.

Basically, curve III can be simplified into three distinct regions, initial linear (OA), transition (AB) and final linear region (BC), as illustrated in Figure 4.9. When the pressure p is larger than the value of the start-point (B) of the final linear portion,

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creep displacement, i.e. a large displacement for a small incremental pressure, will occur.

Therefore, if the pressure-displacement curves resembled curve III in the present study, the ultimate soil pressure p_u acting on the pile(s) is defined as the pressure when it starts to take on the final linear trend, i.e. the pressure at point B in Figure 4.9. The above criterion was selected because there were some difficulties in defining the failure point for curve III in a quantitative manner to obtain consistent results.

In the traditional p - y curves used for active piles (e.g. Brown et al. (1988), McVay et al. (1995) and Reese (1984)), the p is in units of force/length and the y is in units of length. In this study, for convenience, the pressure-displacement (p - y) curves from the model test results refer to the normalised pressure $p/c_u = P/(l \times d \times c_u)$ vs. normalised displacement $y = l/d$ in which l is the pile length and d is the pile width. Hence the curves presented in this study have different units from those of the traditional p - y curves.

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Figure 4.8 Typical load-displacement curves

Figure 4.9 Determination of p_u from curve III

4.3.2 Single Circular Pile with a Diameter of 25 mm (S-1)

A test was firstly performed on a circular pile with a diameter of 25 mm. The testing procedure was presented in Section 3.3. The maximum soil displacement was equal to the diameter of the pile so that the ultimate soil pressure could be fully mobilised. The lateral forces acting on the pile were recorded by the button load cells, and the displacement of the sampling box was recorded by a LVDT. As indicated in Section 4.2.3, the average undrained shear strength c_u of the kaolin samples was 24 kPa.

Figure 4.10 shows the normalised pressure-displacement (p - y) curve for Test S-1. The average soil pressure p was determined by dividing the measured lateral force P by the pile diameter d multiplied by the pile length l , i.e. $p = P/(l \times d)$. It was found that the curve remained linear until $p/c_u \approx 4.5$, which was close to the value of 5 predicted by Baguelin et al. (1977). The normalised ultimate soil pressure p_u/c_u obtained using the definition described in Section 4.3.1 was 11.3 ($y/d = 0.73$). It was within the range of 9.14~11.94 suggested by Randolph and Houlsby (1984) and a little higher than the value of 10.6 obtained by Pan et al. (2000) for a rectangular rigid pile with fixed head and tip conditions. It was also similar to the values obtained by Chen (1994) and Bransby and Springman (1999) in their two-dimensional numerical passive pile analysis, in which the p_u/c_u was 11.4 and 11.75, respectively.

When the test was complete, the cover plate on the sampling box was removed and the pile was gently extracted out so as to observe the surface deformation of the soil sample. Photo 4.1 shows the soil surface failure pattern when the test was complete ($y/d = 1.0$). The spacing of grid lines was 20 mm by 20 mm. There was a semicircular gap (void) of approximately $1d$ at the back of the pile and two cracks at an angle of 80° with respect to the direction of soil movement on both sides of the pile. A near semicircular affected zone of about $2.5d$ (d is the pile diameter) was observed in front of the model pile.

Figure 4.10 Normalised p - y curve for Test S-1

Photo 4.1 Failure pattern for Test S-1 at $y/d = 1.0$

4.3.3 Single Square Pile with a Width of 25 mm (S-2)

To investigate the effects of pile shape on the ultimate soil pressure, Test S-2 was performed on a square pile with a width of 25 mm. Figure 4.11 shows the normalised p - y curve for Test S-2. The curve was almost identical to that of Test S-1 when p/c_u was smaller than 5. The normalised ultimate soil pressure p_u/c_u was 12.6 ($y/d = 0.66$). Compared with Test S-1, it was found that the value of p_u/c_u for a single square pile was 12% higher than that for a single circular pile. This was close to the 15% presented by Adachi et al. (1989) for pile groups with different shapes. The ultimate soil displacement y_u for a square pile was smaller than that for a circular pile.

Photo 4.2 shows the soil surface failure pattern for Test S-2 ($y/d = 1.0$). There was a square gap (void) of approximately $1d$ at the back of the pile and two cracks at an angle of about 45° with respect to the direction of soil movement on both sides of the pile. A circular affected zone of about $3d$ was observed in front of the pile. This was larger than that of the single circular pile (shown in Photo 4.1). This reflected the different shapes of the mobilised soil wedge fan for the different shapes of the piles. The square pile caused a wider wedge fan than the circular pile. Similar results were also found in the three-dimensional finite difference analyses by Chen (2001). The square pile caused a wider wedge fan than the circular pile, and the ultimate soil pressure developed for a square pile was also about 12% higher than that for a circular pile.

Figure 4.11 Normalised p - y curve for Test S-2

Photo 4.2 Failure pattern for Test S-2 at $y/d = 1.0$

4.3.4 Single Circular Pile with a Diameter of 16 mm (S-3)

To investigate the effects of pile diameter on the ultimate soil pressure, Test S-3 was carried out on a circular pile with a smaller diameter of 16 mm. For convenience in comparison with other cases, especially the pile group tests where only circular piles with diameters of 16 mm were used, this test is referred to hereafter as the “standard” test. The maximum soil displacement for this test was $1.1d$.

Figure 4.12 shows the normalised p - y curve for Test S-3. It was found that the curve was quite similar with that of Test S-1. The normalised ultimate soil pressure p_u/c_u was 11.1 ($y/d = 0.78$). The discrepancy of p_u/c_u for Test S-1 and Test S-3 was only 1.8%. It indicated that the ultimate soil pressure was not affected by the pile diameter once the ultimate soil pressure could be fully mobilised. The ultimate soil displacement y_u differed slightly with different pile diameters.

Photo 4.3 shows the soil surface failure pattern for Test S-3 ($y/d = 1.1$). It was quite similar to that for Test S-1 (shown in Photo 4.1). There was a semicircular gap (void) of approximately $1.1d$ at the back of the pile and two cracks approximately perpendicular to the direction of soil movement on both sides of the pile. A near semicircular affected zone of about $2.5d$ was also observed in front of the pile.

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Figure 4.12 Normalised p - y curve for Test S-3

Photo 4.3 Failure pattern for Test S-3 at $y/d = 1.1$

4.3.5 Validation Test (S-4)

An additional test S-4 was conducted on a 16 mm diameter circular pile to check the repeatability of the test results. Figure 4.13 shows the normalised p - y curve for Test S-4. A slight difference between Test S-4 and Test S-3 was observed when y/d was larger than 0.2. The normalised ultimate soil pressure p_u/c_u for Test S-4 was 10.8 ($y/d = 0.78$), which was 2.7% smaller than that for Test S-3. It indicated that the test results were repeatable and reliable under the same testing procedure and operating conditions.

Figure 4.13 Normalised p - y curve for Test S-4

4.4 Summary

A series of preparatory tests were carried out to determine the engineering properties of the clay samples. The results are summarised in Table 4.3. The undrained shear strength from mini-lab vane shear tests was slightly larger than that from UU tests. If a correction factor for plasticity index is used for the mini-lab vane shear tests, the undrained shear strength agreed well with that from UU tests. Under the same operating conditions, it was found that the water content and the undrained shear strength of different consolidated samples were almost identical, verifying the consistently uniform properties of the prepared soil samples.

Table 4.3 Engineering properties of clay

Property	Value
Specific gravity, G_s	2.62
Liquid limit, LL (%)	75
Plastic limit, PL (%)	42
Preconsolidation pressure, σ_p' (kPa)	120~130
Initial void ratio, e_0	1.494
Compression index, C_c	0.55
Recompression index, C_r	0.15
Coefficient of consolidation, C_v (m ² /yr)	40
Coefficient of permeability, k (m/s)	1.1×10^{-8}
Effective angle of friction, ϕ' (°)	22.6
Undrained shear strength, c_u (kPa)	24
Water content, w (%)	59.6
Unit weight, γ (kN/m ³)	16.4

Three model tests for single piles subjected to lateral soil movements were carried out to investigate the effects of pile shape and diameter on the ultimate soil pressure, and an additional single pile test was conducted to check the repeatability of the results. The main results are summarised in Table 4.4 and described below:

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- 1) The ultimate soil pressure p_u for single circular piles ranged from $10.8c_u$ to $11.3c_u$. The y_u to fully mobilise the p_u ranged from $0.73d$ to $0.78d$.
- 2) The p_u for a single square pile was $12.6c_u$, which was 12% higher than that for a single circular pile. The y_u for a square pile was $0.66d$, which was smaller than that for a circular pile.
- 3) The p_u was almost the same for single piles with different diameters. It indicated that the p_u was generally independent of pile diameter. The y_u differed slightly with different pile diameters.
- 4) The test results were repeatable and reliable under the same testing procedure and operating conditions.

Table 4.4 Summary of single pile tests

Test Number	Shape	d (mm)	p_u/c_u	y_u/d
S-1	Circular	25	11.3	0.73
S-2	Square	25	12.6	0.66
S-3	Circular	16	11.1	0.78
S-4	Circular	16	10.8	0.78

Chapter 5

Experimental Results

Part II: Pile Group Tests

5.1 Introduction

In practice, piles are often used in groups. In most situations, the ultimate soil pressure acting on an individual pile within a group is different from a single pile, resulting in a reduction of pile group capacity. The reduction depends mainly on the pile spacing, the number of piles and the arrangement of piles in a group. To better understand the group effects on the response of an individual pile within a group, a series of laboratory model tests on pile groups subjected to lateral soil movements were conducted. This chapter presents the results of pile group tests and highlights the major findings. Where possible, comparisons are also made with test results from the literature.

For convenience, the term “piles in a row” refers to the piles for which the centre-to-centre line is perpendicular to the direction of soil movement; the term “piles in a line” refers to the piles for which the centre-to-centre line is in the direction of soil movement.

5.2 Evaluation of Group Effects

To investigate the group effects on the lateral pile response, it is desirable to define a suitable comparison method. Poulos and Davis (1980) suggested that the group effects might be assessed in terms of loadings or head deflections for laterally loaded piles. In this study, the following expressions were adopted to evaluate group effects of individual piles in a group:

$$F_{pi} = p_{ui} / p_{us} \quad (5-1)$$

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$$F_{yi} = y_{ui} / y_{us} \quad (5-2)$$

in which, F_{pi} = group factor in terms of the ultimate soil pressure for an individual pile in a group,

p_{ui} = ultimate soil pressure for an individual pile in a group,

p_{us} = ultimate soil pressure from the “standard” test,

F_{yi} = group factor in terms of the ultimate soil displacement for an individual pile in a group,

y_{ui} = ultimate soil displacement for an individual pile in a group, and

y_{us} = ultimate soil displacement from the “standard” test.

As mentioned in Section 4.3.4, the “standard” test refers to Test S-3, i.e. the test for a single circular pile with a diameter of 16 mm. The ultimate soil displacement is defined as the minimum soil displacement to fully mobilise the ultimate soil pressure. The p_{us} was $11.1c_u$ and the y_{us} was $0.78d$ from the “standard” test (shown in Figure 4.12).

At the same time, the following expressions were adopted to evaluate group effects of a row of piles in a group and the whole group as well:

$$F_{pri} = \left[\sum_{j=1}^{n(i)} F_{pj}(i) \right] / n(i) \quad (5-3)$$

$$F_p = (F_{p1} + F_{p2} + \dots + F_{pn}) / n \quad (5-4)$$

in which, F_{pri} = group factors in terms of the ultimate soil pressure for the i row of piles in a group,

F_p = group factor in terms of the ultimate soil pressure for a whole group,

$F_{p1}, F_{p2}, F_{pn}, F_{pj}(i)$ = group factors for an individual pile in a group calculated using Equation (5-1),

$n(i)$ = number of piles in the i row, and

n = number of piles in a group.

It should be noted that different individual piles in a group might be subjected to the ultimate soil pressure at different magnitudes of lateral soil movement. Therefore, Equations (5-3) and (5-4) may not reflect the actual group factors for a row of piles or a whole group, but they can provide the worst scenario or situation.

5.3 Two-Pile Group Tests

In deciding the pile spacing and arrangement to be adopted in the tests, consideration was given to the dimensions of the testing vessel. Because of the limited dimensions of the present vessel, only circular piles with diameters of 16 mm were used for group tests. To investigate the effects of pile spacing on the response of a two-pile group, two different centre-to-centre pile spacings, i.e. $3d$ and $6d$ (d is the pile diameter), were adopted for two piles in a row, and three different centre-to-centre pile spacings, i.e. $3d$, $4.5d$ and $6d$, were adopted for two piles in a line. The same constant displacement rate of 1.05 mm/min was used to ensure undrained conditions. The same operating conditions described in Section 4.2.4 were adopted to obtain soil samples with an undrained shear strength of 24 kPa.

5.3.1 Two Piles in a Row (G-1 and G-2)

Test G-1 and G-2 were carried out on two piles in a row with spacings of $3d$ and $6d$, respectively, as illustrated schematically in Figure 3.10 (a) and (b). Figure 5.1 shows the normalised p - y curves for Test G-1 and G-2. The two curves were linear and almost identical as that of the single pile test S-3 when p/c_u was smaller than 3.5. However, with increased soil displacement, the p - y curve of Test G-2 closely resembled that of Test S-3 and differed significantly from that of Test G-1. For Test G-1, the normalised ultimate soil pressure p_u/c_u (using the definition in Section 4.3.1) was 8.8 ($y/d = 0.75$). This was smaller than the p_u/c_u of 11.1 of the single pile test S-3. For Test G-2, the p_u/c_u was 10.8 ($y/d = 0.8$), which was much closer to the p_u/c_u of 11.1 of the single pile test S-3. It indicated that the piles for a two-pile group in a row behaved like isolated single piles when the pile spacing was $6d$ or larger.

Chapter 5. Experimental Results Part II: Pile Group Tests

When the test was complete, as with the single pile tests, the cover plate on the sampling box was removed and then the piles were gently extracted out so as to observe the surface deformation of the soil sample. Photo 5.1 shows the soil surface failure pattern for Test G-1 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of each pile and obvious cracks perpendicular to the direction of soil movement on both sides of the piles. The cracks between the two piles intersected and were connected to each other. Based on the observed deformations of the grid lines, there was a large arc-shaped affected zone of about $2.5d$ width in the direction of soil movement.

Photo 5.2 shows the soil surface failure pattern for Test G-2 ($y/d = 1.1$). The failure pattern for each pile was quite similar to that observed in the single pile test S-3 (shown in Photo 4.3), and differed from that in Test G-1 (shown in Photo 5.1). The cracks developed on both sides of each pile but did not intersect each other, as the piles were too far apart. A disconnected affected zone was observed in front of each pile. This also indicated that the piles for a two-pile group in a row behaved like isolated single piles when the pile spacing was $6d$ or larger.

For comparison, the results from this study and those of Chen and Poulos (1997) and Pan et al. (2002) are shown in Table 5.1 and Figure 5.2. For this study and Pan et al. (2002), the group factor F_{pi} was calculated using Equation (5-1). For Chen and Poulos (1997), the F_{pi} was based on the measured bending moment. All results showed the same trend of group factors increasing with increased pile spacing. With a small pile spacing ($2.5d$ to $3d$), there was a significant reduction in F_{pi} (more than 20%) in comparison to a single pile. The present results showed little group effect ($F_{pi} = 0.97$) when the pile spacing was $6d$. It indicated that the piles behaved like isolated single piles when the pile spacing was $6d$ or larger. According to Chen and Poulos (1997) and Pan et al. (2002), group effects still existed when the pile spacing was $5d$.

Figure 5.1 Normalised p - y curves for Test G-1 ($S_h = 3d$) and G-2 ($S_h = 6d$)

Photo 5.1 Failure pattern for Test G-1 at $y/d = 1.1$

Photo 5.2 Failure pattern for Test G-2 at $y/d = 1.1$

Table 5.1 Group factors for two piles in a row

Reference	Soil type	Pile type and fixity conditions	d (mm)	Spacing	F_{pi}
Present	Clay	Passive, head and tip pinned	16	$3d$	0.79
				$6d$	0.97
Chen and Poulos (1997)	Sand	Passive, free head	25	$2.5d$	0.81
				$5.0d$	0.88
				$7.5d$	0.98
Pan et al. (2002)	Clay	Passive, head and tip fixed	20×6 (rect.)	$3d$	0.67
				$5d$	0.81

Figure 5.2 Effects of pile spacing on F_{pi} for two piles in a row

5.3.2 Two Piles in a Line (G-3, G-4 and G-5)

Test G-3, G-4 and G-5 were carried out on two piles in a line with spacings of $3d$, $4.5d$ and $6d$, respectively, as illustrated schematically in Figure 3.10 (c), (d) and (e). According to the definition in Section 3.4.3, a “front” pile encounters the soil movement before a “back” pile.

Figure 5.3 shows the normalised p - y curves for Test G-3 with $S_v = 3d$. The curves of the “front” and “back” piles were almost identical when p/c_u was smaller than 4.5. However, for the “front” pile, the p - y curve had an upward trend with increased soil displacement, similar to that of the single pile test S-3. The normalised ultimate soil pressure p_u/c_u was 9.7 ($y/d = 0.75$). For the “back” pile, some slight fluctuations occurred when y/d was close to 0.1. The normalised ultimate soil pressure occurred at $p_u/c_u = 5.3$ ($y/d = 0.4$), and thereafter the p/c_u gradually reduced with increased soil displacement.

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Figure 5.4 shows the normalised p - y curves for Test G-4 with $S_v = 4.5d$. As with Test G-3, the p - y curve of the “front” pile had an upward trend with increased soil displacement. The p_u/c_u of the “front” pile was 9.8 ($y/d = 0.77$). For the “back” pile, some slight fluctuations occurred when y/d was close to 0.2 and the p_u/c_u was 7.2 ($y/d = 0.48$).

Figure 5.5 shows the normalised p - y curves for Test G-5 with $S_v = 6d$. For the “front” pile, the shape of the p - y curve was similar to that of the single pile test S-3, and the p_u/c_u was 10.1 ($y/d = 0.76$). For the “back” pile, the p - y curve had an upward trend with increased soil displacement, and the p_u/c_u was 8.4 ($y/d = 0.75$).

A comparison of Figures 5.3, 5.4 and 5.5 indicated that there were minimal differences in the p - y curves for the “front” piles at different pile spacings. However, for the “back” pile, with increased pile spacing, the p - y curve became closer to that of the “front” pile.

Photo 5.3 shows the soil surface failure pattern for Test G-3 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of the “back” pile and obvious cracks at an angle of 45° with respect to the direction of soil movement on both sides of the “back” pile. There was only a little gap (void) at the back of the “front” pile and no obvious cracks on the sides of the “front” pile. Obvious shear failure between the two piles was observed in the sheared zone and the arc-shaped affected zone at the front of the “front” pile was similar to that of the single pile test S-3 (shown in Photo 4.3).

Photo 5.4 shows the soil surface failure pattern for Test G-4 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of the “back” pile and obvious cracks at an angle of 60° with respect to the direction of soil movement on both sides of the “back” pile. A smaller semicircular gap (void) of approximately $0.8d$ developed at the back of the “front” pile and no obvious cracks were observed on the sides of the “front” pile. There was no obvious shear failure between the two piles in Test G-4 as in Test G-3 (shown in Photo 5.3), but there was an affected zone between the two piles and at the front of the “front” pile.

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Photo 5.5 shows the soil surface failure pattern for Test G-5 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of the “back” pile and obvious cracks nearly perpendicular to the direction of soil movement on both sides of the “back” pile. A similar semicircular gap (void) developed at the back of the “front” pile and no obvious cracks were observed on the sides of the “front” pile. No sheared zone was observed between the two piles and the affected zone in front of each pile was similar to that of the single pile test S-3 (shown in Photo 4.3).

A comparison of Photos 5.3, 5.4 and 5.5 indicated that the shear failure between two piles occurred when the pile spacing was $3d$ or smaller. With increased spacing, the shear failure became less obvious. At a spacing of $6d$, there was no obvious shear failure and the piles behaved more like isolated single piles.

For comparison, the results from this study and those of Adachi et al. (1994), Chen and Poulos (1997) and Llyas et al. (2004) are shown in Table 5.2 and Figure 5.6. For this study, the group factor F_{pi} was calculated using Equation (5-1). For Adachi et al. (1994) and Llyas et al. (2004), the F_{pi} was based on the measured load. For Chen and Poulos (1997), the F_{pi} was based on the measured bending moment. The present results showed that the group factor of the “front” pile was always larger than that of the “back” pile in a group. For the “front” pile, the F_{pi} slightly increased with increased pile spacing, and all values of F_{pi} were close to 1.0 (with maximum difference of 13%). For the “back” pile, the F_{pi} was 0.48 when the pile spacing was $3d$. With increased pile spacing, the F_{pi} increased. Even at a pile spacing of $6d$, the F_{pi} was 0.76. It indicated that the group effects still existed when the pile spacing was $6d$ for a two-pile group in a line.

The experimental results from Adachi et al. (1994), Chen and Poulos (1997) and Llyas et al. (2004) also showed that the group factor of the “front” pile was larger than that of the “back” pile. Furthermore, Adachi et al. (1994) also reported that all values of F_{pi} of the “front” pile were close to 1.0 (with maximum difference of 6%). There was a significant reduction in F_{pi} of the “back” pile in comparison to a single pile. The F_{pi} of the “back” pile increased with increased pile spacing. However, the group factors presented by Chen and Poulos (1997) in Table 5.2 differed

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significantly and were generally larger than 1.0. This could be because: a) their conventional 1-g tests were carried out on calcareous sand; b) their F_{pi} was based on the indirect measurement of the bending moment; and c) their piles were subjected to a much higher lateral soil movement of $y/d = 2.4$, while in this study it was 1.1. Another factor could be the different pile head fixity conditions: free head for Chen and Poulos (1997) and head restrained from movement in this study.

The differences in results between this study for piles in clay and the results from Adachi et al. (1994) and Chen and Poulos (1997) shown in Table 5.2 and Figure 5.6 could possibly also be due to the scale and stress effects of the conventional 1-g tests in sand carried out by Adachi et al. (1994) and Chen and Poulos (1997).

Figure 5.3 Normalised p - y curves for Test G-3 ($S_v = 3d$)

Photo 5.3 Failure pattern for Test G-3 at $y/d = 1.1$

Figure 5.4 Normalised p - y curves for Test G-4 ($S_v = 4.5d$)

Photo 5.4 Failure pattern for Test G-4 at $y/d = 1.1$

Figure 5.5 Normalised p - y curves for Test G-5 ($S_v = 6d$)

Photo 5.5 Failure pattern for Test G-5 at $y/d = 1.1$

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Table 5.2 Group factors for two piles in a line

Reference	Soil type	Pile type and fixity conditions	d (mm)	Spacing	Pile position	F_{pi}
Present	Clay	Passive, head and tip pinned	16	$3d$	front	0.87
					back	0.48
				$4.5d$	front	0.88
					back	0.64
				$6d$	front	0.91
					back	0.76
Adachi et al. (1994)	Dense sand	Active, free head	15	$2d$	front	0.97
					back	0.6
				$2.5d$	front	1.0
					back	0.72
				$3d$	front	0.94
					back	0.7
				$4d$	front	1.0
					back	0.8
Chen and Poulos (1997)	Sand	Passive, free head	25	$2.5d$	front	1.31
					back	1.01
				$5.0d$	front	1.59
					back	1.1
				$7.5d$	front	1.2
					back	0.69
Llyas et al.* (2004)	Clay	Active, capped-head	12	$3d$	front	0.80
					back	0.63

* For centrifuge tests.

Figure 5.6 Effects of pile spacing on F_{pi} for two piles in a line

5.4 Three-Pile Group Tests

The next set of tests involved three piles in a row and three piles in a line with a spacing of $3d$ (Figure 3.11(a), (b)). In addition, tests were also carried out on two triangular arrangements of three piles in a group, as illustrated schematically in Figure 3.11(c), (d), since many pile groups are of such arrangements in practice.

5.4.1 Three Piles in a Row (G-6)

Test G-6 was carried out on three piles in a row with the spacing of $3d$, as illustrated schematically in Figure 3.11(a). Figure 5.7 shows the normalised p - y curves for Test G-6. The curves of the “side” pile and the “centre” pile were quite similar in shape. The normalised ultimate soil pressure p_u/c_u of the “side” pile was 7.7 ($y/d = 0.68$),

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while the p_u/c_u of the “centre” pile was 7.5 ($y/d = 0.76$). These values were smaller than the $p_u/c_u = 8.8$ obtained from Test G-1 (two piles in a row with $3d$ spacing).

Photo 5.6 shows the soil surface failure pattern for Test G-6 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of each pile and obvious cracks perpendicular to the direction of soil movement on the sides of the piles. The cracks between the three piles intersected and were connected together. Based on the deformations of the grid lines, a larger arc-shaped affected zone in front of the piles was observed in comparison to Test G-1 (Photo 5.1).

For comparison, the results from this study and those of Chen and Poulos (1997) are shown in Table 5.3. For this study, the group factor F_{pi} was calculated using Equation (5-1). For Chen and Poulos (1997), the F_{pi} was based on the measured bending moment. The present study showed significant reductions in the F_{pi} (approximately 30%) in comparison to a single pile. The reductions of F_{pi} from Chen and Poulos (1997) were 15 ~ 24%. For this study, there were few differences in the group factors for the “side” and “centre” piles. This may be because the heads of all three piles in a row were attached to a rectangular strip and restrained from movement (referred to Photo 3.12), and the pile tips were similarly restrained.

Table 5.3 Group factors for three piles in a row

Reference	Soil type	Pile type and fixity conditions	d (mm)	Spacing	Pile position	F_{pi}
Present	Clay	Passive, head and tip pinned	16	$3d$	side	0.69
					centre	0.68
Chen and Poulos (1997)	Sand	Passive, free head	25	$2.5d$	side	0.85
					centre	0.76

Figure 5.7 Normalised p - y curves for Test G-6 ($S_h = 3d$)

Photo 5.6 Failure pattern for Test G-6 at $y/d = 1.1$

5.4.2 Three Piles in a Line (G-7)

Test G-7 was carried out on three piles in a line with the spacing of $3d$, as illustrated schematically in Figure 3.11(b). Figure 5.8 shows the normalised p - y curves for Test G-7. The three curves were almost identical when p/c_u was smaller than 3.5. However, for the “front” pile, the p - y curve had an upward trend with increased soil displacement, similar to that of the single pile test S-3. The normalised ultimate soil pressure p_u/c_u was 11.1 ($y/d = 0.94$). For the “middle” pile, the p_u/c_u was 5.0 ($y/d = 0.24$), and thereafter the p/c_u gradually reduced with increased soil displacement. For the “back” pile, there were some slight fluctuations when y/d was close to 0.2. The normalised ultimate soil pressure occurred at $p_u/c_u = 5.8$ ($y/d = 0.54$), and thereafter the p/c_u gradually reduced with increased soil displacement.

Photo 5.7 shows the soil surface failure pattern for Test G-7 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of the “back” pile and obvious cracks at an angle of 60° with respect to the direction of soil movement on both sides of the “back” pile. There was only a little gap (void) at the back of the “middle” and “front” piles. Obvious shear failure between the three piles was observed in the sheared zone, similar to Test G-3 which comprised of two piles in a line with a spacing of $3d$ (shown in Photo 5.3). An arc-shaped affected zone at the front of the “front” pile was also observed, which was similar to that of the single pile test S-3 (shown in Photo 4.3).

For comparison, the results from this study and those of Cox et al. (1984) and Chen and Poulos (1997) are shown in Table 5.4. For this study, the group factor F_{pi} was calculated using Equation (5-1). For Cox et al. (1984) and Chen and Poulos (1997), the F_{pi} was based on the measured load and bending moment, respectively. The present results showed that the F_{pi} of the “front” pile was much larger than those of the “middle” and “back” piles in a group. The F_{pi} of the “front” pile was 1.0. In comparison to a single pile, the F_{pi} of the “middle” and “back” piles reduced significantly (about 50%) with slight differences in the F_{pi} of the “middle” and “back” piles.

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The group factor of the “front” pile from the present study was similar to those of Cox et al. (1984) and Chen and Poulos (1997). Although a similar trend of lower group factors of the “middle” and “back” piles were also obtained by Cox et al. (1984) and Chen and Poulos (1997), the group factors from this study were significantly lower, possibly because of the different pile head and pile tip fixity conditions.

Table 5.4 Group factors for three piles in a line

Reference	Soil type	Pile type and fixity conditions	d (mm)	Spacing	Pile position	F_{pi}
Present	Soft clay	Passive, head and tip pinned	16	$3d$	front	1.00
					middle	0.45
					back	0.52
Cox et al. (1984)	Very soft clay	Active, head fixed	25.4	$3d$	front	0.93
					middle	0.73
					back	0.75
Chen and Poulos (1997)	Sand	Passive, free head	25	$2.5d$	front	1.10
					middle	0.70
					back	0.95

Figure 5.8 Normalised p - y curves for Test G-7 ($S_v = 3d$)

Photo 5.7 Failure pattern for Test G-7 at $y/d = 1.1$

5.4.3 Three Piles in Triangular Arrangements (G-8 and G-9)

In practical situations, it is common to find three piles grouped in a triangular arrangement. Test G-8 (illustrated schematically in Figure 3.11(c)) was carried out on three piles in a triangular arrangement, where two piles were in the front row and one pile was in the back row. Test G-9 (illustrated schematically in Figure 3.11(d)) was carried out on three piles in a reversed triangular arrangement, where one pile was in the front row and two piles were in the back row. For both tests, the pile spacings were the same with $S_h = S_v = 3d$.

Figure 5.9 shows the normalised p - y curves for Test G-8. The two curves were almost identical when p/c_u was smaller than 4. However, the p - y curve of the “front” pile had an upward trend with increased soil displacement. The normalised ultimate soil pressure p_u/c_u of the “front” pile was 10.1 ($y/d = 0.92$). For the “back” pile, some slight fluctuations occurred when y/d was close to 0.2 and the p_u/c_u was 5.3 ($y/d = 0.33$). The p/c_u reduced gradually with increased soil displacement when y/d was larger than 0.4.

Figure 5.10 shows the normalised p - y curves for Test G-9. The p - y curve of the “front” pile had an upward trend with increased displacement. The normalised ultimate soil pressure p_u/c_u was 9.4 ($y/d = 0.86$). For the “back” pile, some slight fluctuations occurred when y/d was close to 0.2 and the p_u/c_u was 6.7 ($y/d = 0.72$).

Photo 5.8 shows the soil surface failure pattern for Test G-8 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of the “back” pile and long cracks at an angle of 60° with respect to the direction of soil movement on both sides of the “back” pile. A crushed unsymmetrical semicircular gap (void) was observed at the back of each “front” pile. There was an obvious triangular shaped shear failure between the “front” pile and the “back” piles in the sheared zone. Compared with the affected zone of the single pile test S-3 (shown in Photo 4.3), a much larger arc-shaped affected zone was observed at the front of the “front” piles.

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Photo 5.9 shows the soil surface failure pattern for Test G-9 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of each “back” pile and obvious cracks perpendicular to the direction of soil movement on both sides of the “back” pile. The cracks between the two “back” piles intersected and were connected together. There was a smaller semicircular gap (void) of approximately $0.7d$ at the back of the “front” pile. No obvious shear failure between the “front” and “back” piles was observed, which was different from that of Test G-8 (shown in Photo 5.8).

The group factors F_{pi} for Test G-8 and G-9, which were calculated using Equation (5-1), are listed in Table 5.5. For both tests, the F_{pi} of the “back” pile was significantly less than that of the “front” pile.

Table 5.5 Group factors for three piles in triangular arrangements

Testing Number	Spacing	Pile position	F_{pi}
G-8	$S_h = S_v = 3d$	“front”	0.91
		“back”	0.48
G-9	$S_h = S_v = 3d$	“front”	0.85
		“back”	0.60

Ng et al. (2001) carried out an active pile test of a three-pile group embedded in superficial deposits and decomposed rocks. The arrangement of the three-pile group was the same as that of Test G-8. For comparison, the results of Test G-8 and those of Ng et al. (2001) are shown in Table 5.6. For this study, the group factor F_p for the whole group was calculated using Equation (5-4). For Ng et al. (2001), the F_p was based on the measured load. It was found the F_p of this study was almost the same as that of Ng et al. (2001). They all indicated there were significant reductions in the F_p (approximately 25%) in comparison to a single pile.

Figure 5.9 Normalised p - y curves for Test G-8 ($S_h = S_v = 3d$)

Photo 5.8 Failure pattern for Test G-8 at $y/d = 1.1$

Figure 5.10 Normalised p - y curves for Test G-9 ($S_h = S_v = 3d$)

Photo 5.9 Failure pattern for Test G-9 at $y/d = 1.1$

Table 5.6 Comparison of group factors for three-pile groups

Reference	Soil type	Pile type and fixity conditions	d (mm)	Spacing ($S_h = S_v$)	F_p
Present (G-8)	Soft clay	Passive, head and tip pinned	16	$3d$	0.76
Ng et al. (2001)	Mixture*	Active, head fixed	1500	$3d$	0.75

*A mixture of sand, silt and clay

5.5 Four-Pile Group Tests (G-10 and G-11)

Test G-10 and G-11 were carried out on four piles in a square arrangement with $S_h = S_v = 3d$ and $S_h = S_v = 6d$, respectively, as illustrated schematically in Figure 3.12 (a) and (b).

Figure 5.11 shows the normalised p - y curves for Test G-10. The shapes of the p - y curves were similar to those of Test G-3 (shown in Figure 5.3), which comprised of two piles in a line with a spacing of $3d$. The curves of the “front” and “back” piles were almost identical when p/c_u was smaller than 3. However, the p - y curve of the “front” pile had an upward trend with increased soil displacement, similar to that of the single pile test S-3. The normalised ultimate soil pressure p_u/c_u of the “front” pile was 9.9 ($y/d = 0.85$). The p - y curve of the “back” pile differed significantly from that of the “front” pile with increased soil displacement. The p_u/c_u of the “back” pile was 4.6 ($y/d = 0.16$). Some slight fluctuations occurred when y/d was close to 0.2. The p/c_u reduced gradually with increase soil displacement when y/d was larger than 0.45.

Figure 5.12 shows the normalised p - y curves for Test G-11. The shapes of the p - y curves were similar to those of Test G-5 (shown in Figure 5.5), which comprised of two piles in a line with a spacing of $6d$. With increased soil displacement, the p - y curve of the “front” pile closely resembled that of the single pile test S-3. The

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normalised ultimate soil pressure p_u/c_u was 10.9 ($y/d = 0.79$). For the “back” pile, the p - y curve had an upward trend with increased soil displacement and the p_u/c_u was 7.5 ($y/d = 0.72$).

Photo 5.10 shows the soil surface failure pattern for Test G-10 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of each “back” pile and a crack at an angle of 45° with respect to the direction of soil movement on the outer side of each “back” pile. There was only a little gap (void) at the back of each “front” pile and no obvious cracks on the sides of each “front” pile. The cracks between two “back” piles intersected and were connected together. Obvious shear failure between the “front” and “back” piles was observed in the sheared zone. Compared with the affected zone of the single pile test S-3 (shown in Photo 4.3), a much larger arc-shaped affected zone was observed at the front of the “front” piles.

Photo 5.11 shows the soil surface failure pattern for Test G-11 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of each “back” pile and obvious cracks nearly perpendicular to the direction of soil movement on both sides of each “back” pile. A long crack between the two “back” piles was also observed. There was a similar semicircular gap (void) at the back of the “front” pile and no obvious cracks on both sides of the “front” pile. No sheared zone was observed between the “front” and “back” piles. Based on the observed deformations of the grid lines, there were two individual arc-shaped affected zones at the front of the “front” piles and the “back” piles, respectively.

For comparison, the results from this study and those of Chen and Poulos (1997) and Llyas et al. (2004) are shown in Table 5.7 and Figure 5.13. For this study, the group factor F_{pi} was calculated using Equation (5-1). For Chen and Poulos (1997) and Llyas et al. (2004), the F_{pi} was based on the measured bending moment and load, respectively. The present results showed that the F_{pi} of the “front” pile was larger than that of the “back” pile in a group. The F_{pi} of the “front” pile was 0.89 when $S_h = S_v = 3d$, and was 0.98 when $S_h = S_v = 6d$. It indicated that the “front” piles in a 2×2 group behaved more like isolated single piles with increased pile spacing. The F_{pi} of the “back” pile was 0.41 when $S_h = S_v = 3d$. With increased pile spacing,

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the F_{pi} of the “back” pile increased. Even at $S_h = S_v = 6d$, the F_{pi} was 0.68. It indicated that the group effects still existed at $S_h = S_v = 6d$ for a 2×2 group.

The experimental results from Chen and Poulos (1997) and Llyas et al. (2004) also showed that the group factors of the “front” piles were larger than that of the “back” piles. However, the group factors presented by Chen and Poulos (1997) in Table 5.7 were generally close to or larger than 1.0. As mentioned in 5.3.2, this could be because: a) their tests were carried out on calcareous sand; b) their F_{pi} was based on the indirect measurement of the bending moment; and c) their piles were subjected to a much higher lateral soil movement of $y/d = 2.4$, while in this study it was 1.1. Another factor could be the different pile head fixity conditions: free head for Chen and Poulos (1997) and head restrained from movement in this study.

Figure 5.11 Normalised p - y curves for Test G-10 ($S_h = S_v = 3d$)

Photo 5.10 Failure pattern for Test G-10 at $y/d = 1.1$

Figure 5.12 Normalised p - y curves for Test G-11 ($S_h = S_v = 6d$)

Photo 5.11 Failure pattern for Test G-11 at $y/d = 1.1$

Table 5.7 Group factors for four-pile groups

Reference	Soil type	Pile type and fixity conditions	d (mm)	Spacing ($S_h = S_v$)	Pile position	F_{pi}
Present	Clay	Passive, head and tip pinned	16	$3d$	front	0.89
					back	0.41
				$6d$	front	0.98
					back	0.68
Chen and Poulos (1997)	Sand	Passive, free head	25	$2.5d$	front	1.08
					back	0.96
				$5d$	front	1.36
					back	1.03
Llyas et al. (2004)	Clay	Active, capped-head	12	$3d$	front	0.96
					back	0.78

Figure 5.13 Effects of pile spacing on F_{pi} for 4-pile groups

5.6 Five-Pile Group Test (G-12)

Test G-12 was carried out on five piles in a group, where four piles were in a rectangular arrangement and one pile was at the centre of the rectangle, as illustrated schematically in Figure 3.13. The S_h and S_v were $3d$.

Figure 5.14 shows the normalised p - y curves for Test G-12. All curves were almost identical when p/c_u is smaller than 2. However, with increased soil displacement, the p - y curve of the “front” pile closely resembled to that of the single pile test S-3. The normalised ultimate soil pressure p_u/c_u of the “front” piles was 10.6 ($y/d = 0.80$). For the “middle” pile, some fluctuations occurred when y/d was close to 0.2, and the p_u/c_u was 5.3 ($y/d = 0.4$). For the “back” pile, the p_u/c_u was 4.5 ($y/d = 0.23$), and the p/c_u reduced gradually with increase soil displacement when y/d was larger than 0.5.

Photo 5.12 shows the soil surface failure pattern for Test G-12 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of each “back” pile. There were obvious and long cracks at an angle of 70° and several short cracks at an angle of 25° with respect to the direction of soil movement on the outer side of each “back” pile. The cracks between the two “back” piles intersected and were connected together. There was a little gap (void) at the back of the “middle” pile and “front” piles. No obvious cracks were observed on both sides of the “middle” and “front” piles. There was a triangular shaped shear failure between the “front” and “middle” piles in the sheared zone. Compared with the affected zone of the single pile test S-3 (shown in Photo 4.3), a much larger arc-shaped affected zone was observed at the front of the “front” piles.

The group factors F_{pi} for Test G-12, which were calculated using Equation (5-1), are listed in Table 5.8. The results showed that the F_{pi} of the “front” pile was much larger than that of the “middle” and “back” piles in a five-pile group. The F_{pi} of the “front” pile was 0.95. In comparison to a single pile, the F_{pi} of the “middle” and “back” piles reduced significantly (about 55%) with slight differences in the F_{pi} of the “middle” and “back” piles.

Figure 5.14 Normalised p - y curves for Test G-12 ($S_h = S_v = 3d$)

Photo 5.12 Failure pattern for Test G-12 at $y/d = 1.1$

Table 5.8 Group factors for the five-pile group

Testing Number	Spacing	Pile position	F_{pi}
G-12	$S_h = S_v = 3d$	“front”	0.95
		“middle”	0.48
		“back”	0.41

5.7 Six-Pile Group Tests

In practical situations, there are two common arrangements for a six-pile group. Test G-13 (illustrated schematically in Figure 3.14 (a)) was carried out on a 2×3 group, i.e. six piles in two rows of three piles. Test G-14 (illustrated schematically in Figure 3.14 (b)) was carried out on a 3×2 group, i.e. six piles in three rows of two piles. The S_v and S_h were $3d$.

5.7.1 Six Piles in Two Rows in a Group (G-13)

Figure 5.15 shows the normalised p - y curves for Test G-13. All curves were almost identical when p/c_u was smaller than 2. However, the p - y curves of the “front” row piles had an upward trend with increased soil displacement, similar to that of the single pile test S-3. The normalised ultimate soil pressure p_u/c_u of the “front-side” piles was 9.6 ($y/d = 0.77$), and the p_u/c_u of the “front-centre” pile was 9.0 ($y/d = 0.74$). The p - y curves of the “back” row piles differed significantly from those of the “front” row piles. The p_u/c_u of the “back-side” pile was 4.5 ($y/d = 0.2$), and the p_u/c_u of the “back-centre” piles was 4.3 ($y/d = 0.34$). The p/c_u of the “back” row piles reduced gradually with increased soil displacement when y/d was larger than 0.5.

Photo 5.13 shows the soil surface failure pattern for Test G-13 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of each “back” row pile and obvious cracks at an angle of 45° with respect to the direction of soil movement on the outer sides of the “back-side” piles. The cracks between the piles in the back row intersected and were connected together. There was a little gap (void) at the back

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of each front row pile. Shear failure between the “front” and “back” piles in a line was observed clearly. Compared with the affected zone of the single pile test S-3 (shown in Photo 4.3), a much larger arc-shaped affected zone was observed at the front of the “front” piles.

For comparison, the results from this study and those from “active” pile field tests by Meimon et al. (1986) are shown in Table 5.9. For this study, the group factor F_{pri} for a row of piles was calculated using Equation (5-3). For Meimon et al. (1986), the group factor was based on the measured load. The present results showed that the F_{pri} of the “front” row piles was much larger than that of the “back” row piles. The F_{pri} of the “front” row piles was 0.84, and the F_{pri} of the “back” row piles was 0.4. The “active” pile results of Meimon et al. (1986) also showed that the load distribution ratio of the “front” row piles was much larger than that of the “back” row piles.

Table 5.9 Group factors for six piles in two rows in a group ($S_h = S_v = 3d$)

Reference	Test type and pile type	Soil type	d (mm)	Pile position	F_{pri}
Present tests	Laboratory, passive pile	Soft clay	16	“front” row	0.84
				“back” row	0.4
Meimon et al. (1986)	Field, active pile	Silt	284 × 270 (H-pile)	“front” row	0.9
				“back” row	0.5

Figure 5.15 Normalised p - y curves for Test G-13 ($S_h = S_v = 3d$)

Photo 5.13 Failure pattern for Test G-13 at $y/d = 1.1$

5.7.2 Six Piles in Three Rows in a Group (G-14)

Figure 5.16 shows the normalised p - y curves for Test G-14. All the curves were almost identical when p/c_u was smaller than 2.5. However, the p - y curve of the “front” pile had an upward trend with increased soil displacement, similar to that of the single pile test S-3. The normalised ultimate soil pressure p_u/c_u was 9.0 ($y/d = 0.77$). The p - y curves of the “middle” and “back” piles differed significantly from that of the “front” pile. The p_u/c_u of the “middle” pile was 4.2 ($y/d = 0.28$), and the p_u/c_u of the “back” pile was 3.9 ($y/d = 0.17$).

Photo 5.14 shows the soil surface failure pattern for Test G-14 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of each “back” pile. Obvious cracks between two piles in the “back” row intersected and were connected together. There was only a little gap (void) at the back of each “middle” and “front” row pile. Obvious shear failure between three piles in a line was also observed in the sheared zone, similar to Test G-7 which comprised of three piles in a line with a spacing of $3d$ (shown in Photo 5.7). Compared with the affected zone of the single pile test S-3 (shown in Photo 4.3), a much larger arc-shaped affected zone was observed at the front of the “front” piles.

The group factors F_{pri} for Test G-14, which were calculated using Equation (5-3), are listed in Table 5.10. The results showed that the F_{pri} of the “front” row piles was much larger than that of the “middle” and “back” row piles. The F_{pri} of the “front” row piles was 0.81. In comparison to a single pile, the F_{pri} of the “middle” and “back” row piles reduced significantly (about 65%) with slight differences in the F_{pri} of the “middle” and “back” row piles.

Table 5.10 Group factors for six piles in three rows in a group

Testing Number	Spacing	Pile position	F_{pri}
G-14	$S_h = S_v = 3d$	“front” row	0.81
		“middle” row	0.38
		“back” row	0.35

Figure 5.16 Normalised p - y curves for Test G-14 ($S_h = S_v = 3d$)

Photo 5.14 Failure pattern for Test G-14 at $y/d = 1.1$

5.8 Nine-Pile Group Test (G-15)

In practical situations, it is also common to find nine piles in a group. Test G-15 was carried out on a 3×3 group, i.e. nine piles in three rows of three piles, as illustrated schematically in Figure 3.15. The S_h and S_v were $3d$.

Figure 5.17 shows the normalised p - y curves for Test G-15. The p - y curves of the “front” row piles had an upward trend with increased soil displacement, similar to that of the single pile test S-3. The normalised ultimate soil pressure p_u/c_u of the “front-side” pile was 8.8 ($y/d = 0.82$), and the p_u/c_u of the “front-centre” pile was 8.4 ($y/d = 0.86$). The p - y curves of the “middle” row and “back” row piles differed significantly from those of the “front” row piles. For the “middle-side” pile, the p_u/c_u was 4.2 ($y/d = 0.24$), and thereafter the p/c_u was almost constant with increased soil displacement. For the “middle-centre” pile, the p_u/c_u of the “middle-centre” pile was 4.0 ($y/d = 0.59$), and thereafter the p/c_u was almost constant with increased soil displacement. For the “back-side” pile, the p_u/c_u was 3.9 ($y/d = 0.15$), and then the p/c_u reduced gradually with increased soil displacement. For the “back-centre” pile, the p_u/c_u was 3.1 ($y/d = 0.11$), and then some slight fluctuations occurred.

Photo 5.15 shows the soil surface failure pattern for Test G-15 ($y/d = 1.1$). There was a semicircular gap (void) of approximately $1.1d$ at the back of each “back” row pile and obvious cracks at an angle of 45° with respect to the direction of soil movement on the outer side of each “back-side” pile. The cracks between the piles in the “back” row intersected and were connected together. There was only a little gap (void) at the back of each “middle” or “front” row pile. Obvious shear failure between three piles in a line was observed in the sheared zone, similar to Test G-7 which comprised three piles in a line with a spacing of $3d$ (shown in Photo 5.7). Compared with the affected zone of the single pile test S-3 (shown in Photo 4.3), a much larger arc-shaped affected zone was observed at the front of the “front” piles.

For comparison, the results from this study and the “active” pile results from Brown et al. (1987, 1988), Rollins et al. (1998) and Llyas et al. (2004) are shown in Table 5.11. The group factor F_{pri} for a row of piles was calculated using Equation (5-3) for

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this study. For Brown et al. (1987, 1988), Rollins et al. (1998) and Llyas et al. (2004), the F_{pri} was based on the measured load. The present results showed that the F_{pri} of the “front” row piles was much larger than that of the “middle” row and “back” row piles in a 3×3 group. The F_{pri} of the “front” row piles was 0.78. In comparison to the $F_{pi} = 1.0$ for a single pile, the F_{pri} of the “middle” row and “back” row piles reduced significantly (about 65%) with slight differences in the F_{pri} of the “middle” row and “back” row piles. The active pile results of Brown et al. (1987, 1988), Rollins et al. (1998) and Llyas et al. (2004) also indicated that the load distribution ratio of the “front” row piles was larger than that of the “middle” row and “back” row piles.

Table 5.11 Group factors for nine-pile groups ($S_h = S_v = 3d$)

Reference	Test type and pile type	Soil type	d (mm)	F_{pri} (row)		
				Front	Middle	Back
Present	Laboratory, passive pile	Soft clay	16	0.78	0.39	0.33
Brown et al. (1987)	Field active pile	Stiff clay	272	0.7	0.6	0.5
				0.7	0.5	0.4
Brown et al. (1988)	Field, active pile	Sand	272	0.8	0.4	0.3
Rollins et al. (1998)	Field Active pile	Layer silt	324	0.6	0.4	0.4
Llyas et al. (2004)	Centrifuge, active pile	Kaolin clay	12	0.65	0.50	0.48

Figure 5.17 Normalised p - y curves for Test G-15 ($S_h = S_v = 3d$)

Photo 5.15 Failure pattern for Test G-15 at $y/d = 1.1$

5.9 Discussions

The p_u/c_u , the y_u/d , the group factors F_{pi} and F_{yi} for individual piles in a group as well as the group factors F_p for a whole group from model pile group tests are summarised in Table 5.12. Figures 5.18 and 5.19 show the group factors F_{pi} and F_p for pile groups with $S_h = S_v = 3d$ and $S_h = S_v = 6d$, respectively.

For pile group tests, the p_u acting on an individual pile within a group was different from a single pile. The group factors F_{pi} , in terms of the p_u for individual piles in a group, were generally smaller than 1.0. The group factors F_{pi} of the “front” row piles were generally close to 1.0, and always much larger than those of the “middle” row or “back” row piles. The differences in the F_{pi} between the “middle” row piles and the “back” row piles were less significant, and so were the differences in the F_{pi} between the “side” piles and the “centre” pile in the same row. Considering the group factors F_{yi} , in terms of the ultimate soil displacement to mobilise the p_u for individual piles in a group, Table 5.12 indicates that the F_{yi} for the “front” row piles was also generally close to 1.0 and always larger than that of the “middle” row or “back” row piles.

Table 5.13 summarises the group behaviour of the individual pile in the different pile group configurations. The following are the main observations:

- (a) Because of the effect of the centre pile in a 3-pile-in-a-row group with close spacing of $3d$, the F_{pi} of the side pile in the 2-pile group was lower than that in the 3-pile group. Similar results have been found by Chen and Poulos (1997) in their model tests.
- (b) In the 4-pile group, the back row piles may be “supporting” the front row piles so that the F_{pi} is higher than that of the 2-pile group. However, with the increased number of rows, the “supporting” effect is reduced so that the F_{pi} of the 2×3 group is slightly lower than that of the 4-pile group.
- (c) In the 3×2 group, the back row piles may be “supporting” the front row piles so that the F_{pi} is higher than that of the 3-pile group. However, with the increased number of rows, the “supporting” effect is reduced so that the F_{pi} of the 3×3 group is slightly lower than that of the 3×2 group.

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Table 5.12 Summary of pile group test results

Pile group configuration	Testing Number	Spacing	Pile Num	p_u/c_u	y_u/d	F_{yi}	F_{pi}	F_p
G-1	$S_h = 3d$	1	8.8	0.75	0.96	0.79	0.79
	G-2	$S_h = 6d$	1	10.8	0.8	1.03	0.97	0.97
G-3	$S_v = 3d$	1	9.7	0.75	0.96	0.87	0.68
			2	5.3	0.4	0.51	0.48	
	G-4	$S_v = 4.5d$	1	9.8	0.77	0.99	0.88	0.76
			2	7.2	0.48	0.61	0.64	
	G-5	$S_v = 6d$	1	10.1	0.76	0.97	0.91	0.84
			2	8.4	0.75	0.96	0.76
G-6	$S_h = 3d$	1	7.7	0.68	0.87	0.69	0.69
			2	7.5	0.76	0.97	0.68
G-7	$S_v = 3d$	1	11.1	0.94	1.21	1.00	0.66
			2	5.0	0.24	0.31	0.45	
			3	5.8	0.54	0.69	0.52
G-8	$S_h = 3d$ $S_v = 3d$	1	10.1	0.92	1.18	0.91	0.76
			2	5.3	0.33	0.42	0.48
G-9	$S_h = 3d$ $S_v = 3d$	1	9.4	0.86	1.10	0.85	0.68
			2	6.7	0.72	0.92	0.60	

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Table 5.12 (continued)

G-10	$S_h = 3d$ $S_v = 3d$	1	9.9	0.85	1.09	0.89	0.65
	G-11	$S_h = 6d$ $S_v = 6d$	2	4.6	0.16	0.21	0.41	
			1	10.9	0.79	1.01	0.98	0.83
			2	7.5	0.72	0.92	0.68
G-12	$S_h = 3d$ $S_v = 3d$	1	10.6	0.80	1.03	0.95	0.64
			2	5.3	0.40	0.51	0.48	
			3	4.5	0.23	0.29	0.41
G-13	$S_h = 3d$ $S_v = 3d$	1	9.6	0.77	0.99	0.86	0.62
			2	9.0	0.74	0.95	0.81	
			3	4.5	0.20	0.26	0.41	
			4	4.3	0.34	0.44	0.39
G-14	$S_h = 3d$ $S_v = 3d$	1	9.0	0.77	0.99	0.81	0.51
			2	4.2	0.28	0.36	0.38	
			3	3.9	0.17	0.22	0.35
G-15	$S_h = 3d$ $S_v = 3d$	1	8.8	0.82	1.05	0.79	0.49
			2	8.4	0.86	1.10	0.76	
			3	4.2	0.24	0.31	0.38	
			4	4.0	0.59	0.76	0.36	
			5	3.9	0.15	0.19	0.35	
			6	3.1	0.11	0.14	0.28	

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Figure 5.18 Group factors for pile groups with $S_h = S_v = 3d$
(F_p for a whole group in brackets)

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Figure 5.19 Group factors for pile groups with $S_h = S_v = 6d$
 (F_p for a whole group in brackets)

Table 5.13 Comparison of individual pile in a pile group

Set number	Test number	Pile position	F_{pi}
(a)	G-1 (two piles in a row)	side	0.79
	G-6 (three piles in a row)		0.69
(b)	G-1 (two piles in a row)	front	0.79
	G-10 (four piles 2×2)		0.89
	G-14 (six piles 2×3)		0.81
(c)	G-6 (three piles in a row)	front-side	0.69
	G-13 (six piles 3×2)		0.86
	G-15 (nine piles 3×3)		0.79

5.9.1 Effect of Number of Piles

Figure 5.20 shows the group factor F_p , in terms of the ultimate soil pressures for the whole group, plotted with number of piles, n . Only pile groups with spacings of $3d$ were considered. There was an obvious trend that the F_p reduced with an increased number of piles.

Figure 5.20 Effects of n on F_p

5.9.2 Effect of Pile Spacing

As shown in Table 5.12 and Figures 5.18 and 5.19, the pile spacing was a key parameter influencing the ultimate soil pressure acting on pile groups subjected to lateral soil movements. Generally, the group factors F_{pi} or F_p increased with increased pile spacing. For two piles in a row with a spacing of $6d$, the F_p was 0.97. It indicated that the piles for a two-pile group in a row behaved like isolated single piles when the pile spacing was $6d$ or larger.

The results indicating that the group factors generally increased with increasing pile spacing were consistent with the concept proposed by Focht and Koch (1973) and Ooi and Duncan (1994) that the behaviour of a group of piles consisted of two

components: a) a component due to the nonlinear soil behaviour occurring close to the individual piles, and b) a component due to pile-soil-pile interaction through the less highly stressed soil further from the piles. When the pile group spacing was small, the second component significantly affected the pile behaviour.

From Photos 5.3, 5.7, 5.10, 5.13, 5.14 and 5.15, it was found that the shear failure between piles in a line occurred when the pile spacing was equal to $3d$. This may have caused significant reductions of p_u for the “middle” or “back” row piles in clay. Fleming et al. (1985) suggested that as the pile group spacing was reduced, failure would occur with a slip plane linking the piles in a line, as shown in Figure 5.21. The ultimate soil pressure p_u on the “back” pile would be $2 \cdot S_v \cdot \tau / d$, in which S_v is the pile spacing and τ is the friction on the side of the block of the soil between the two piles. For two piles in a line with a spacing of $3d$, the present results of p_u on the “back” pile was $5.3c_u$, which was close to the prediction value of $6c_u$ given by Fleming et al. (1985). It indicated that the shear failure between piles in a line would govern when the shearing resistance of the soil between the piles was less than the ultimate resistance of an isolated pile.

Figure 5.21 Block failure under lateral load as predicted by Fleming et al (1985)

5.9.3 Effect of Pile Arrangement

Comparing two piles in a row with two piles in a line, or three piles in a row with three piles in a line or three piles in triangular arrangements in Table 5.12, there was a 10% discrepancy of the group factors F_p for a pile group with the same pile number but different arrangements. It indicated that the pile arrangement still had some effects on the ultimate soil pressure acting on the piles.

5.10 Summary

A series of laboratory tests have been carried out on pile groups subjected to lateral soil movements. Different numbers (2~9) of piles in a group with various configurations and different pile-to-pile spacings have been considered. The group factor F_{pi} , F_{yi} , and F_p , which were calculated using Equations (5-1), (5-2) and (5-4), respectively, are summarised in Table 5.12 and Figure 5.18 and 5.19. For comparison, the results from this study and those from the literature are shown in Tables 5.1, 5.2, 5.3, 5.4, 5.6, 5.7, 5.9, and 5.11. Considering the variation in soil properties in various field and laboratory studies, the group factors obtained in this study were reasonably consistent with those obtained from the literature. Other main conclusions from the experimental results are summaries below.

In general

1. The p_u acting on an individual pile within a group was different from a single pile. The group factors F_{pi} , in terms of the p_u for individual piles in a group, were generally smaller than 1.0.
2. The group factors F_{pi} of the “front” row piles were generally slightly less than 1.0, and always much larger than those of the “middle” row or “back” row piles.
3. The differences in the F_{pi} between the “middle” row piles and the “back” row piles were less significant, and so were the differences in the F_{pi} between the “side” piles and the “centre” pile in the same row.
4. The group factors F_{yi} of the “front” row piles were also generally close to 1.0 and always larger than those of the “middle” row or “back” row piles.

Effect of pile number, spacing and arrangement

1. There was an obvious trend that the group factors F_p , in terms of the ultimate soil pressures for the whole group, reduced with an increased number of piles.
2. Pile spacing was a key parameter governing the ultimate soil pressure acting on piles subjected to lateral soil movements. Generally, the group factors F_{pi} and F_p increased with increased pile spacing.
3. For two piles in a row with the pile spacing of $6d$, the F_p was 0.97. It indicated that the piles for a two-pile group in a row behaved like isolated single piles when the pile spacing was $6d$ or larger.
4. For pile groups with a spacing of $3d$ or smaller in clay, shear failure between piles in a line occurred. This may have caused significant reduction of p_u and F_{pi} for the “middle” or “back” row piles.
5. For a pile group with the same pile number but different arrangements, a 10% difference still existed in the group factors. It indicated that pile arrangement still had some effects on the ultimate soil pressure acting on the passive piles.

Chapter 6

3-D Finite Element Analysis of Single Pile Model Tests

6.1 Introduction

Although 2-D plane-strain geometry has been assumed to simulate the behaviour of piles subjected to lateral soil movements by many researchers, there are some limitations of the 2-D approach. For cases when an assumed soil movement is applied only in a horizontal plane (Lane and Griffiths 1988; Chen 1994; Bransby 1995) with uniform initial stress and boundary conditions, factors such as ground heave, pile flexibility and pile fixity conditions which have significant influence on the behaviour of lateral loaded piles are not taken into account. Another common approach is to analyse the piles in plane strain with an assumed soil movement in a vertical plane (Randolph and Springman 1991; Steward et al. 1993) with each pile row replaced by a sheet pile wall. Consequently, the relative soil displacement past the piles cannot be modelled. Therefore, in order to better understand the mechanism of complicated soil-pile interaction, a series of 3-D finite element analyses were performed in this study.

This chapter presents the 3-D back analyses of the single pile model tests described in Chapter 4. Further analyses of full-scale single piles and pile groups are presented in Chapter 7. Back analyses of two case studies from the literature are described in Chapter 8.

6.2 Finite Element Program

The finite element program ABAQUS/Standard version 6.3 (Hibbitt, Karlsson and Sorensen, Inc. 2002) running on a SUN Workstation was used for all analyses. The same program was also used to conduct 3-D analysis of lateral loaded piles by Trochanis (1988), Brown and Shie (1990a, b) and Pan (1998). The software package

ABAQUS/CAE (Hibbitt, Karlsson and Sorensen, Inc. 2002) was used to perform pre-processing and post-processing. The program provides a versatile capacity for numerical analyses, including a large number of material models, a variety of procedures to perform total stress and effective stress analysis and a geostatic state (initial stress) that is established at the beginning of the analysis.

6.3 Back Analysis for Model Test S-1

In this section, the 3-D back analysis for Model Test S-1, i.e. a single circular pile with the diameter of 25 mm, is described. The details presented include 3-D numerical geometry and boundary conditions, mesh, element types, material constitutive laws, and increment scheme as well as numerical results.

6.3.1 3-D Geometry and Boundary Conditions

Figure 6.1 3-D geometry and boundary conditions for Test S-1

For the back analysis of model tests, the geometry and boundary conditions corresponded to the dimensions and conditions of the sampling box. Figure 6.1 shows the 3-D finite element geometry and boundary conditions used for Test S-1, i.e. a circular pile with a diameter of 25 mm subjected to uniform lateral soil

movement. Because of symmetry, only half of the problem was considered. The surface BFGC represented the symmetrical face with symmetrical constraints ($U_2 = 0$, in which U_2 is the displacement of the 2-direction). The surface AEHD was restrained from moving in the 2-direction ($U_2 = 0$), and the surfaces CGHD and BFEA were restrained from moving in the 3-direction ($U_3 = 0$). A uniform soil displacement y was applied incrementally to the surface ABCD, while the surface EFGH was unrestrained. The pile was 150 mm long with an outer diameter $d = 25$ mm and a wall thickness $t = 3.5$ mm. Two inner nodes S, one at the head and the other at the tip of the pile, were restrained from moving in any direction ($U_1 = U_2 = U_3 = 0$) so as to restrain the pile at the head and tip, as in the model test set-up. Surface-based contacts, i.e. contact pairs, were used to simulate interaction problems between soil and pile, and are described in Section 6.3.3. As the soil moved past the pile, the stresses acting on the perimeter of the pile would change from its initial uniform state to a non-uniform state. The total lateral force P acting on the pile was determined from the total reaction forces in the 1-direction of the nodes S. Thereafter, the average lateral soil pressure p , which was defined as the lateral force P divided by the pile diameter d multiplied by the pile length l , i.e. $p = P/(l \times d)$, was determined.

6.3.2 3-D Mesh

The 3-D finite element mesh used for the analysis was optimised taking into account the computing capacity limitations of the Sun Workstation. The swept meshing technique provided by ABAQUS/CAE to mesh complex solid and surface regions was used. The adopted 3-D mesh for Test S-1 is shown in Figure 6.2. Eight elements were used in each horizontal plane to model the half-circular shape of the pile. Wakai and Ugai (1995) have shown that this mesh refinement was sufficiently accurate. Six elements in the vertical plane were used both for the pile and the soil. An additional analysis was performed using a similar mesh with 12 elements in the vertical plane. The differences in the results of these two analyses were marginal, which indicated that the six elements thick mesh in the vertical plane could provide sufficient accuracy. The mesh consisted of a total of 959 elements, 4,585 nodes and 13,092 degrees of freedom. The run-time was approximately 3 hours (CPU time).

Figure 6.2 3-D mesh for Test S-1

6.3.3 Element Types

The pile and the soil were modelled using 20-node quadrilateral brick elements. Reduced integration with 8 integration points (element type C3D20R) was used instead of full integration with 27 integration points (element type C3D20) to minimise computational time. In addition, C3D20R elements are reported to generally yield more accurate results than the corresponding fully integrated elements C3D20 (Hibbitt, Karlsson and Sorensen, Inc. 2002). For compatibility between the soil-pile interface elements, 27-node quadrilateral brick elements with reduced integration (C3D27R) were used to model the soil around the pile adjacent to the soil-pile interface.

The interface element (defined as contact pairs in ABAQUS) was assumed to have a zero initial thickness; thus, it was basically a two-dimensional element. This element consisted of a 9-node surface, which was compatible with the 9-node side of the 27-node brick element of the soil, and another 8-node surface, which was compatible with the 8-node side of the 20-node brick element representing the pile, as shown in Figure 6.3.

Figure 6.3 Interface element

6.3.4 Material Constitutive Models

Pile model

The pile was assumed to be linear elastic. The Young's modulus E_p and Poisson's ratio ν_p of the pile were $E_p = 2.1 \times 10^8$ kPa and $\nu_p = 0.3$, respectively, to represent the stainless steel pile used in the model tests.

Soil model

A Mohr-Coulomb elastic-plastic constitutive model with nonassociated flow rule using the large-strain mode analysis was used for the kaolin clay. Since the loading rate in the model tests was as fast as 1.05 mm/min, total stress analysis with undrained conditions was assumed. Therefore, the value of cohesion was chosen to match the undrained shear strength ($c_u = 24$ kPa) measured in laboratory tests. Since both the dilation angle ψ and friction angle ϕ were specified as zero, the yielding of the clay strata was actually governed by the Tresca criterion. The elastic modulus of the soil E_s was assumed as constant with depth and was determined by a constant E_s/c_u ratio of 200, which was close to the tangent modulus obtained from the stress-strain relationship of unconsolidated-undrained triaxial tests as described in Chapter 4. The Poisson's ratio ν_s of the soil was assumed to be 0.495 to simulate undrained conditions. The total unit weight of soil γ_s and the coefficient of earth pressure at rest K_0 were assumed as 16.4 kN/m³ and 1.0, respectively, to generate initial stresses.

Interface model

The concepts used in the implementation of interface elements, i.e. contact pairs (Figure 6.3), in ABAQUS are quite simple. An average interface surface is constructed from the average of the coordinates of corresponding nodes on the two opposite surfaces of the interface element, and a local system is defined for recording the shear and pressure stresses between the surfaces when in contact. Contact between two opposite interface nodes is checked by computing the clearance between the two nodes in a specified direction. Positive clearance means that separation has occurred at the interface and no forces are transmitted from one surface to the other. In contrast, when the two surfaces are in contact, the classical Coulomb's friction theory is used, as shown in Figure 6.4. Two contacting surfaces can carry shear stresses up to a certain magnitude, i.e. critical shear stress, across their interface before they start sliding relative to one another; this state is known as sticking. The Coulomb friction model defines this critical shear stress as a fraction of the contact pressure between the surfaces. The fraction μ known as the friction coefficient was assumed equal to $\tan 22.6^\circ$, i.e. the soil effective friction angle measured in the CU tests (described in Chapter 4). This is discussed later in the parametric study in Section 6.4.

At the same time, to more realistically model the interface and to add numerical stability, a default value of elastic slip, ξ_i , was used which allowed a limited amount of shear deformation before slippage took place (Trochanis 1988; Brown and Shie 1991). The default value, $\xi_i = 0.005 l_i$, generally works very well, providing a conservative balance between efficiency and accuracy (Hibbitt, Karlsson and Sorensen, Inc. 2002), in which l_i is the characteristics contact surface length.

Figure 6.4 The friction model

6.3.5 Increment Scheme

An automatic increment scheme in ABAQUS was used for all analyses. As described in Figure 6.1, a uniform soil displacement was applied incrementally to the surface ABCD. The soil movement step was automatically subdivided appropriately if convergence to a desirable small tolerance in solving the non-linear stiffness equations was not reached in a priori specified number of iterations.

6.3.6 Numerical Results for Test S-1

Figure 6.5 shows the computed normalised p - y curve for Test S-1. The curve was obtained from the reaction forces of the pile nodes, as mentioned in Section 6.3.1 (denoted by the symbol S in Figure 6.1). The shape of the computed normalised p - y curve agreed well with that of the model test. The computed normalised ultimate soil pressure p_u/c_u (using the definition described in Section 4.3.1) was 10.2 ($y/d = 0.6$). This was 9.7% lower than the p_u/c_u of 11.3 ($y/d = 0.73$) for the model test.

Figure 6.5 Computed normalised p - y curve for Test S-1

The plots of the deformed mesh and plastic strain contours at failure are shown in Figure 6.6. The plan view of incremental soil displacement vectors at selected displacements is shown in Figure 6.7. These figures clearly showed the gap (void) developed at the back of the pile and the affected zone at the front of the pile, which were consistent with those observed in the model test (shown in Photo 4.1). The near semicircular affected zone observed in the numerical analysis was about $2d$ at $y/d = 0.6$, which was slightly smaller than that of about $2.5d$ at $y/d = 1.0$ in the model test. However, the limitation of this numerical model was that the cracks observed on the sides of the pile in the model test (shown in Photo 4.1) could not develop.

(a) Plan view

(b) General view

Figure 6.6 Deformed mesh and plastic strain contours for Test S-1 ($y/d = 0.6$)

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(a) $y/d = 0.006$

(b) $y/d = 0.06$

(c) $y/d = 0.6$

Figure 6.7 Plan view of incremental soil displacement vectors for Test S-1

6.4 Back Analysis for Model Test S-2

A similar 3-D back analysis was carried out for Model Test S-2, in which the pile was square with a width $d = 25$ mm and a wall thickness $t = 3.5$ mm instead of the circular pile in the Model Test S-1. The adopted 3-D mesh for Test S-2 is shown in Figure 6.8. Because of symmetry, only half of the problem was considered. Eight elements were used in each horizontal plane to model the pile, and six elements in the vertical plane were used both for the pile and the soil. A total of 1,033 elements, 5,064 nodes and 14,685 degrees of freedom were used, and the computational time was approximately 3 hours (CPU time). The same boundary conditions, element types and material properties used for Test S-1 were adopted.

Figure 6.8 3-D mesh for Test S-2

Figure 6.9 shows the computed normalised p - y curve for Test S-2. The shape of the computed normalised p - y curve agreed well with that of the model test. The computed normalised ultimate soil pressure p_u/c_u was 11.0 ($y/d = 0.6$). This was 12.7% lower than the p_u/c_u of 12.6 ($y/d = 0.66$) for the model test. The computed p_u/c_u of the single square pile was 8% higher than that of the single circular pile.

This agreed well with the model test results, which showed that a square pile gave a larger ultimate soil pressure than a circular pile.

Figure 6.9 Computed normalised p - y curve for Test S-2

The plots of the deformed mesh and plastic strain contours at failure are shown in Figure 6.10. The plan view of incremental soil displacement vectors at selected displacements is shown in Figure 6.11. These figures clearly showed the gap (void) developed at the back of the pile and the affected zone at the front of the pile, which were consistent with those observed in the model test (shown in Photo 4.2). The near semicircular affected zone observed in the numerical analysis was about $2.3d$ at $y/d = 0.6$, which was slightly smaller than that of about $3d$ at $y/d = 1.0$ in the model test. Comparing Figure 6.10 with Figure 6.6, there was a larger affected zone in front of the square pile, which was in agreement with the results that a square pile gave a larger ultimate soil pressure than a circular pile.

(a) Plan view

(b) General view

Figure 6.10 Deformed mesh and plastic strain contours for Test S-2 ($y/d = 0.6$)

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(a) $y/d = 0.006$

(b) $y/d = 0.06$

(c) $y/d = 0.6$

Figure 6.11 Plan view of incremental soil displacement vectors for Test S-2

6.5 Parametric Studies

Parametric studies were performed to examine the effects of various soil parameters on the p - y curves as well as the effects of using a linear elastic soil model instead of the Mohr-Coulomb elastic-plastic soil model. The numerical analyses were carried out for the single circular pile with a diameter of 25 mm (shown in Figure 6.1) and the material properties used were as described in Section 6.3.4. The parameters varied included the soil elastic modulus E_s , the undrained shear strength c_u and the friction coefficient μ at the soil-pile interface.

6.5.1 Effect of E_s

Poulos (1973) has noted that ideally E_s should be back calculated from the results of a full-scale in-situ lateral load test or plate load tests at various depths. In practice, E_s is usually estimated on the basis of correlations with strength properties, or from SPT or CPT values. The following empirical expression has been suggested for soft clay (Poulos 1995):

$$E_s = 150 \sim 400c_u \quad (c_u = \text{the undrained shear strength}) \quad (6-1)$$

To examine the influence of soil elastic modulus on the p - y curves, additional analyses were carried out for $E_s = 400c_u$ and $E_s = 100c_u$, respectively, while the other parameters remained unchanged. For the analysis in Section 6.3, $E_s = 200c_u$ was assumed. The computed results are shown in Figure 6.12. It was found that E_s influenced the initial slope of the p - y curves, but had little effect on the normalised ultimate soil pressure p_u/c_u . Similar findings were also reported by Chen and Poulos (1993) and Pan (1998) in their finite element analyses. The results also indicated that the influence of E_s on the p - y curves and consequently the pile behaviour might be significant in situations where the magnitude of the lateral soil movement was small (less than $y/d = 0.3$).

Figure 6.12 Influence of E_s on normalised p - y curves

6.5.2 Effect of c_u

To examine the influence of the undrained shear strength on the p - y curves, additional analyses were carried out for $c_u = 12$ kPa and $c_u = 48$ kPa, respectively. For the analysis in Section 6.3, $c_u = 24$ kPa was assumed. The E_s/c_u ratio was assumed to be 200 and other parameters remained unchanged. The computed results are shown in Figure 6.13. It was found that c_u had little effect on the normalised p - y curves so long as the E_s/c_u ratio remained the same.

Figure 6.13 Influence of c_u on normalised p - y curves

6.5.3 Effect of μ

To examine the influence of the friction coefficient at the soil-pile interface, μ was decreased from the original value of 1.0μ to 0.5μ and 0, i.e. representing full friction (rough), half friction and zero friction (smooth), respectively. The computed normalised p - y curves are shown in Figure 6.14. There was a 4% difference between zero and full friction, and minimal difference between half and full friction. In comparison with the exact analytical solution presented by Randolph and Houlsby (1984) in which the ultimate soil pressure varied from $9.14c_u$ (smooth) to $11.94 c_u$ (rough), this study indicated that the lateral pile response was not sensitive to the friction coefficient used at the interface, so long as the frictional behaviour was assumed. Similar results (using ABAQUS) have also been reported by Brown and Shie (1991) and Pan (1998) using similar interface elements to investigate the behaviour of laterally loaded piles. This may be one of the limitations of this program.

Figure 6.14 Influence of μ on normalised p - y curves

6.5.4 Effect of Soil Constitutive Model

An additional analysis was also carried out using a linear elastic soil model with a constant E_s/c_u ratio of 200. The computed results together with those using the Mohr-Coulomb elastic-plastic soil model and from the model test are shown in Figure 6.15. For y/d less than about 0.002 (i.e. until $p \approx 4.5c_u$), all three curves were almost identical. Then with increased soil displacement, the normalised p - y curves of the Mohr-Coulomb soil model and the model test exhibited non-linear behaviour, while the normalised p - y curve of the elastic model remained linear. The results indicated that the elastic model could give good predictions only at very small displacements ($y/d \leq 0.002$). At large displacements, the use of a linear elastic model would result in an overestimation of the lateral soil pressure acting on the pile.

Figure 6.15 Influence of soil constitutive model on normalised p - y curves

6.6 Summary

A series of 3-D finite element analyses were carried out to back analyse the single pile model tests. The computed normalised ultimate soil pressures were in good agreement with those measured from the model tests. The results also indicated that a square pile gave a larger ultimate soil pressure than a circular pile. The computed deformed mesh, plastic strain contours and incremental soil displacement vector plots were consistent with the observed behaviour from the model tests.

From the parametric study, the following conclusions can be drawn:

- 1) The soil elastic modulus E_s influenced the initial slope of the normalised p - y curves, but had little effect on the normalised ultimate soil pressure. In situations where the magnitude of the lateral soil movement was small, the influence of E_s on the pile behaviour might be significant.
- 2) The undrained shear strength c_u had little effect on the normalised p - y curves so long as the E_s/c_u ratio remained the same.

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- 3) With the present interface model, the p - y curves were not sensitive to the friction coefficient μ at the soil-pile interface, so long as the frictional behaviour was assumed.
- 4) The elastic soil model could give good prediction only at very small displacements ($y/d \leq 0.002$). At large displacements, the use of a linear elastic model would result in an overestimation of the lateral soil pressure acting on the pile.

Chapter 7

3-D Finite Element Analysis of Full-Scale Piles

7.1 Introduction

As mentioned in Chapter 3, the present experimental apparatus has been designed to apply a uniform rectangular profile of lateral soil movements on single piles and pile groups to determine the ultimate soil pressure. Only steel piles with a diameter of 16 mm or 25 mm and an embedded length of 150 mm were considered in the model tests. However, in practice, the majority of piles have larger diameters or widths and can be either rigid or flexible depending on the pile length. The behaviour of piles subjected to lateral soil movements is influenced by various conditions, such as the pile fixity conditions, the magnitude of soil movement and the shape of the translating soil movement profile. The soil pressure acting on the piles is dependent on the soil movement and may not reach the ultimate value measured in the model tests if the piles are only subjected to a small magnitude of lateral soil movement. Moreover, the soil pressure varies along the pile shaft and may not reach the ultimate value along the entire length of the pile shaft, particularly for the flexible piles.

Therefore, further 3-D analyses with larger/longer (full-scale) piles are desirable to investigate the response of single piles and pile groups subjected to lateral soil movement in clay. Unlike the back analyses of the model tests in Chapter 6 when the surface of the soil was restrained from moving vertically (to simulate the model tests conditions), the movement of the soil ground surface was unrestrained in the full-scale pile analyses. These numerical analyses can provide insights into not only the soil pressure, but also the lateral deflection, the shear force and the bending moment distributions along the pile shaft. In addition, these numerical analyses can be used to verify the ultimate soil pressure results found in the model tests as well.

The 3-D finite element analysis of a full scale pile is based on the same soil model in the analysis for the experimental tests in Chapter 6. However, as the c_u is affected

by the deformation mode, boundary conditions, rate of loading, confining stress level and other variables, the c_u is unlikely to be constant for a full-scale problem. Hence, the use of a simple elastic-perfectly plastic model with a single c_u value over a soil thickness of 25 m for a full scale analysis below will give findings that are at best indicative only and this needs to be recognised in realistic case studies.

In this Chapter, 3-D analyses of full-scale single piles are presented. Subsequently, 3-D analyses of full-scale pile groups are also presented.

7.2 Full-Scale Single Piles

This section firstly presents the 3-D model and mesh of a full-scale single circular pile subjected to lateral soil movements. Then the effects of the pile flexibility, the magnitude of soil movement, the pile head boundary conditions and the shape of the translating soil movement profile are examined.

7.2.1 3-D Model and Mesh

Hypothetical three-dimensional finite element studies were conducted to investigate the behaviour of a single circular pile undergoing lateral soil movements. The adopted mesh is shown in Figure 7.1.

Figure 7.1 3-D mesh of a single pile undergoing lateral soil movements

Because of symmetry, only half of the problem was considered. For simplicity, a uniform translating soil movement of up to $0.45d$ (pile diameter, $d = 1$ m) was considered and applied incrementally to the left and right boundaries, which were $9d$ away from the centre of the pile in order to minimise the boundary effects. This uniform translating soil movement represents a simplified upper bound condition for passive piles in actual field situations. The effects of the shape of the translating soil movement profile are presented in Section 7.2.5. In the 3-D model, the ground surface of the soil was unrestrained, while the bottom surface of the soil was restrained from moving vertically. Restrained pinned-head and pinned-tip conditions were assumed for the pile. Eight elements were used for each horizontal section to model the half-circular shape of the pile, and nine elements in the vertical plane were used both for the pile and the soil. The element types for the soil, the pile and the interface were the same as those described in Section 6.3.3. A total of 1,115 elements, 5,043 nodes and 14,160 degrees of freedom were used for this analysis, and the computational time was approximately 5 hours (CPU time).

The pile was modelled as linear elastic. Five types of pile, i.e. a rigid pile ($K_R = 0.1$), a relatively stiff pile ($K_R = 0.01$), a pile of medium flexibility ($K_R = 0.001$), a flexible pile ($K_R = 1 \times 10^{-4}$), and a very flexible pile ($K_R = 1 \times 10^{-5}$), were used for analyses. K_R is the flexibility factor as defined in Equation (3-1). The embedded length L , diameter d , Poisson's ratio ν_p and unit weight γ_p of all these piles were $L = 25$ m, $d = 1$ m, $\nu_p = 0.3$ and $\gamma_p = 25$ kN/m³, respectively.

The Mohr-Coulomb elastic-plastic constitutive model with nonassociated flow rule using the large-strain mode analysis was used for the clay. Total stress analysis with undrained conditions was assumed. The parameters used for the clay are listed in Table 7.1. The friction coefficient μ of the soil-pile interface was assumed as equal to $\tan 22.6^\circ$. The water table was kept constant at the ground surface.

Table 7.1 Summary of clay parameters

Parameter	Value
Undrained shear strength, c_u (kPa)	55
Dilation angle, ψ ($^\circ$)	0
Friction angle, ϕ ($^\circ$)	0
Soil elastic modulus, $E_s = 200c_u$ (kPa)	11,000
Poisson's ratio, ν_s	0.495
Total unit weight, γ_s (kN/m ³)	16.4
Coefficient of earth pressure at rest, K_0	1.0

7.2.2 Effect of Relative Pile Flexibility

Figure 7.2 and 7.3 show the deflected pile shape of the rigid pile ($K_R = 0.1$) and the pile of medium flexibility ($K_R = 0.001$) corresponding to the lateral soil movement of $0.45d$, respectively. Because the zero-tension interface elements were used in the analyses, the numerical analyses resulted in gaps (void) between the pile and the soil along the entire depth for both the rigid pile and the pile of medium flexibility. This is unlikely to occur in reality.

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Figure 7.4 shows the distribution of pile deflections, shear forces, bending moments and normalised soil pressures along the pile shaft for $K_R = 0.1$, $K_R = 0.01$, $K_R = 0.001$, $K_R = 1 \times 10^{-4}$, and $K_R = 1 \times 10^{-5}$. In the figure, z is the depth below the soil surface, ρ is the pile deflection, S_f is the shear force, M is the bending moment and p_u is the ultimate soil pressure obtained from numerical analysis, i.e. $p_u = 10.2c_u$ (shown in Figure 6.5). The shear force for each pile section was calculated from the following equation:

$$S_f = \sum_1^8 \tau_i \cdot A_i \quad (7-1)$$

in which τ_i is the horizontal shear stresses at the centroid of the pile elements and A_i is the area of each small zone, as shown in Figure 7.5.

Figure 7.2 Deflected shape of the rigid pile ($K_R = 0.1$, $y/d = 0.45$)

**Figure 7.3 Deflected shape of the pile of medium flexibility
($K_R = 0.001$, $y/d = 0.45$)**

Figure 7.4 Effects of relative pile flexibility ($y/d = 0.45$)

Figure 7.5 View of pile element segments

The distribution of the bending moments along the pile shaft was obtained indirectly by integrating the area of the shear force diagram. Because restrained pinned-head and pinned-tip conditions were assumed for the pile, the bending moments were taken as zero at the top and bottom of the pile. The normalised soil pressure along the pile shaft was obtained from the stresses at the centroid of the interface elements at the same level, which are shown as a function of circumferential direction around the pile in Figure 7.6. A circumferential direction of 0° corresponded to the front face of the pile in the direction of soil movement. Since gapping occurred behind the pile, all of the stresses beyond 90° were zero. The normalised soil pressure at a given depth was calculated by integrating the area under each curves multiplied by the direction cosine function. It should be noted that the soil pressures on the pile varied during the process of soil movement. They also depended on the relative displacements between the pile and the soil. When the soil movements exceeded the pile deflection, this resulted in induced forces on the pile shaft and consequently soil pressures acting on the pile.

The results in Figure 7.4 indicated that the behaviour of the passive piles was significantly influenced by the pile flexibility. For a pile with restrained pinned-head and pinned-tip conditions, the maximum bending moment increased with increased K_R , and the maximum pile deflection decreased with increased K_R .

For the rigid pile ($K_R = 0.1$), there was no deflection at the top and bottom of the pile because of the assumed pile fixity conditions. The pile deflections along the pile

shaft were very small, with $\rho_{\max} = 0.005d$ when $y = 0.45d$. The maximum bending moment occurred at the middle of the pile shaft. The normalised soil pressure p/c_u at the ground surface was 2.5, which was close to the value of 2 suggested by Poulos (1995) for passive pile design. The low value at the ground surface was due to the near-surface effects, i.e. ground surface heave, as shown in Figure 7.7 and 7.8. With increased depth, the normalised soil pressure p/c_u increased rapidly to 10.2 at the depth of 4 m ($z/L = 0.16$). Thereafter, the p/c_u remained constant with increased depth. The constant value was due to the small deflection of the rigid pile shaft and almost constant large relative displacement between the pile and the soil.

For the relatively stiff pile ($K_R = 0.01$), the pile deflections were also small with $\rho_{\max} = 0.05d$ at the middle of the pile shaft. The maximum bending moment also occurred at the middle of the pile shaft. Up to a depth of 4 m ($z/L = 0.16$), the curve of the normalised soil pressures was similar to that of the rigid pile ($K_R = 0.1$). The p/c_u was 2.5 at the ground surface and increased rapidly to 10.2 at the depth of 4 m. Thereafter, with increased depth, the p/c_u decreased gradually to 9.3 at the middle section of the pile shaft and then increased again to 10.2. The slightly lower soil pressure was due to the smaller relative displacement between the pile and the soil at the middle section of the pile shaft, in comparison to the case with $K_R = 0.1$.

For the pile of medium flexibility ($K_R = 0.001$), the maximum pile deflection $\rho_{\max} = 0.38d$ occurred at the middle of pile shaft. The maximum bending moment also occurred at the middle of the pile shaft. The curve of the normalised soil pressures was similar to that of the relatively stiff pile ($K_R = 0.01$). The p/c_u was 2.5 at the ground surface and increased rapidly to 10.2 at the depth of 4 m ($z/L = 0.16$). A smaller value of $p/c_u = 4.5$ occurred at the middle section of the pile shaft.

For the flexible pile ($K_R = 1 \times 10^{-4}$), the maximum pile deflection at the middle of pile shaft was $0.45d$, which was equal to the value of the applied soil movement ($y = 0.45d$). It indicated that the middle section of the pile almost moved together with the translating soil, and there was minimal relative displacement between the pile and the soil. The bending moments along the pile shaft were small, with the maximum bending moment at $z/L = 0.2$ and $z/L = 0.8$. The p/c_u was also 2.5 at the

ground surface and increased rapidly to 8.6 at the depth of 1 m ($z/L = 0.04$). The p/c_u decreased rapidly from 8.8 at $z/L = 0.16$ to zero at $z/L = 0.4$. There was zero soil pressure from $z/L = 0.4$ to $z/L = 0.6$. Subsequently, the p/c_u increased gradually to 10.3 at the tip of the pile. The zero soil pressure from $z/L = 0.4$ to $z/L = 0.6$ was because there was minimal relative displacement between the pile and the soil at the middle sections of the pile shaft.

For the very flexible pile ($K_R = 1 \times 10^{-5}$), the pile deflections from $z/L = 0.28$ to $z/L = 0.76$ were approximately $0.45d$. It also indicated that there was minimal relative displacement between the pile and the soil. This resulted in zero soil pressure from $z/L = 0.28$ to $z/L = 0.76$. The bending moments along the pile shaft were very small, with the maximum bending moment at $z/L = 0.2$ and $z/L = 0.8$. The curve of the normalised soil pressures was similar to that of the flexible pile ($K_R = 1 \times 10^{-4}$).

(a) $K_R = 0.1$

(b) $K_R = 0.001$

Figure 7.6 Interface stresses at selected depths ($y/d = 0.45$)

Figure 7.7 Deformed mesh close to ground surface ($y/d = 0.45$)

Figure 7.8 Ground surface heave ($y/d = 0.45$)

7.2.3 Effect of Magnitude of Soil Movement

To examine the effects of the magnitude of the translating soil movement y , the responses of the passive piles for $y = 0.15d$, $0.3d$, $0.45d$ and $0.6d$ were compared. Only the rigid pile ($K_R = 0.1$) and the pile of medium flexibility ($K_R = 0.001$) with restrained pinned-head and pinned-tip conditions were considered. Figures 7.9 and 7.10 show the variation in deflection, shear force, bending moment and normalised soil pressure distributions with various magnitudes of soil movement for the rigid pile and the pile of medium flexibility, respectively.

For the rigid pile ($K_R = 0.1$), the pile deflections, shear forces and bending moments increased as y increased. The normalised soil pressures p/c_u at the ground surface were nearly the same and close to 2.5. Subsequently, the p/c_u increased rapidly to a maximum value at $z/L = 0.16$ and then remained nearly constant with increasing depth. For y larger than $0.45d$, the maximum soil pressure did not change. It indicated that the ultimate value had been reached. The ultimate values for the rigid piles were in good agreement with those from FEM (described in Chapter 6) and those from the model tests.

For the pile of medium flexibility ($K_R = 0.001$), the pile deflections, shear forces and bending moments also increased as y increased. The normalised soil pressures p/c_u at the ground surface were nearly the same and close to 2.5. There were significant differences of the p/c_u at the middle section of the pile shaft for different y values. For $y = 0.15d$, the soil pressure was zero at the middle section of the pile shaft and the pile displacement followed the soil movement almost exactly, and in consequence, minimal bending moments were developed. As y increased, the relative displacement between the pile and the soil in the middle section of the pile shaft increased. A maximum soil pressure p/c_u of 6.5 occurred when $y = 0.6d$. It can be seen that the soil pressures reached the ultimate value measured in the model tests only for sections close to the top and bottom of the pile and only when the soil movement was larger than $0.45d$. These indicated that for the pile of medium flexibility with pinned-head and pinned-tip conditions, the soil pressures acting on most of the pile shaft were

significantly less than the ultimate value obtained from the model tests or from plastic theory for rigid piles (Randolph and Houlsby 1984).

Figure 7.11 shows the normalised p - y curves at selected depths z for the rigid pile ($K_R = 0.1$). All the p - y curves except for that at $z = 0$ were almost identical. They were linear until $p/c_u \approx 4.5$. Then the p/c_u increased gradually with increased soil displacement. The soil pressure reached the ultimate value and remained almost constant when $y/d = 0.45$. The p - y curve at $z = 0$ differed significant from others. The p/c_u increased linearly to 2.4 at $y/d = 0.02$ and then remained almost constant. As mentioned in Section 7.2.2, the low value of p/c_u at the ground surface was due to the near-surface effects.

Figure 7.12 shows the normalised p - y curves at selected depths z for the pile of medium flexibility ($K_R = 0.001$). Only the p - y curves at $z = 0$ and 25 m were similar to those of the rigid pile. The p/c_u at $z = 4\sim 22$ m were always lower than those of the rigid pile corresponding to the same y/d . The lower p/c_u was due to the smaller relative displacement between the soil and the pile at each specified depth.

Figure 7.9 Effects of magnitudes of soil movement for the rigid pile ($K_R = 0.1$)

Figure 7.10 Effects of magnitudes of soil movement for the pile of medium flexibility ($K_R = 0.001$)

Figure 7.11 Normalised p - y curves at selected z for the rigid pile ($K_R = 0.1$)

Figure 7.12 Normalised p - y curves at selected z for the pile of medium flexibility ($K_R = 0.001$)

7.2.4 Effect of Pile Head Boundary Conditions

To examine the effects of the pile head boundary conditions, a 3-D analysis was carried out on the pile of medium flexibility ($K_R = 0.001$) with free head and pinned-tip condition subjected to a uniform soil movement $y = 0.45d$. Figure 7.13 shows the results of the pile of medium flexibility with different pile head boundary conditions. It was found that the provision of the head restraint reduced the pile movements near the surface, but also increased the shear forces and bending moments. For the free head pile, the movement of the top of pile was slightly larger than the surface lateral soil movement, resulting in “negative” soil pressures, i.e. soil pressures acting on the opposite side of the pile, on the upper part of the pile shaft. The maximum bending moment occurred at $z/L = 0.7$, which was lower than that for the pinned-head pile. It can also be seen that the soil pressures for the free-head pile reached the ultimate value only in the lower portion of the pile ($z/L \geq 0.88$). As in Section 7.2.3, the results also indicated that the soil pressures acting on most of the pile shaft ($K_R = 0.001$) were significantly less than the ultimate value.

Figure 7.13 Effects of pile head boundary conditions ($K_R = 0.001$, $y/d = 0.45$)

7.2.5 Effect of Shape of Soil Movement Profile

In the previous analyses, a uniform lateral soil movement profile was assumed in order to examine the major factors that influence the pile behaviour and represented the upper bound conditions. In actual field situations, the soil movements may be non-uniform. Therefore, an additional trapezoidal soil displacement profile was considered to investigate the effects of soil movement distributions, as shown in Figure 7.14. Only the pile of medium flexibility ($K_R = 0.001$) with free head and pinned-tip conditions was considered. Comparisons were also made with the 2-D boundary element results by Poulos and Davis (1980), as shown in Figure 7.15.

Figure 7.14 Types of soil movement profile at the boundary

For the pile of medium flexibility ($K_R = 0.001$), the head movement was largely dependent on the magnitude of the soil surface movement. The maximum shear force and bending moment for the pile subjected to a rectangular soil movement were larger than those for the trapezoidal profile. The maximum bending moment for the trapezoidal soil movement profile occurred at $z/L = 0.6$, which was higher than the $z/L = 0.7$ for the rectangular soil movement profile. As with the analyses described in Section 7.2.4, the movements of the top of the free head piles for both types of soil movement profiles were slightly larger than the surface soil movement, resulting in “negative” soil pressures on the upper part of the pile shaft.

The results presented by Poulos and Davis (1980) are also shown in Figure 7.15 (a) and (c), for the case of $L/d = 25$, $\nu_s = 0.5$, $K_R = 0.001$ and $E_s/p_u = 10$. The present

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results were reasonable compared with those by Poulos and Davis (1980). Generally, the shapes of the normalised pile deflection and bending moment profiles were similar. The movements of the top of the piles for the present 3-D analyses were slightly larger than the 2-D analyses by Poulos and Davis (1980). There was a large discrepancy in the maximum pile bending moments between the present analyses and those of Poulos and Davis (1980). These differences may be because various soil parameters were used and the present analyses took into account the shear stresses developed between the soil and the pile as well as ground surface heave.

Figure 7.15 Effects of shape of soil movement profiles ($K_R = 0.001$, $y/d = 0.45$)

7.2.6 Effect of Thickness of Moving Soil Mass

In the previous analyses, the assumed uniform soil movement was applied along the entire embedded length of the pile. In actual field situations, for instance, piles in moving slopes, only the upper part of the pile may be subjected to lateral soil movements, as shown in Figure 7.16. Therefore, additional analyses were carried out for $z_s/L = 0.28, 0.4, 0.52$ and 0.64 to study the influence of the thickness of the moving soil mass z_s . Only the rigid pile ($K_R = 0.1$) with pinned head and pinned-tip conditions was considered, and the magnitude of soil movement y was $0.45d$.

Figure 7.16 Soil movement profile with moving layer

Figure 7.17 shows the results for different z_s . It was found that the z_s had a significant influence on the deflection profiles and the bending moment profiles. The maximum pile deflection and bending moment increased as z_s increased. The normalised soil pressures p/c_u at the ground surface for different z_s were nearly the same and close to 2.5. However there were significant differences of the p/c_u between $z_s/L < 1.0$ and $z_s/L = 1.0$. Because of the stable soil layer, the pile displacement resulted in “negative” soil pressures, i.e. soil pressures acting on the opposite side of the pile, on the lower part of the pile shaft. As z_s increased, the portion of the lower part of the pile that was subjected to “negative” soil pressure decreased, and the p/c_u curve became closer to that of $z_s/L = 1.0$.

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Figure 7.17 Effects of thickness of the moving soil mass ($K_R = 0.1$, $y/d = 0.45$)

7.3 Full-Scale Pile Groups

The response of full-scale 2-pile groups with different arrangements, i.e. two piles in a row and two piles in a line, subjected to lateral soil movements are investigated in this section. The limitations of the present 3-D model used for the analyses of pile groups are also discussed.

7.3.1 Two Piles in a Row

Hypothetical three-dimensional finite element studies were conducted to investigate the behaviour of two piles in a row undergoing lateral soil movements. The adopted mesh is shown in Figure 7.18. S_h is the pile centre-centre spacing for two piles in a row.

Figure 7.18 Mesh for two piles in a row undergoing lateral soil movements

Because of symmetry, only half of the problem was considered. Only rigid piles ($K_R = 0.1$) with pinned-head and pinned-tip conditions subjected to a uniform soil movement $y = 0.45d$ were considered. The boundary conditions, types of elements, material properties were the same as those described in Section 7.2.1. A total of 1,814 elements, 7,803 nodes and 21,585 degrees of freedom were used for the analyses, and the computational time was approximately 6 hours (CPU time).

Figure 7.19 shows the 3-D FEM results for two piles in a row with spacings of $2d$, $3d$ and $6d$. Figure 7.20 shows the comparison between the group factors obtained from the FEM results and those from the experimental model tests. The experimental results presented in Section 5.3.1 showed that the group factors increased with increased pile spacing and there was approximately a 20% reduction in the group factor for two piles with a spacing of $3d$ in comparison to two piles with a spacing of $6d$. Similar results were also obtained from the experimental tests by Chen and Poulos (1997) and Pan et al. (2002). However, the 3-D FEM results showed little reduction of group factors with decreasing pile spacing. The 3-D FEM results were consistent with the 2-D FEM analyses by Bransby (1995) and 3-D analyses by Brown and Shie (1990b). The overestimation of group factors from the 3-D FEM results could be due to the limitations of the elastic-plastic Mohr-Coulomb clay model used in this study. As shown in Figure 7.21, although obvious differences of accumulated plastic strain were developed in front of the pile, the plastic zone intersected the line of symmetry for $2d$ and $3d$ spacings and there was no intersection for the $6d$ spacing, the soil was still able to move around the pile smoothly. The actual cracks in the soil between piles that were observed in the model tests (shown in Photo 5.1) were not developed in the finite element analyses. It may be that the cracks developed in the soil between piles causes a redistribution of stresses, and in consequence, a reduction of the normalised ultimate soil pressures acting on the piles.

Figure 7.19 Effects of pile spacing for two piles in a row ($K_R = 0.1, y/d = 0.45$)

Figure 7.20 Comparison between FEM and experimental results for two piles in a row

(a) $S_h = 2d$

(b) $S_h = 3d$

(c) $S_h = 6d$

Figure 7.21 Plastic strain contours for two piles in a row at various spacings

7.3.2 Two Piles in a Line

Hypothetical three-dimensional finite element studies were also conducted to investigate the behaviour of two piles in a line undergoing lateral soil movements. The adopted mesh is shown in Figure 7.22. S_v is the pile centre-centre spacing for two piles in a line.

Figure 7.22 Mesh for two piles in a line undergoing lateral soil movements

Because of symmetry, only half of the problem was considered. Only rigid piles ($K_R = 0.1$) with pinned-head and pinned-tip conditions subjected to a uniform soil movement $y = 0.45d$ were considered. The boundary conditions, types of elements, material properties were the same as those described in Section 7.2.1. A total of 2,218 elements, 9,697 nodes and 26,949 degrees of freedom were used for the analyses, and the computational time was approximately 10 hours (CPU time).

Figures 7.23, 7.24 and 7.25 show the 3-D results for two piles in a line with spacings of $3d$, $4.5d$ and $6d$, respectively. Figure 7.26 shows the comparison

between the group factors obtained from the FEM results and those from the experimental model tests. Both the FEM analyses and experimental tests showed the same trend that the F_{pi} of the “front” pile was larger than that of the “back” pile. The FEM results showed little reduction in the group factors of the “front” pile for decreasing pile spacing and some reduction in the group factors of the “back” pile for decreasing pile spacing. However, the experimental results showed more significant reduction in the group factors both for the “front” pile and “back” pile. The higher group factors from the 3-D FEM results may be due to the limitations of the elastic-plastic Mohr-Coulomb clay model used in this study. A comparison of the plastic strain contours for two piles in a line with spacing $3d$ from FEM analysis (Figure 7.27) with the failure pattern for Test G-3 (shown in Photo 5.3) indicated that the observed shear failure developed between two piles in a line in the model tests was not developed in the finite element analysis. As suggested by Fleming et al (1985), and also discussed in Section 5.9.2, the shear failure between piles in a line would govern when the shearing resistance of the soil between the piles was less than the ultimate resistance of an isolated pile. Therefore, the differences of the FEM results and the experimental results indicated that the present 3-D model could not successfully model the soil shear failure between two piles in a line, and resulted in the overestimation of the group factors.

Figure 7.23 Behaviour of two piles in a line with $S_v = 3d$ ($K_R = 0.1$, $y/d = 0.45$)

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Figure 7.24 Behaviour of two piles in a line with $S_v = 4.5d$ ($K_R = 0.1$, $y/d = 0.45$)

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Figure 7.25 Behaviour of two piles in a line with $S_v = 6d$ ($K_R = 0.1, y/d = 0.45$)

Figure 7.26 Comparison between FEM and experimental results for two piles in a line

Figure 7.27 Plastic strain contours for two piles in a line with $S_v = 3d$

7.4 Summary

As a follow up to the 3-D back analyses of the single pile model tests described in Chapter 6, further 3-D analyses were conducted to investigate the response of full-scale single piles and piles groups subjected to lateral soil movements in clay.

From the analyses of full-scale single piles, the following conclusions can be drawn:

- 1) The behaviour of the passive piles was significantly influenced by the pile flexibility factor K_R . For a pile with restrained pinned-head and pinned-tip conditions, the maximum bending moment increased with increased K_R , and the maximum pile deflection decreased with increased K_R . The normalised soil pressures p/c_u at the ground surface for different K_R were almost the same and were about 2.5 due to the near-surface effects. The p/c_u depended on the relative displacements between the pile and the soil. For the pile of $K_R \leq 0.001$, the small relative displacements resulted in the small soil pressures acting on most of the pile shaft.

Based on the soil distribution profiles for the pile of $K_R \geq 0.01$ (Figures 7.4 and 7.9), a simplified limiting soil pressure profile can be used for preliminary design (for example, in p - y type analysis), as shown in Figure 7.28. This profile is only applicable for $z_s = L$ and for piles with pinned-head and pinned-tip conditions.

Figure 7.28 Design pressure profile for the pile of $K_R \geq 0.01$

- 2) The behaviour of the passive piles was significantly influenced by the magnitude of the translating soil movement y . The pile deflections, shear forces and bending moments increased as y increased. The ultimate soil pressures p_u/c_u for the rigid piles were in good agreement with those from the model tests. The p/c_u acting on most of the pile shaft for the pile of $K_R \leq 0.001$ was significantly less than the ultimate value obtained from the model tests or from plastic theory for the rigid piles.
- 3) The behaviour of the passive piles was significantly influenced by the pile head boundary conditions. The provision of the head restraint reduced the pile movements near the surface, but also increased the shear forces and bending moments. For the free head pile of medium flexibility, the movement of the top of pile was slightly larger than the surface soil movement, resulting in “negative” soil pressure on the upper part of the pile shaft.
- 4) The behaviour of the passive pile was significantly influenced by the shape of soil movement profiles. The maximum shear force and bending moment in the pile for a rectangular soil movement profile were larger than those for a trapezoidal profile.

- 5) The behaviour of the passive piles was significantly influenced by the thickness of the moving soil mass z_s . For the pile with pinned head and pinned-tip conditions, the maximum pile deflection and bending moment increased as z_s increased.

From the analyses of full-scale groups, the following conclusions can be drawn:

- 1) The present 3-D elastic-plastic Mohr-Coulomb constitutive model could not successfully model the pile-soil-pile interaction behaviour for the soil between piles in a group, and thereby overestimated the group factors.
- 2) For two piles in a row, the 3-D FEM results showed little reduction of group factors with decreasing pile spacing. The actual cracks in the soil between piles observed in the model tests were not developed in the finite element analyses. It may be that the cracks developed in the soil between piles causes a redistribution of stresses, and in consequence, a reduction of the normalised ultimate soil pressures acting on the piles
- 3) For two piles in a line, the group factor F_{pi} of the “front” pile was larger than that of the “back” pile. However, the observed shear failure developed between two piles in a line in the model tests was not developed in the finite element analysis. As the shear failure between piles in a line would govern when the pile spacing was small, this resulted in significant reduction of the normalised soil pressures acting on the piles for the model tests in comparison to the FEM analyses.

Chapter 8

Case Studies

8.1 Introduction

This chapter describes the back analyses of two case studies from the literature using the 3-D finite element program ABAQUS. The purpose of these case studies is: (a) to further validate the present 3-D model to predict full-scale test data; and (b) to further examine the soil-pile interaction behaviour from the field tests.

As mentioned in Chapter 7, the present 3-D elastic-plastic Mohr-Coulomb constitutive model could not successfully model the pile-soil-pile interaction behaviour for the soil between piles in a group, and thereby overestimated the group factors. Therefore, only the behaviour of single piles was considered in this Chapter.

8.2 Case 1

8.2.1 Background

Carrubba et al. (1989) reported a case where a reinforced concrete pile was used to stabilise a sliding slope. The 1.2 m diameter test pile of 22 m length was instrumented with pressure cells along its shaft and an inclinometer at its centre. The sliding slope had a sliding surface at a depth of 9.5 m from the ground surface, as indicated by inclinometer data. The pile developed a plastic hinge at a depth of 12.5 m below the ground surface, as indicated by pressure cell data. No information was given regarding the soil movement profile to cause the pile to yield. The undrained shear strength for both the moving soil layer and the stable layer was 30 kPa, as estimated from unconsolidated-undrained triaxial tests.

Chen and Poulos (1997) using a boundary element program PALLAS made the following assumptions to match the measured bending moments:

- a) The elastic modulus of soil, $E_s = 500 c_u = 15$ MPa (uniform with depth);
- b) The soil movement was uniform from the ground surface down to a depth of 9.5 m, with a magnitude of 95 mm.
- c) The Young's modulus of pile E_p was 20,000 MPa and the bending stiffness $E_p I_p$ was 2035.8 MN-m².

In the present 3-D analysis using ABAQUS, E_s was assumed as $200c_u$, i.e. 6 MPa; E_p was assumed as 38,950 MPa, and the wall thickness of pipe pile was assumed as 0.1 m so that the $E_p I_p$ of pile was still 2035.8 MN-m²; K_R was 1.4×10^{-3} (Equation 3-1); the soil movement profile was taken to be the same as Chen and Poulos (1997).

8.2.2 3-D Model and Mesh

Figure 8.1 shows the adopted mesh for Case 1. Only half of the problem was considered because of symmetry. A uniform translating soil movement of 95 mm was applied incrementally to the left and right upper boundaries from the ground surface down to a depth of 9.5 m. The left and right boundaries from 9.5 m to the bottom surface were restrained from moving horizontally. The ground surface of the soil was unrestrained, while the bottom surface of the soil was restrained from moving vertically. Free-head and free-tip conditions were assumed for the pile. Eight elements were used for each horizontal section to model the half-circular shape of the pile, and ten elements in the vertical plane were used both for the pile and the soil. The element types for the soil, the pile and the interface were the same as those described in Section 6.3.3.

The pile was modelled as linear elastic. The Mohr-Coulomb elastic-plastic constitutive model with nonassociated flow rule using the large-strain mode analysis was used for the clay. Total stress analysis with undrained conditions was assumed. The parameters used for the pile and the soil are listed in Table 8.1. The water table was assumed to be constant at the ground surface.

Figure 8.1 3-D mesh of Case 1

Table 8.1 Parameters used for Case 1

Parameter	Value
Young's modulus of pile, E_p (MPa)	38,950
Wall thickness of pile, t , (m)	0.1
Embedded length of pile, L (m)	22
Diameter of pile, d (m)	1.2
Poisson's ratio of pile, ν_p	0.3
Unit weight of pile, γ_p (kN/m ³)	25
Undrained shear strength, c_u (kPa)	30
Dilation angle, ψ (°)	0
Friction angle, ϕ (°)	0
Soil elastic modulus, $E_s = 200c_u$ (MPa)	6
Poisson's ratio of soil, ν_s	0.495
Total unit weight of soil, γ_s (kN/m ³)	16
Coefficient of earth pressure at rest, K_0	1.0
Friction coefficient of the soil/pile interface, μ	$\tan 22.6^\circ$

8.2.3 Results

The predicted results are shown in Figure 8.2, together with the field measurements and the results of Chen and Poulos (1997). Reasonably good agreement between the predictions and the measurements was observed for the deflection, the shear force, the bending moment and the distributed lateral load profiles. The results of Chen and Poulos (1997) were closer to the field measurements since the soil elastic modulus E_s was chosen to match the measured bending moment profile. The pile deflection profile showed that most of the upper pile portion moved with the moving soil. The pile head deflected more than the soil at the surface, which resulted in “negative” soil pressures on the upper part of the pile shaft. The maximum soil pressure was $5.5c_u$, which was obtained from the maximum distributed lateral load divided by $c_u \times d$. It indicated that the soil pressure acting on the pile was less than the ultimate value.

It should be pointed out that the Case 1 study has the following limitations: since this case study was for a concrete pile stabilising a slope, the Young's modulus of concrete needed to be matched with load duration; also no information on soil movement profile was available. Hence, the results from the 3-D analysis of this case study are indicative only. However, because of the lack of other reliable full-scale instrumented cases from the literature, Case 1 was selected and reasonable results were obtained using the assumptions suggested by Chen and Poulos (1997).

Figure 8.2 Predicted results for Case 1

8.2.4 Influence of z_s

To study the influence of the thickness of the moving soil mass, z_s (shown in Figure 8.1), four additional analyses were carried out in which z_s was 1.9 m, 2.85 m, 3.8 m and 22 m, respectively. The z_s had a significant influence on the predicted deflection and the bending moment profiles, as shown in Figure 8.3. It should be noted that free-head and free-tip pile boundary conditions were assumed for z_s of 1.9 m, 2.85 m, 3.8 m and 9.5 m, while free-head and pinned-tip pile boundary conditions were assumed for z_s of 22 m. Consequently, the lower portion of the pile tilted backwards for z_s of 1.9 m, 2.85 m, 3.8 m and 9.5 m, while the deflection of pile tip was zero for z_s of 22 m. It was also found that the pile head deflected less than the soil for z_s of 1.9 m, 2.85 m and 3.8 m, while the pile head deflected more than the soil for z_s of 9.5 m and 22 m. The maximum bending moment increased significantly when z_s increased from 1.9 m ($z_s/L = 0.085$) to 3.8 m ($z_s/L = 0.17$), but the increases were not as significant for $z_s > 3.8$ m.

—△— $z_s = 1.9$ m —○— $z_s = 2.85$ m —■— $z_s = 3.8$ m —▲— $z_s = 9.5$ m —×— $z_s = 22$ m

Figure 8.3 Influence of z_s

8.3 Case 2

8.3.1 Background

Kalteziotis et al. (1993) reported a case where two rows of piles were used to stabilise a sliding slope on which a semi-bridge construction had been built (Figure 8.4). Two years after the completion of the semi-bridge construction, some cracks appeared in the road-pavement and a rather large inclination of the wing-wall of the bridge was observed. The site consisted mainly of neogene lacustrine deposits, of a thickness of more than one hundred meters, overlying bedrock of Triassic marl. The lacustrine deposits included alternating layers of conglomerates and clayey or sandy marls with lenses or thin intercalations of sandstones. Laboratory tests on samples obtained from borings showed that most of material could be classified as low to medium plasticity clays or high plasticity clays.

Figure 8.4 Plan view of the sliding zone and location of the instrumented piles and inclinometers (after Kalteziotis et al. 1993)

In order to protect piers which were supporting the semi-bridge from being further damaged by the sliding, two rows of 1 m diameter concrete bored piles of 12 m length were installed in the vicinity of the foundation piers at the downhill area, as shown in Figure 8.4. The centre-to-centre spacing of the piles was 2.5 m. Two of these piles, one at each row, i.e. AM1 and AV2 in Figure 8.4, were replaced by steel pipe piles with an external diameter of 1.03 m, thickness of 18 mm and bending stiffness $E_p I_p$ of 1540 MNm^2 , which was about the same as that of the concrete bored piles. These two steel piles were instrumented with strain gauges and inclinometers (P1, P2, N1, N2) in order to study the lateral reaction mechanism in a landslide, but results were presented only for one of them, i.e. the AM1 pile in the front row. At the same time, two inclinometer tubes, i.e. G12 and G13, were installed at the uphill area of the experimental site to monitor the displacements at the level of the semi-bridge.

The lateral soil movement profile recorded by inclinometer G13 is shown in Figure 8.5. These measurements indicated that the sliding surface was about 4 m below the surface and these values could be taken to be the soil movements in the analysis.

Figure 8.5 Measured lateral soil movement profiles (after Kalteziotis et al. 1993)

In the present approach, c_u was taken to be 100 kPa and 356 kPa for the moving soil layer and the stable layer respectively, as suggested by Chow (1996) from pressuremeter data. The elastic modulus of soil E_s was assumed as $200c_u$, i.e. 20 MPa and 71.2 MPa for the moving soil layer and the stable layer, respectively. The Poisson's ratio of the soil ν_s was assumed as 0.495.

8.3.2 3-D Model and Mesh

Figure 8.6 shows the adopted mesh for Case 2. As mentioned above, the results of field measurements were presented only for the pile in the front row, i.e. the AM1 pile in Figure 8.4. From the experimental results of Tests G-3, G-10 and G-13 (2, 4 and 6 piles in two-row groups) described in Chapter 5, it was found that the behaviour of the front pile in two-row groups was close to a single isolated pile. Therefore, the measured front pile was considered as a single isolated pile in this analysis. The same assumptions were also taken by Chen and Poulos (1997) in their analysis. A uniform soil movement of 7.5 mm from the surface down to 4 m was assumed and applied $9d$ away from the centre of the pile since it was close to the measured lateral soil movement profile on 22 October 1992 shown in Figure 8.5. The boundary conditions were the same as those for Case 1. Eight elements were used for each horizontal section to model the half-circular shape of the pile, and six elements in the vertical plane were used both for the pile and the soil. The element types for the soil, the pile and the interface were the same as those described in Section 6.3.3.

The pile was modelled as linear elastic. The Mohr-Coulomb elastic-plastic constitutive model with nonassociated flow rule using the large-strain mode analysis was used for the clay. Total stress analysis with undrained conditions was assumed. The parameters used for the pile and the soil are listed in Table 8.2. The water table was assumed to be constant at the ground surface.

Figure 8.6 3-D mesh of Case 2

Table 8.2 Parameters used for Case 2

Parameter	Value
Young's modulus of pile, E_p (MPa)	210,000
Wall thickness of pile, t , (m)	0.018
Embedded length of pile, L (m)	12
Diameter of pile, d (m)	1.03
Poisson's ratio of pile, ν_p	0.3
Unit weight of pile, γ_p (kN/m ³)	25
Undrained shear strength, c_u (kPa)	100 and 356
Dilation angle, ψ (°)	0
Friction angle, ϕ (°)	0
Soil elastic modulus, $E_s = 200c_u$ (MPa)	20 and 71.2
Poisson's ratio of soil, ν_s	0.495
Total unit weight of soil, γ_s (kN/m ³)	16
Coefficient of earth pressure at rest, K_0	1.0
Friction coefficient of the soil/pile interface, μ	$\tan 22.6^\circ$

8.3.3 Results

The predicted results are shown in Figure 8.7, together with the comparison with the field measurements and the results of Chow (1996). Reasonably good agreement between the predictions and the measurements was observed for the deflection, the shear force, the bending moment and the distributed lateral load profiles. The predicted maximum shear force was 50 kN at a depth of 3 m, while the measured value was 30 kN at a depth of 2.75 m. The predicted maximum bending moment was 140 kN-m at a depth of 5 m, while the measured value was 150 kN-m at a depth of 6 m. The predicted maximum soil pressure was $0.3c_u$, which was obtained from the maximum distributed lateral load divided by $c_u \times d$. It indicated that the soil pressure acting on the pile was significantly less than the ultimate value.

Figure 8.7 Predicted results for Case 2

8.4 Summary

From the above case studies, the following conclusions can be drawn:

- 1). The 3-D finite element analyses using the elastic-plastic Mohr-Coulomb soil model can generally give reasonably good predictions of the single pile behaviour, including both the magnitudes and locations of the maximum bending moment and the pile deflection.
- 2). For free head piles subjected to lateral soil movements, the z_s significantly affected the pile deflection profile. For Case 1, the maximum bending moment increased significantly when z_s/L increased from $z_s/L = 0.085$ to 0.17 , but the increases were not as significant for $z_s/L > 0.17$.
- 3). The soil pressure acting on the pile was dependent on the magnitude of soil movement and did not reach the ultimate value since the piles were only subjected to relatively small lateral soil movements in these two case studies.

Chapter 9

Simplified Method for Estimating Maximum Pile Bending Moment

9.1 Introduction

Although 3-D finite element analysis can handle complicated problems and obtain useful results, it is still computational expensive to perform and it requires a skilled user to set up the 3-D model, select the key parameters and extract the solutions. In practical design, charts or equations may be more convenient for the designer, particularly when there is a lack of detailed site information. Therefore, a simplified method to estimate the maximum pile bending moment has been developed based on the experimental and numerical results described in previous chapters.

9.2 Proposed Design Method

Based on the experimental results described in Chapter 4 and 5, and numerical results described in Chapter 6, 7 and 8, a simplified method was developed to predict the maximum bending moment for piles subjected to lateral soil movements. As shown in Figure 7.9(c), the maximum bending moment for a single rigid pile ($K_R = 0.1$) with pinned head and tip conditions subjected to a rectangular soil movement of $0.6d$ was approximately $0.005 \times p_u \times L^3$. Since the maximum bending moment M_{max} of passive piles is influenced by other factors such as the pile flexibility, magnitude of the soil movement, the pile head boundary conditions, soil movement profile, and the thickness of moving soil mass, modifications factors have been developed and the following equation is proposed:

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$$M_{max} = F_{pi} \cdot \mu_f \cdot \mu_m \cdot \mu_r \cdot \mu_s \cdot \mu_z \cdot 0.005 \cdot p_u \cdot L^3 \quad (9-1)$$

in which

F_{pi} is the pile group factor,

μ_f is the pile flexibility reduction factor,

μ_m is the magnitude of soil movement factor,

μ_r is the pile head boundary condition factor,

μ_s is the soil movement profile factor,

μ_z is the thickness of moving soil mass z_s (shown in Figure 7.16) factor,

p_u is the ultimate soil pressure, and

L is the embedded length of the pile.

This method assumes that the magnitude of the free field soil movement, the soil movement profile and the thickness of moving soil mass are known. The following describes how the modification factors are determined.

F_{pi} is obtained from the experimental results of single piles and pile groups subjected to lateral soil movements, as defined in Equation (5-1) and listed in Table 5.12. For easy reference, it is presented in Figure 9.1.

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Group factors in bold for $S_h = 3d$ or $S_v = 3d$.

^a() Group factors for $S_h = 6d$ or $S_v = 6d$, ^b() Group factors for $S_v = 4.5d$.

Figure 9.1 Group factors F_{pi}

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The pile flexibility reduction factor μ_f is shown in Figure 9.2. It was derived from Figure 7.4 based on the maximum bending moment for the piles with different K_R values. In the figure, E_p is the Young's modulus of the pile, I_p is the moment of inertia of the pile and E_s is the elastic modulus of the soil. In the present approach, E_s is assumed to be constant with depth to be consistent with previous assumptions made in the numerical analyses. The value of μ_f was obtained by dividing the value of the maximum bending moment for any specified K_R by that for $K_R = 0.1$. It is found that the μ_f is significantly influenced by the K_R . The μ_f is small and less than 0.11 for $K_R \leq 1 \times 10^{-4}$. The μ_f exceeds 0.75 when $K_R > 0.001$.

Figure 9.2 Modification factor for μ_f

The magnitude of soil movement factor μ_m can be obtained from the simplified tri-linear curve in Figure 9.3 or from Equations (9-2) to (9-4). The FEM curve in Figure 9.3 was derived from the computed normalised p - y curves for Test S-1 (Figure 6.5). The value of μ_m was obtained by dividing the value of the normalised soil pressure at any specified magnitude of normalised soil movement (y/d) by the ultimate value (for simplicity, assumed as $p_u = 10c_u$). The proposed simplified curve is a simple approximation to the FEM curve and can be mathematically expressed as:

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$$\mu_m = 10 \cdot y/d, \quad \text{when } y/d \leq 0.05 \quad (9-2)$$

$$\mu_m = 0.9 \cdot y/d + 0.45, \quad \text{when } 0.6 \geq y/d > 0.05 \quad (9-3)$$

$$\mu_m = 1.0, \quad \text{when } y/d > 0.6 \quad (9-4)$$

The μ_m increases linearly from zero to 0.5 at $y/d = 0.05$. The μ_m then increases gradually from 0.5 to 1.0 at $y/d = 0.6$, and thereafter the μ_m is constant and has reached the ultimate value.

Figure 9.3 Modification factor for μ_m

The pile head boundary condition factor μ_r is 0.5 for a free-head and pinned-tip pile and 1.0 for a pinned-head and pinned-tip pile. The values of μ_r were obtained from Figure 7.13 based on the maximum bending moment for the piles with different head conditions. Only pinned pile tips were considered as it has been assumed that the piles were embedded into a stiffer stratum, due to the limited end bearing capacity in the soft clay.

To account for the shape of the soil movement profile, μ_s is 0.5 for a trapezoidal profile and 1.0 for a rectangular profile. The values of μ_r were obtained from Figure

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7.15 based on the maximum bending moment for the piles with different soil movement profiles.

The factor μ_z accounts for the thickness of moving soil mass z_s (Figure 7.16), and is shown in Figure 9.4. The curves were derived from Figure 7.17 and Figure 8.3 based on the maximum bending moment for the pinned-head pile and free-head pile with different values of z_s . The values of μ_z were obtained by dividing the value of the maximum bending moment for any specified value of z_s/L by that for $z_s/L = 1.0$. It is found that the z_s is more significant for the pinned-head pile. The μ_z increases almost linearly with increased z_s/L . For the free head pile, the μ_z increases significantly from 0.26 at $z_s/L = 0.085$ to 0.88 at $z_s/L = 0.17$, but the increases are not significant for $z_s/L > 0.17$.

Figure 9.4 Modification factor for μ_z

p_u is the ultimate soil pressure, which for simplicity is assumed as:

$$\text{Clay: } p_u = 10c_u \quad (9-5)$$

in which c_u is the average undrained shear strength of the moving soil mass. Although this thesis has focused on the behaviour of the passive piles in clay, two preliminary case analyses in the following section using this method were also carried out for

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piles in sand. Because of the limited guidelines in the literature on the ultimate soil pressure for passive piles in sand, the following recommendation by Goh et al. (1997) was adopted to estimate the ultimate soil pressure for sand:

$$\text{Sand: } p_u = p_L \quad (9-6)$$

in which p_L is the limiting pressure from pressuremeter tests. Typical values of p_L proposed by Briaud (1992) are shown in Table 9.1.

Table 9.1 Typical pressuremeter parameters for sand (Briaud 1992)

Material	Characteristic	p_L (kPa)
Sand	Loose	0-500
	Medium dense	500-1,500
	Dense	1,500-2,500
	Very dense	>2,500

A flow chart outlining the above procedure for determination of the maximum pile bending moment is shown in Figure 9.5.

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Figure 9.5 Procedure for determination of maximum pile bending moment

9.3 Validation

The design method proposed in Section 9.2 was used to estimate the maximum pile bending moment for five cases from the literature. In Case 1, 2 and 3, the free-head single piles embedded in clay slopes were analysed. Case 4 and 5 involved the pinned-head single piles embedded in sand adjacent to embankment construction.

9.3.1 Analysis of Case 1

The background of Case 1 is described in detail in Section 8.2. The relevant parameters are $L = 22$ m, $d = 1.2$ m, $z_s = 9.5$ m, $E_p I_p = 2,035.8$ MN·m², $y = 0.095$ m, $c_u = 30$ kPa and $E_s = 200c_u = 6$ MPa. Following the procedure outlined in Figure 9.5, this gives $F_{pi} = 1.0$ for the single pile; $K_R = 2035.8 \times 10^3 / (200 \times 30 \times 22^4) = 1.4 \times 10^{-3}$ and $\mu_f = 0.75$ from Figure 9.2; $y/d = 0.095/1.2 = 0.08$ and $\mu_m = 0.9 \times 0.08 + 0.45 = 0.52$ from Equation (9-3); $\mu_r = 0.5$ for the free-head pile; $\mu_s = 1.0$ for the rectangular profile of the soil movement; $z_s/L = 0.43$ and $\mu_z = 0.94$ from Figure 9.4. Hence, $M_{max} = 1.0 \times 0.75 \times 0.52 \times 0.5 \times 1.0 \times 0.94 \times 0.005 \times 10 \times 30 \times 22^3 = 2.96$ MN·m > 2.3 MN·m (measured).

9.3.2 Analysis of Case 2

The background of Case 2 is described in detail in Section 8.3. The relevant parameters are $L = 12$ m, $d = 1.03$ m, $z_s = 4.0$ m, $E_p I_p = 1,540$ MN·m², $y = 0.0075$ m, $c_u = 100$ kPa and $E_s = 200c_u = 20$ MPa. Following the procedure outlined in Figure 9.5, this gives $F_{pi} = 1.0$ for the single pile; $K_R = 1,540 \times 10^3 / (200 \times 100 \times 12^4) = 3.7 \times 10^{-3}$ and $\mu_f = 0.79$ from Figure 9.2; $y/d = 0.0075/1.03 = 0.0073$ and $\mu_m = 0.9 \times 0.0073 + 0.45 = 0.46$ from Equation (9-3); $\mu_r = 0.5$ for the free-head pile; $\mu_s = 1.0$ for the rectangular profile of the soil movement; $z_s/L = 0.33$ and $\mu_z = 0.93$ from Figure 9.4. Hence, $M_{max} = 1.0 \times 0.79 \times 0.46 \times 0.5 \times 1.0 \times 0.93 \times 0.005 \times 10 \times 100 \times 12^3 = 0.15$ MN·m $= 0.15$ MN·m (measured).

9.3.3 Analysis of Case 3

Esu and D'Elia (1974) reported a case where a reinforced concrete pile was installed into a sliding slope. The slope consisted mainly of clay and the upper layer of 7.5 m thickness was subjected to lateral movement. The pile was 30 m long, 0.79 m in diameter and the bending stiffness ($E_p I_p$) was 360 MN·m². The pile was instrumented to determine the bending moments.

Chen and Poulos (1997) using a boundary element program PALLAS made the following assumptions to match the measured bending moment:

- $c_u = 40$ kPa, as suggested by Maugeri and Matta (1991);
- The elastic modulus of soil E_s increases linear from zero at the surface to 16 MPa at the pile tip;
- Since the soil movement profile to cause the pile to yield was not reported, a uniform soil movement of 110 mm was assumed from the surface down to a depth of 7.5 m.

In the present approach, E_s was assumed as $200c_u$, i.e. 8 MPa; the soil movement profile was taken to be the same as that assumed by Chen and Poulos (1997).

Therefore, the relevant parameters are $L = 30$ m, $d = 0.79$ m, $z_s = 7.5$ m, $E_p I_p = 360$ MN·m², $y = 0.11$ m, $c_u = 40$ kPa and $E_s = 200c_u = 8$ MPa. Following the procedure outlined in Figure 9.5, this gives $F_{pi} = 1.0$ for the single pile; $K_R = 360 \times 10^3 / (200 \times 40 \times 30^4) = 5.6 \times 10^{-5}$ and $\mu_f = 0.07$ from Figure 9.2; $y/d = 0.11/0.79 = 0.14$ and $\mu_m = 0.9 \times 0.14 + 0.45 = 0.58$ from Equation (9-3); $\mu_r = 0.5$ for the free-head pile; $\mu_s = 1.0$ for the rectangular profile of the soil movement; $z_s/L = 0.25$ and $\mu_z = 0.91$ from Figure 9.4. Hence, $M_{max} = 1.0 \times 0.07 \times 0.58 \times 0.5 \times 1.0 \times 0.91 \times 0.005 \times 10 \times 40 \times 30^3 = 1.0$ MN·m > 0.86 MN·m (measured).

9.3.4 Analysis of Case 4 and 5

De Beer and Wallays (1972) reported a field test at Zelzate in Belgium to study the influence of embankment construction on adjacent pile foundations. The soil deposit

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consisted mainly of sand. Two single piles, a steel pipe pile and a reinforced concrete pile, were measured. For convenience, the steel pile is referred to as Case 4 and the reinforced concrete pile is referred to as Case 5. The steel pipe pile was 28 m in length, 0.9 m in diameter, 15 mm in wall thickness, and the bending stiffness ($E_p I_p$) was $858 \text{ MN}\cdot\text{m}^2$. The reinforced concrete pile was 23.2 m in length, 0.6 m in diameter, and the bending stiffness ($E_p I_p$) was $127 \text{ MN}\cdot\text{m}^2$. The pile head of these two cases was restrained from lateral displacement. The lateral soil movement profile was given, as shown in Figure 9.6.

Figure 9.6 Measured soil movement profile (De Beer and Wallays 1972)

In the present approach, E_s was assumed as 30 MPa and constant with depth, as suggested by Chen and Poulos (1997). Because the moving soil layer consisted mainly of medium dense sand, two different p_L values (from Table 9.1) representing possible lower and upper bound values were considered: $p_{L1} = 1000 \text{ kPa}$ and $p_{L2} = 1500 \text{ kPa}$. Each pile was considered as a single pile with pinned-head and pinned-tip boundary conditions. In this simplified analysis, the soil movement profile was assumed to be trapezoidal with a maximum displacement of 57 mm at the surface.

For Case 4 described above, the relevant parameters are $L = 28 \text{ m}$, $d = 0.9 \text{ m}$, $z_s = 17.5 \text{ m}$ (from Figure 9.6), $E_p I_p = 858 \text{ MN}\cdot\text{m}^2$, $y = 0.057 \text{ m}$, $E_s = 30 \text{ MPa}$, $p_{L1} = 1000 \text{ kPa}$ and $p_{L2} = 1500 \text{ kPa}$. Following the procedure outlined in Figure 9.5, this gives F_{pi}

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= 1.0 for the single pile; $K_R = 858 \times 10^3 / (30 \times 10^3 \times 28^4) = 4.6 \times 10^{-5}$ and $\mu_f = 0.068$ from Figure 9.2; $y/d = 0.057/0.9 = 0.051$ and $\mu_m = 0.9 \times 0.051 + 0.45 = 0.5$ from Equation (9-3); $\mu_r = 1.0$ for restrained pinned-head pile; $\mu_s = 0.5$ for the trapezoidal profile of the soil movement; $z_s/L = 0.63$ and $\mu_z = 0.52$ from Figure 9.4. Hence, for $p_{L1} = 1000$ kPa, $M_{max1} = 1.0 \times 0.068 \times 0.5 \times 1.0 \times 0.5 \times 0.52 \times 0.005 \times 1000 \times 28^3 = 0.97 \text{ MN}\cdot\text{m} < 1.25 \text{ MN}\cdot\text{m}$ (measured). For $p_{L2} = 1500$ kPa, $M_{max2} = 1.0 \times 0.068 \times 0.5 \times 1.0 \times 0.5 \times 0.52 \times 0.005 \times 1500 \times 28^3 = 1.46 \text{ MN}\cdot\text{m} > 1.25 \text{ MN}\cdot\text{m}$ (measured).

For Case 5 described above, the relevant parameters are $L = 23.2$ m, $d = 0.6$ m, $z_s = 17.5$ m (from Figure 9.6), $E_p I_p = 127 \text{ MN}\cdot\text{m}^2$, $y = 0.057$ m, $E_s = 30$ MPa, $p_{L1} = 1000$ kPa and $p_{L2} = 1500$ kPa. Following the procedure outlined in Figure 9.5, this gives $F_{pi} = 1.0$ for the single pile; $K_R = 127 \times 10^3 / (30 \times 10^3 \times 23.2^4) = 1.5 \times 10^{-5}$ and $\mu_f = 0.034$ from Figure 9.2; $y/d = 0.057/0.6 = 0.095$ and $\mu_m = 0.9 \times 0.095 + 0.45 = 0.54$ from Equation (9-3); $\mu_r = 1.0$ for the restrained pinned-head pile; $\mu_s = 0.5$ for the trapezoidal profile of the soil movement; $z_s/L = 0.75$ and $\mu_z = 0.68$ from Figure 9.4. Hence, for $p_{L1} = 1000$ kPa, $M_{max1} = 1.0 \times 0.034 \times 0.54 \times 1.0 \times 0.5 \times 0.68 \times 0.005 \times 1000 \times 23.2^3 = 0.38 \text{ MN}\cdot\text{m} > 0.3 \text{ MN}\cdot\text{m}$ (measured). For $p_{L2} = 1500$ kPa, $M_{max2} = 1.0 \times 0.034 \times 0.54 \times 1.0 \times 0.5 \times 0.68 \times 0.005 \times 1500 \times 23.2^3 = 0.58 \text{ MN}\cdot\text{m} > 0.3 \text{ MN}\cdot\text{m}$ (measured).

9.3.5 Discussion of Design Method

The results of the proposed design method are summarised in Table 9.2, together with those based on the design charts of Chen and Poulos (1997). The proposed method generally gave an upper-bound estimation of the maximum pile bending moments for the different cases. Case 4 and 5 were for piles embedded in sand. Provided the p_u for sand can be estimated, the analyses showed that the proposed design method could also be used for sand, although there was a somewhat larger discrepancy for Case 5. The method may be useful for preliminary design when there is a lack of detailed site information.

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Table 9.2 Summary of the results from proposed method

Cases	Reference	Pile head boundary conditions	y/d	$M_{predicted}/M_{measured}$	
				Present	Chen and Poulos (1997)
1	Carrubba et al. (1989)	Free-head	0.08	1.29	1.13
2	Kalteziotis et al. (1993)	Free-head	0.0073	1.0	1.07
3	Esu and D'Elia (1974)	Free-head	0.14	1.16	1.3
4	De Beer and Wallays (1972)	Pinned-head	0.051	^a 0.78	—
				^b 1.17	—
5	De Beer and Wallays (1972)	Pinned-head	0.095	^a 1.28	—
				^b 1.93	—

^a $p_{L1} = 1000 \text{ kPa}$, ^b $p_{L2} = 1500 \text{ kPa}$

The method assumes that the magnitude of the free field soil movement, the soil movement profile and the thickness of soil moving mass are known. This design method can be applied to both the free-head piles and restrained pinned-head piles, while the design charts of Chen and Poulos (1997) can only be applied to free-head piles. However, it should be noted that this proposed method cannot be used for fixed-head or fixed-tip piles.

Only single piles were considered in the analyses of Case 1~5. Unfortunately, there were no reported cases in the literature for passive pile groups in clay for comparisons to be carried out. As mentioned in Section 5.10, the group factors (shown in Figure 9.1) obtained in this study were reasonably consistent with those obtained from the literature. Therefore, these group factors may be used for pile group analyses, but requires further validation with instrumented case studies.

It should be pointed out that in this proposed design method, which only gives a "first-estimate" for preliminary design, particularly when there is a lack of detailed site information, the c_u is assumed as an average value and constant with depth. As mentioned in Chapter 2, the c_u is not a fundamental material property. In addition, the c_u value is unlikely to be constant with depth for a thick soil deposit. Its

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measurement is also affected by the boundary conditions, rate of loading, confining stress level, pile installation, time and other variables. Therefore, further validation of the influence of c_u is desirable.

9.4 Summary

In this chapter, a simplified method has been proposed to estimate the maximum pile bending moment. The method may be useful for preliminary design when there is a lack of detailed site information. The method generally gave an upper-bound estimation of the maximum pile bending moments in the cases considered.

Chapter 10

Conclusions and Suggestions

10.1 Conclusions

In this thesis, experimental and numerical studies have been conducted to investigate the behaviour of pile foundations embedded in soft clay when subjected to lateral soil movements. Based on the above studies, a simplified method to estimate the maximum pile bending moment was developed. Detailed conclusions have been presented in the previous chapters, and the major findings are summarised below.

10.1.1 Experimental Work

A specially designed laboratory apparatus was fabricated and utilised to carry out a series of tests on single piles and pile groups in soft clay undergoing lateral movements. The model piles were instrumented with button load cells to measure the ultimate soil pressure p_u acting on the piles.

For single pile tests, the effects of pile shape and diameter on the ultimate soil pressure were examined. The p_u for single circular piles ranged from $10.8c_u$ to $11.3c_u$. The ultimate soil displacement y_u to fully mobilise the p_u ranged from $0.73d$ to $0.78d$. The p_u for a square pile was 12% higher than that for a circular pile. The p_u for single passive piles was generally independent of pile diameter.

For pile group tests, the p_u acting on an individual pile within a group was different from a single pile. The group factors F_{pi} , in terms of the p_u for individual piles in a group, were generally smaller than 1.0. The group factors F_{pi} of the “front” row piles were generally slightly less than 1.0, and always much larger than those of the “middle” row or “back” row piles. The differences in the F_{pi} between the “middle” row piles and the “back” row piles were less significant, and so were the differences in the F_{pi} between the “side” piles and the “centre” pile in the same row. The group factors F_{yi} , in terms of the ultimate soil displacement for individual piles in a group,

of the “front” row piles were also generally close to 1.0 and always larger than those of the “middle” row or “back” row piles.

The p_u acting on an individual pile within a group differed with different pile spacings, numbers of piles and arrangements of piles. There was an obvious trend that the group factors F_p , in terms of the p_u for the whole group, reduced with an increased number of piles. Pile spacing was a key parameter governing the ultimate soil pressure acting on piles subjected to lateral soil movements. Generally, the group factors F_p increased with increased pile spacing. For two piles in a row with a spacing of $6d$, the F_p was 0.97. It indicated that the piles for a two-pile group in a row behaved like isolated single piles when the pile spacing was $6d$ or larger. For pile groups with a spacing of $3d$ or smaller in clay, shear failure between piles in a line occurred. This may have caused significant reduction of p_u for the “middle” or “back” row piles. For a pile group with the same pile number but different arrangements, a 10% difference still existed in the F_p .

10.1.2 Numerical Work

To better understand the mechanism of complicated soil-pile interaction, a series of 3-D finite element analyses were carried out using the finite element program ABAQUS.

For the back analysis of the single pile model tests, the computed ultimate soil pressures were in good agreement with those measured from the model tests. As with the experimental results, it was also found that a square pile gave a larger ultimate soil pressure than a circular pile. From the parametric study, it was found that the soil elastic modulus influenced the initial slope of the p - y curves, but had little effect on the normalised ultimate soil pressure. The undrained shear strength c_u had little effect on the normalised p - y curves so long as the E_s/c_u ratio remained the same. With the present interface model, the p - y curves were not sensitive to the friction coefficient μ at the soil-pile interface, so long as frictional behaviour was assumed. The elastic soil model could give good predictions only at very small displacements ($y/d \leq 0.002$).

For the analyses of full-scale single piles, the behaviour of the passive piles was significantly influenced by the pile flexibility factor K_R , the magnitude of the translating soil movement y , the pile head boundary conditions, the shape of soil movement profiles and the thickness of moving soil mass z_s . For a pile with restrained pinned-head and pinned-tip conditions, the maximum bending moment increased with increased K_R , and the maximum pile deflection decreased with increased K_R . The soil pressures at the ground surface for different K_R were almost same and were about $2.5c_u$ due to the near-surface effects. The soil pressures acting on most of the pile shaft for the pile of $K_R \leq 0.001$ were significantly less than the ultimate value obtained from the model tests or from plastic theory for the rigid piles. The pile deflections, shear forces and bending moments increased as y increased. The provision of the head restraint reduced the pile movements near the surface, but also increased the shear forces and bending moments. The maximum shear force and bending moment in the pile for a rectangular soil movement profile were greater than those for a trapezoidal profile. For the pile with pinned head and pinned-tip conditions, the maximum pile deflection and bending moment increased as z_s increased.

For the analyses of full-scale pile groups, the present 3-D elastic-plastic Mohr-Coulomb constitutive model could not successfully model the pile-soil-pile interaction behaviour for the soil between piles in a group, and thereby overestimated the group factors. Therefore, it appears that any further research work should be concentrated on the soil constitutive relationship to better model the pile-soil-pile interaction behaviour.

The present 3-D elastic-plastic Mohr-Coulomb soil model was also used to predict single pile behaviour of two cases from the literature. The theoretical predictions were found to be in reasonably good agreement with the measured results.

10.1.3 Proposed Design Method

Based on the experimental results and numerical results, a simplified method has been proposed to estimate the maximum pile bending moment. The method may be

useful for preliminary design when there is a lack of detailed site information. The method generally gave an upper-bound estimation of the maximum pile bending moments in the cases considered.

10.2 Suggestions for Further Research

Although considerable research work has already been carried out to study the behaviour of the piles subjected to lateral soil movements, some uncertainties still remain and therefore further research in the following areas is suggested:

Experimental work

- 1) Conduct large-scale model tests and field tests to validate the findings from this study.
- 2) Extend the present work to clay with different layers and soil properties.
- 3) Extend the present work to sand.

Numerical work

- 1) Investigate soil constitutive relationship to better model the pile-soil-pile interaction behaviour of pile groups.
- 2) Further 3-D analyses and validation of the influence of c_u are desirable since the c_u is significantly affected by factors such as the boundary conditions, rate of loading, confining stress level, pile installation, and time.
- 3) Further validate the simplified design method to estimate the maximum pile bending moment for practical applications, such as in a design scenario where the response of the pile is not known and hence the c_u value cannot be selected from a wide tenable range or in a design scenario where the axial loading may lead to P- δ effect among other effects.
- 4) Simplified soil movement profiles have been used for the laboratory and field cases considered in the present work. Therefore, further analysis on the response of lateral loaded piles in different situations, e.g. pile driving, excavation, tunnelling, embankment, etc, is desirable.

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